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FOUNDATION AND CIVIL ENGINEERING SITE DEVELOPMENT

Site hydrology and land planning are two initial factors that influence land use and foundation design. Part 1 addresses these concerns. Site hydrology involves both subsurface and surface water content and movement. Land planning develops construction techniques intended to accommodate hydrologic problems and provide best use of the parcel. Coverage of the topic will be rather cursory—as a rule, foundation engineers are not involved with the early stages of development, but an awareness of the potential problems is beneficial.

SECTION 1A
WATER BEHAVIOR IN SOILS

ROBERT WADE BROWN

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1A.1 MOISTURE REGIMES

The regime of subsurface water can be divided into two general classifications: the *aeration zone* and the *saturation zone*. The saturation zone is more commonly termed the *water table* or *groundwater*, and it is, of course, the deepest. The aeration zone includes the capillary fringe, the intermediate belt (which may include one or more perched water zones), and, at the surface, the *soil water belt*, often referred to as the *root zone* (Fig. 1A.1). Simply stated, the soil water belt provides moisture for the vegetable and plant kingdoms; the intermediate belt contains moisture essentially in dead storage—held by molecular forces; and the perched ground water, if it occurs, develops essentially from water accumulation either above a relatively impermeable stratum or within an unusually permeable lens. Perched water occurs generally after heavy rain and is relatively temporary. The capillary fringe contains capillary water originating from the water table. The soil belt can contain capillary water available from rains or watering; however, unless this moisture is continually restored, the soil will eventually desiccate through the effects of gravity, transpiration, and/or evaporation. When it does so, the capillary water is lost. The soil belt is also the zone that most critically influences both foundation design and stability. This will be discussed in the following sections. As stated, the more shallow zones have the greatest influence on surface structures. Unless the water table is quite shallow, it will have little, if any, material influence on the behavior of foundations of normal residential structures. Furthermore, the surface of the water table, the *phreatic boundary*, will not normally deflect or deform except under certain conditions, such as when it is in the proximity of a producing well. Then the boundary will *draw down* or recede.

Engineers sometimes allude to a “natural” buildup of surface soil moisture beneath slab foundations due to the lack of evaporation. This phenomenon is often referred to as *center doming* or *cen-*

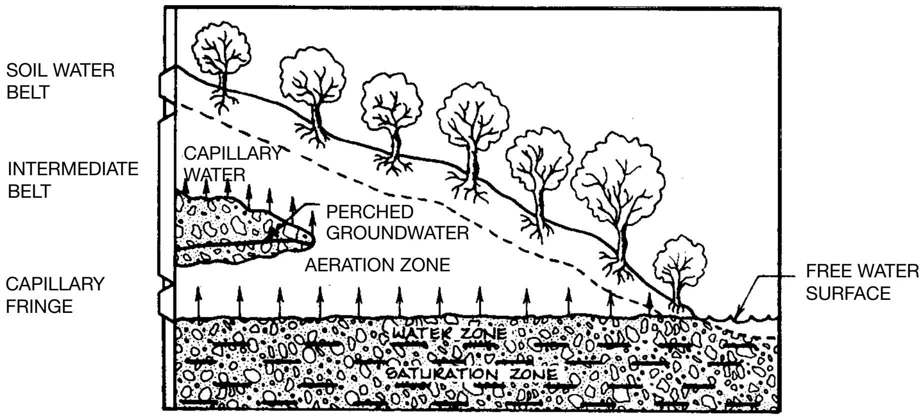

FIGURE 1.A1 Moisture regimes.

ter lift (refer to Sec. 7A.3). If the source for this moisture is assumed to be the water table and if the water table is deeper than about 10 ft (3 m),* the boundary (as well as the capillary fringe) is not likely to "dome"; hence, no transfer of moisture to the shallow soils would be likely. The other source of moisture could involve the capillary or osmotic transfer from underlying soils to the dryer, more shallow soils. When expansive soils are involved, this intrusion of moisture can cause the soil to swell. This swell is ultimately going to be rather uniform over the confined area. (This expansive soil has a much greater lateral than vertical permeability.) Again no "natural doming" is likely to occur. Refer to Sec. 1A.8.

Following paragraphs will provide further discussion concerning water migration in various soils as represented by several noted authorities.

1A.2 SOIL MOISTURE VERSUS WATER TABLE

Alway and McDole [1]† conclude that deep subsoil aquifers (e.g., water table) contribute little, if any, moisture to plants and, hence, to foundations. Upward movement of water below a depth of 12 in (30 cm) was reportedly very slow at moisture contents approximating field capacity. *Field capacity* is defined as the residual amount of water held in the soil after excess gravitational water has drained and after the overall rate of downward water movement has decreased (zero capillarity). Soils at lower residual moisture content will attract water and cause it to flow at a more rapid rate. Water tends to flow from wet to dry in the same way as heat flows from hot to cold—from higher energy level to lower energy level.

Rotmistrov [1] suggests that water does not move to the surface by capillarity from depths greater than 10 to 20 in (25 to 50 cm). This statement does not limit the source of water to the water table or capillary fringe. Richards [1] indicates that upward movement of water in silty loam can develop from depths as great as 24 in (60 cm). McGee [1] postulates that 6 in (15 cm) of water can be brought to the surface annually from depths approaching 10 ft (300 cm). Again, the source of water is not restricted in origin.

The seeming disparity among results obtained by these hydrologists is likely due to variation in

*The abbreviations of units of measure in this book are listed in Appendix C.
†Numbers in brackets indicate references at the end of the sections.

experimental conditions. Nonetheless, the obvious consensus is that the water content of the surface soil tends to remain relatively stable below very shallow depths and that the availability of soil water derived from the water table ceases when the boundary lies at a depth exceeding the limit of capillary rise for the soil. In heavy soils (e.g., clays), water availability almost ceases when the water source is deeper than 4 ft (120 cm), even though the theoretical capillary limit normally exceeds this distance. In silts, the capillary limit may approximate 10 ft (300 cm), as compared to 1 to 2 ft (30 to 60 cm) for sands. The height of capillary rise is expressed by Eq. (1A.1).

$$\pi \gamma_T r^2 h_c = T_{\mathrm{st}}\, 2\pi r \cos \alpha$$

or (1A.1)

$$h_c = \frac{2T_{\mathrm{st}}}{r\gamma_T} \cos \alpha$$

where h_c = capillary rise, cm
T_{st} = surface tension of liquid at temperature T, g/cm
r = radius of capillary pore, cm
α = meniscus angle at wall or angle of contact
γ_T = unit weight of liquid at temperature T, g/cm^2

For behavior in soils, the radius r is difficult, if not impossible, to establish. It is dependent upon such factors as void ratio, impurities, grain size and distribution, and permeability. Since the capillary rise varies inversely with effective pore or capillary radius, this value is required for mathematical calculations. Accordingly, capillary rise, particularly in clays, is generally determined by experimentation. In clays, the height and rate of rise are impeded by the soil's swell (loss of permeability) upon invasion of water. Fine noncohesive soils will create a greater height of capillary rise, but the rate of rise will be slower. More information on soil moisture, particularly that dealing with clay soils, will be found in Parts 6, 7, and 9 of this volume.

1A.3 SOIL MOISTURE VERSUS AERATION ZONE

Water in the upper or aeration zone is removed by one or a combination of three processes: Transpiration, evaporation, and gravity.

1A.3.1 Transpiration

Transpiration refers to the removal of soil moisture by vegetation. A class of plants, referred to as *phreatophytes*, obtain their moisture, often more than 4 ft (120 cm) of water per year, principally from either the water table or the capillary fringe. This group includes such seemingly diverse species as reeds, mesquite, willows, and palms. Two other groups, *mesophytes* and *xerophytes*, obtain their moisture from the soil water zone. These include most vegetables and shrubs, along with some trees. In all vegetation, root growth is toward soil with greater available moisture. Roots will not penetrate a dry soil to reach moisture. The absorptive area of the root is the tip, where root hairs are found. The loss of soil moisture by transpiration follows the root pattern and is generally somewhat circular about the stem or trunk. The root system develops only to the extent necessary to supply the vegetation with required water and nutrition. Roots not accessible to water will wither and die. These factors are important to foundation stability, as will be discussed in following sections.

In many instances, transpiration accounts for greater loss of soil moisture than does *evaporation*.

In another process, *interception*, precipitation is caught and held by foliage and partially evaporated from exposed surfaces. In densely planted areas, interception represents a major loss of rainfall, perhaps reaching as high as 10 to 25% of total precipitation [1].

1A.3.2 Gravity and Evaporation

Gravity tends to draw all moisture downward from the soil within the aeration zone. *Evaporation* tends to draw moisture upward from the surface soil zone. Both forces are retarded by molecular, adhesive, and cohesive attraction between water and soil as well as by the soil's capacity for capillary recharge. If evaporation is prevented at the surface, water will move downward under the forces of gravity until the soil is drained or equilibrium with an impermeable layer or saturated layers is attained. In either event, given time, the retained moisture within the soil will approximate the "field capacity" for the soil in question.

In other words, if evaporation were prevented at the soil surface, as, for example, by a foundation, an "excessive" accumulation of moisture would initially result. However, given sufficient time, even this protected soil will reach a condition of moisture equilibrium somewhere between that originally noted and that of the surrounding uncovered soil. The natural tendency of covered soil is to retain a moisture level above that of the uncovered soil, except, of course, during periods of heavy inundation (rains) when the uncovered soil reaches a temporary state at or near saturation. In this latter instance, the moisture content decreases rapidly with the cessation of rain or other sources of water.

The loss of soil moisture from beneath a foundation caused by unabated evaporation might tend to follow a triangular configuration, with one leg vertical and extending downward into the bearing soil and the other leg being horizontal and extending under the foundation. The relative lengths of the legs of the triangle would depend upon many factors, such as the particular soil characteristics, foundation design, weather, and availability of moisture (Fig.1A.2).

Davis and Tucker [2] reported the depth as about 5 ft (1.5 m) and the penetration approximately 10 ft (3 m). In any event, the affected distances (legs of the triangle) are relatively limited. As with all cases of evaporation, the greatest effects are noted closer to the surface. In an exposed soil, evaporation forces are ever present, provided the relative humidity is less than 100%. The force of gravity is effective whether soil is covered or exposed.

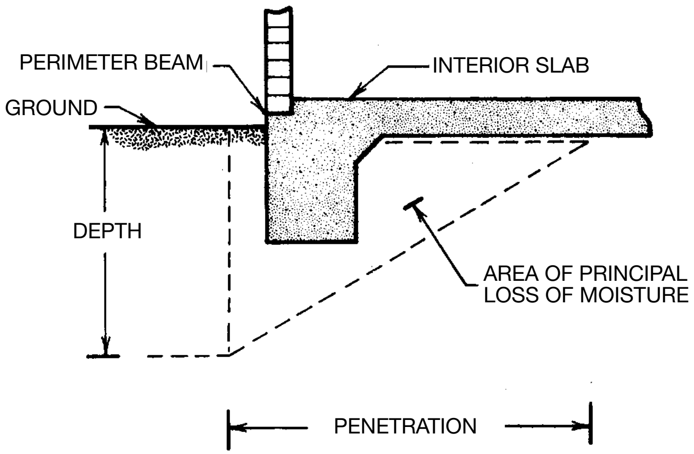

FIGURE 1A.2 Typical loss of soil moisture from beneath a slab foundation during prolonged drying cycle.

1A.4 PERMEABILITY VERSUS INFILTRATION

The infiltration feature of soil is more directly related to penetration from rain or water at the surface than to subsurface vertical movement. The exceptions are those relatively rare instances in which the ground surface us within the capillary fringe. Vertical migration or permeation of the soil by water infiltration could be approximately represented by the single-phase steady-state flow equation, as postulated by Darcy [3].

$$Q = -\left(\frac{Ak}{\mu}\right)\left(\frac{\Delta P}{L} + g_c \sin \alpha\gamma\right) \tag{1A.2}$$

where Q = rate of flow in direction L
A = cross-sectional area of flow
k = permeability
μ = fluid viscosity
$\dfrac{\Delta P}{L}$ = pressure gradient in direction L
L = direction of flow
γ = fluid density
α = meniscus angle at wall or angle of contact = angle of dip ($\alpha > 0$ if flow L is up dip)
g_c = gravity constant

If $\alpha = 90°$, $\sin \alpha = 1$, and, simplified, Eq. (1.2) becomes

$$Q = -\left(\frac{Ak}{\mu L}\right)(\Delta P + g_c h\gamma)$$

where $h = L \sin \alpha$ and $g_c h\gamma$ is the hydrostatic head.

If $H = \Delta P + g_c h\gamma$, where H is the fluid flow potential, then

$$Q = -\left(\frac{Ak}{\mu}\right)\left(\frac{H}{L}\right)$$

When flow is horizontal, the gravity factor g_c drops out. Any convenient set of units may be used in Eq. (1A.2) so long as the units are consistent. Several influencing factors represented in this equation pose a difficult deterrent to mathematical calculations. For example, the coefficient of permeability k can be determined only by experimental processes and is subject to constant variation, even within the same soil. The pore sizes, water saturation, particle gradation, transportable fines, and mineral constituents all affect the effective permeability k.

In the instance of expansive clays, the variation is extremely pronounced and subject to continuous change upon penetration by water. The hydraulic gradient ΔP and the distance over which it acts, ΔL, are also elusive values. For these reasons, permeability values are generally established by controlled field or laboratory tests in which the variables can be controlled. In the case of clean sand, the variation is not nearly as extreme, and reasonable approximations for k are often possible.

In essence, Eq. (1A.2) provides a clear understanding of factors controlling water penetration into soils but does not always permit accurate mathematical calculation. The rate of water flow does not singularly define the moisture content or capacity of the soil. The physical properties of the soil, available and residual water, and permeability each affect infiltration. A soil section 3 ft (90 cm) thick may have a theoretical capacity for perhaps 1.5 ft (0.46 m) of water. This is certainly more water than results from a serious storm; hence, the moisture-holding capacity is seldom, if ever, the limiting criterion for infiltration. That is as it would appear from the foregoing paragraphs.

To better comprehend the variations in the permeability coefficient k, consider the following values, sometimes considered "typical" for various soils (after Terzaghi and Peck, *Soil Mechanics in Engineering*, 2nd Ed., Wiley, New York, 1967):

Sand: 10^{-3} to 10^{-5} cm/s (1000 to 10 ft/year)

Silty Clay: 10^{-5} to 10^{-7} cm/s (10 to 0.1 ft/year)

Clay: less than 10^{-7} cm/s (less than 0.1 ft/year)

In a more specific vein, Dr. Malcomb Reeves reported permeability values for London clay of 1 cm/day (2.78×10^{-4} cm/s or 278 ft/year); refer to Sec. 6A.6. In the case of expansive soils, the horizontal permeabilities K_h often exceed the indicated values K_v by a factor of 10 or more. This is because of the presence of fissures, roots, induced fractures, bedding planes, etc.

In addition to the problems of permeability, infiltration has an inverse time lag function. Figure 1A.3 is a typical graphical representation of the relationship between infiltration and runoff with respect to time. At onset of rain, more water infiltrates, but over time, most of the water runs off and little is added to the infiltration.

Clays have a greater tendency for runoff, as opposed to infiltration, than sands. The degree of the slope of the land has a comparable effect, since steeper terrains deter infiltration. Only the water that penetrates the soil is of particular concern with respect to foundation stability. The water that fails to penetrate the soil is briefly discussed in Section 1A.5.

1A.5 RUNOFF

Any soil at a level above the capillary fringe tends to lose moisture through the various forces of gravity, transpiration, and evaporation. Given sufficient lack of recharge water, the soil water belt

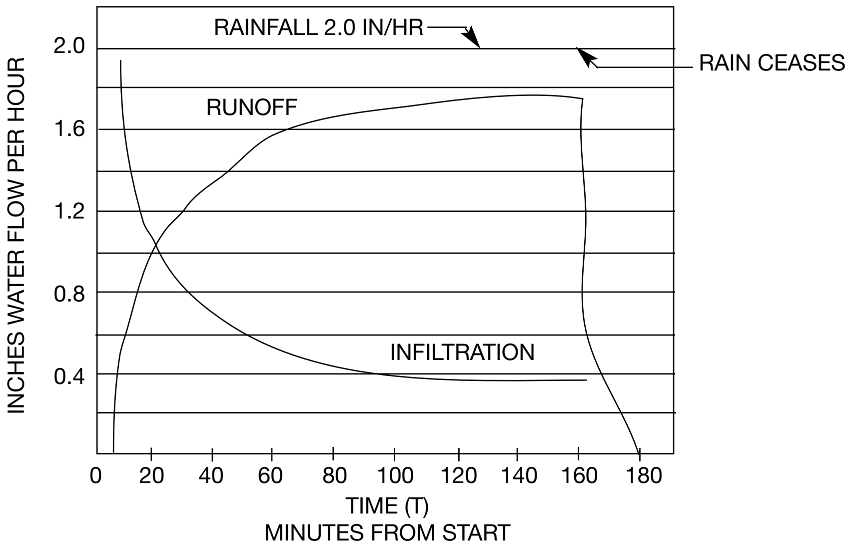

FIGURE 1A.3 Typical case of infiltration versus runoff after a 2 in/h rainfall.

will merge with and become identical in character to the intermediate belt. However, nature provides a method for replenishing the soil water through periodic rainfall. Given exposure to rain, all soils absorb water to some varying degree, dependent upon such factors as residual moisture content, soil composition and gradation, and time of exposure. The excess water not retained by the soil is termed *runoff* (Fig. 1A.3).

As would be expected, sands have a high absorption rate and clays have a relatively low absorption rate. A rainfall of several inches over a period of a few hours might saturate the soil water belt of sands, but penetrate no more than 6 in in a well-graded, high-plasticity soil. A slow, soaking rain would materially increase penetration in either case. The same comparison holds whether the source of water is rain or watering. Parts 7 and 9 also develop the importance of maintaining soil moisture to aid in preventing or arresting foundation failures.

1A.6 GROUNDWATER RECHARGE

Even in arid areas, an overabundance of water can occur sporadically due, principally, to storm runoff. If these surpluses can be collected and stored, a renewable resource is developed that involves conservation during periods of plenty for future use during times of shortage. Generally, this storage can be in the form of surface reservoirs or recharged aquifers [5].

Surface reservoirs suffer losses from evaporation, as well as occasional flooding, and are somewhat limited because of topographical demands.

Underground storage can be realized through natural groundwater recharge or artificial recharge. The obvious advantage to either form of underground storage is high capacity, simplicity, no evaporation losses, and low costs. Natural groundwater recharge occurs when aquifers are unconfined, surface soils are permeable, and vadose (aeration) zones have no layers that would restrict downward flow. When and where the foregoing conditions do not exist, artificial recharge is necessary. The latter requires that a well be drilled into the aquifer. Such wells can be used to inject water into or remove water from the aquifer, or both, depending on supply and demand. The prime storage zones include limestone, sand, gravel, clayey sand, sandstone, and glacial drift aquifers. The quality of the aquifers and recharge water depends mostly upon availability. Under the most adverse conditions, appropriate thought, well design, and operation procedures can produce potable water. Additional detail on this topic can be found in Ref. 5.

1A.7 CLAY SOIL

Preceding sections have suggested the influence of groundwater hydrology on foundation stability. This is most certainly true when the foundation-bearing soil contains an expansive clay. One complex and misunderstood aspect is the effect roots have on soil moisture. Without question, transpiration removes moisture from the soil. Exactly how much, what type, and from where represent the basic questions. If the roots take only pore (or capillary) water and/or remove the moisture from depths deeper than about 3 to 7 ft (1 to 2 m), the moisture loss is not likely to result in shrinkage of the soils sufficient to threaten foundation stability.

1A.8 SOIL MOISTURE VERSUS ROOT DEVELOPMENT

Logically, in semiarid climates, the root pattern would tend to develop toward deeper depths. In wetter areas, the root systems would be closer to the surface. In that instance, the availability of moisture would be such that the roots' needs could be supplied without desiccation of the soil; see Figs.

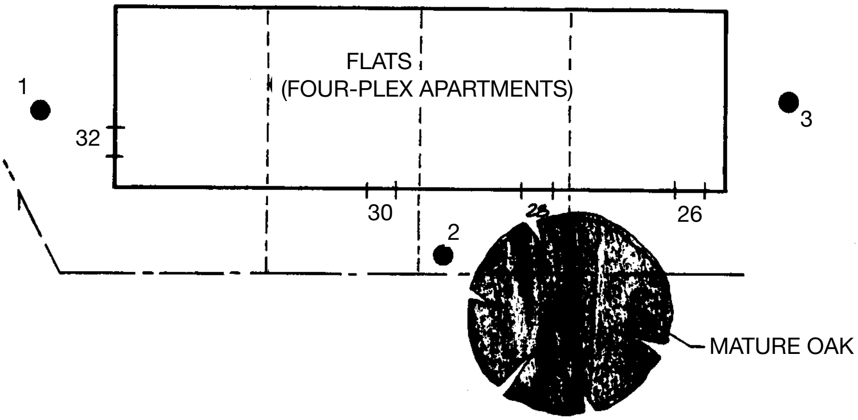

FIGURE 1A.4 Location plan.

1A.4 and 1A.5 and Table 1A.1. [An explanation of the Atterberg limits (LL, PL, and PI) is given in Sec. 2A.]

The soil in question is identified as a London clay with physical and chemical characteristics similar to many of the typical fat clays found in the United States. The London climate has a C_W factor* in range of 35 to 40, which is similar to that for Mississippi and Washington. Note that the soil moisture content remains constant from 2 to 5 m (6.6 to 16.4 ft) despite the close proximity of the mature oak tree (Table 1.1). Although this observation might be surprising, it is by no means an isolated instance. The test borings provided data on the loss of soil moisture, but there was nothing to indicate the root pattern. This information is not critical but would have been interesting. Note, however, that all tests commenced below the 2 ft (0.6 m) level, which seems to be the maximum depth from which roots remove moisture in this environment. (Refer to Sec. 6A.6, "Clay Mineralogy," and Sec. 7B.5, "Expansive Soils," for additional information concerning water behavior in clay soils.) In areas with more extreme climates and the same general soil, the root development pattern would more closely resemble that in Fig. 1A.6. It is worth mentioning that, during earlier growth stages, particularly if the tree is being conscientiously watered, the root system might be quite shallow—within the top 1 ft (30 cm) or so. Dry weather (lack of "surface" moisture) forces the roots to seek deeper soils for adequate water. The surface roots can remain dormant in a low-moisture environment for extended periods of time and become active again when soil moisture is restored.

Although the so-called *fat clays* are generally impermeable, thus limiting true capillary transfer of water, intrinsic fractures and fissures allow the tree or plant root system to pull water from soil a radial distance away somewhat in excess of the normal foliage radius. A side point worthy of mention is that when transpiration is active, evaporation diminishes (the shaded areas lose less moisture). The net result is often a conservation of soil moisture. The depth within which seasonal soil moisture varies is often referred to as the *soil active zone*. The total soil moisture change involves both evaporation and transpiration.

With respect to Fig. 1A.6, Dr. Don Smith, Botanist at The University of North Texas, Denton, suggests certain generalities:

1. D_1 is in the range of 2 ft (0.6 m) maximum.
2. W_r is in the range of 1.25XW, where W is the natural canopy diameter (unpruned).

*C_W is the climatic factor developed by the Building Research Advisory Bulletin (BRAB). It is used in the design of slab-on-ground foundations.

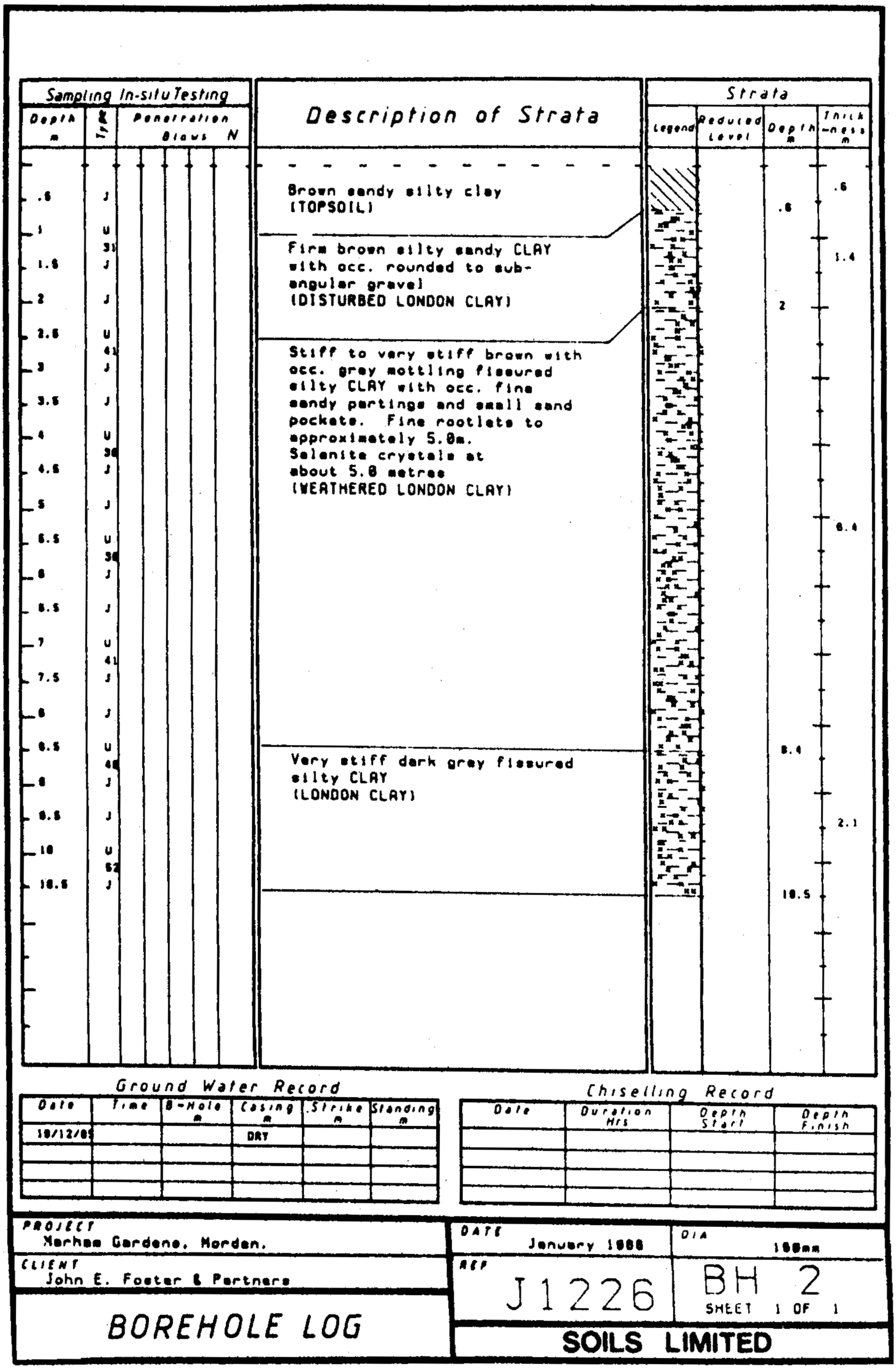

FIGURE 1A.5 Borehole log.

TABLE 1A.1 Atterberg Limits and Soil Moisture for London Clay BH No. 2: Brown-Gray Mottled Silty Clay

Depth						
m	ft	LL, %*	PL, %*	PI, %*	*W*, %*	Soil classification
2.0	6.6	93	27	66	30	CE
3.5	11.5	86	28	59	30	CV
4.5	14.8	89	28	61	30	CV
5.0	16.4	85	26	59	29	CV

*LL = liquid limit; PL = plastic limit; PI = plasticity index; *W* = natural moisture. The British Soil Classification uses CV for soils with an LL between 70 and 90 and CE for soils with an LL in excess of 90.

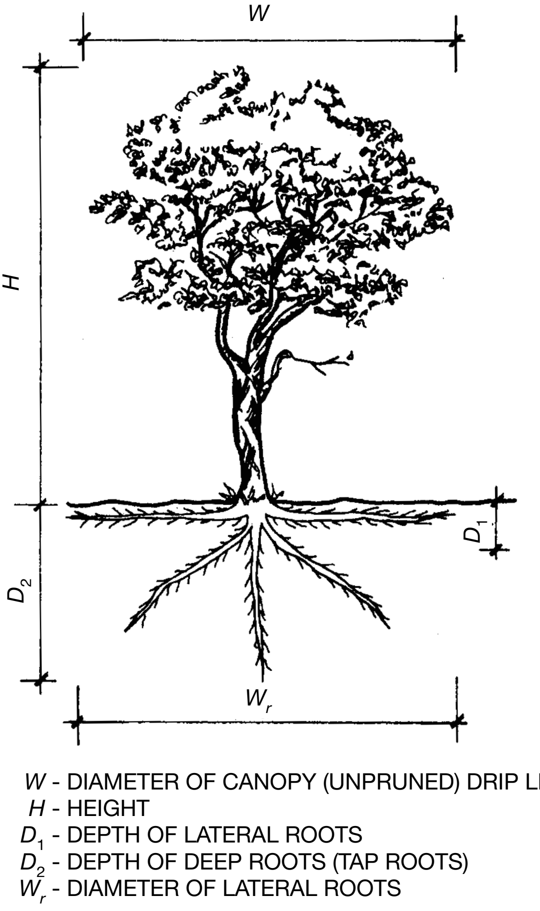

FIGURE 1A.6 Root system.

3. When moisture is not readily available at D_1, the deeper roots D_2 increase activity to keep the tree's needs satisfied. If this is not possible, the tree wilts.
4. H has no direct correlation to W_R, D_1, or D_2 except the indirect relation that H is relative to the age of the tree.

T. T. Koslowshi [6] and the National House-Building Council [7] suggest values for D_2, and the effective D_1, as shown in Table 1A.2. Note that the depth of soil moisture loss due to the near surface feeder roots is not to be confused with depth of total soil moisture loss (activity zone). The important point is that soil moisture losses from either transpiration or evaporation normally occur from relatively shallow depths. Both Tucker and Davis [2] and Tucker and Poor [8] report test results that indicate that 84% of total soil moisture loss occurs within the top 3 to 4 ft (1 to 1.25 m) (Fig. 1A.7). The soil involved was the Eagle Ford (Arlington, Texas) with a PI in the range of 42. Other scientists, such as Holland and Lawrence [9], report similar findings. The last publication suggests soil moisture equilibrium below about 4 ft (1.25 m) from test data involving several different clay soils in Australia with PIs ranging from about 30 to 60.

It might be interesting to note that the data accumulated by Tucker, Davis, and others [2,8,10] seem to indicate both minimal losses (if any) in soil moisture beneath the foundation and shallow

TABLE 1A.2a Depth of Tree Roots, Plains Area, United States*

Name	Age, years	D_2, ft (m)
Plantanus occidentalis (American sycamore)	6	7 (2.1)
Juglans nigra (black walnut)	6	5 (1.5)
Quercus rubra (red oak)	6	5 (1.5)
Carya ovata (shag bark hickory)	6	5 (1.5)
Fraxinus americana (ash)	6	5 (1.5)
Populus deltoides (poplar or cottonwood)	6	6 (1.8)
Robinia pseudoacacia (black locust)	Unknown	24–27 (7.3–8.2)

*After Ted Koslowski [6].

TABLE 1A.2b Depth of Tree Roots, London, England (PI above 40)*

Name	Age	D_1 m (ft)†	H (height), m (ft)
High water demand			
Elm	Mature	3.25 (10.6)	18–24 (59–79)
Oak	Mature	3.25 (10.6)	16–24 (52–79)
Willow	Mature	3.25 (10.6)	16–24 (52–79)
Moderate water demand			
Ash	Mature	2.2 (7.2)	23 (75)
Cedar	Mature	2.0 (6.6)	20 (65.6)
Pine	Mature	2.0 (6.6)	20 (65.6)
Plum	Mature	2.0 (6.6)	10 (32.8)
Sycamore	Mature	2.2 (7.2)	22 (72)
Low water demand			
Holly	Mature	1.55 (4.9)	12 (39.4)
Mulberry	Mature	1.45 (4.7)	9 (29.5)

*After National House-Building Council, United Kingdom [7].
†Interpolation of *maximum* depth of root influence on foundation design at $D = 2$ m, per Ref. 7.

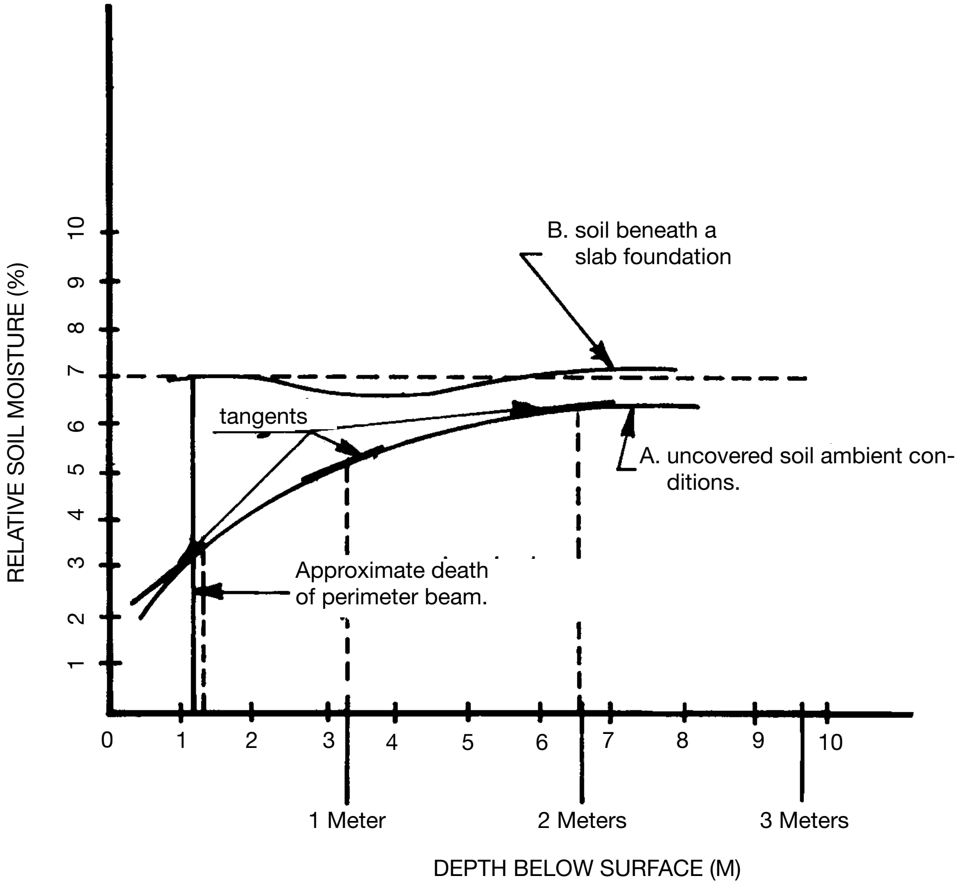

FIGURE 1A.7 Typical loss of soil moisture versus depth during a prolonged drying cycle. The tangent lines indicate the dramatic change in comparative soil moisture versus depth. (*From Davis and Tucker, Ref. 2.*)

losses outside the perimeter (Fig. 1A.7). Curve B presents moisture values taken from soil beneath the foundation. These data suggest slightly higher moisture levels than those plotted in curve A but also reflect a generally uniform buildup. The data in Fig. 1A.7 show that, while soil moisture varies to a depth of perhaps 7 ft (2.14 cm), over 85% of total soil moisture change occurred within the top 3 ft or so. Data published by McKeen and Johnson [12] reflect the same general conclusion. Their data reflect a relationship between the depth of the active zone, which varies with both suction (or capillary) pressure, and the number of cycles of wetting and drying that occur within the year. Nonetheless, between 80 and 90 percent of the total soil moisture variation occurred within the top 1.5 m (4.5 ft). Komornik presents data on an Israeli soil that show similar results [13]. The depth of moisture change extended to 11½ ft (3.5 m), but approximately 71% of the total change occurred within the top 3.2 ft (1 m). Sowa presented data that suggest an active depth of 0.3 to 1.0 m (1 to 3.2 ft) for a Canadian soil [14]. These observations, again, would seem to support the foregoing conclusions and opinions. A source for similar information can be found in “Building Near Trees” [7].

This document presents data compatible with those previously cited. Again the only question involves the issue of whether the tree height *H* is the important dimension describing root behavior or whether the canopy width *W* is the true concern, as apparently believed by most botanists.

Other authorities who agree with the statements concerning shallow feeder roots are John Haller [15], Neil Sperry [16], and Gerald Hall [17]. Haller states that the majority of feeder roots are found within 1 to 1½ ft (30 to 45 cm) of the surface. He explains that ". . . it is here that the soil is the richest and aeration the simplest." Both air and nutrition (water) are required by the healthy tree. Sperry and Hall concur. Deeper root systems are present but their primary function is to provide stability to the tree. In fact, the tap roots have the principal relationship to the tree height. This correlation is exploited by Bonsai growers who dwarf trees by shortening the tap root.

Many geotechnical engineers do not seem to share these views expressed by botanists. Dr. Poor seems to feel that the radial extent of a tree's root pattern is greater (H to $1.5H$) and the depth of moisture loss to transpiration is deeper [8]. Part of the apparent basis for his beliefs are presented in Fig. 1A.7 and in Sec. 1A.8.1 as item 11. These data as interpreted by the author seem to provide a limit on root radius of $0.5W$ (canopy width) and transpiration effective depth due to shallow feeder roots of less than 2.0 ft (61 cm) [11]. These values are of primary concern to foundation stability.

The overall maximum depth of effective soil moisture loss (active zone) appears to be in the range of about 1 to 4 m (3.2 to 12.8 ft), depending on the proximity of trees and geographic location [8,9,12–14,18]. Transpiration losses at depths below 2 m (6.6 ft), may not materially influence foundation stability [18]. These conclusions are also supported by the author's experience from 1963 to the present. The root systems for plants and shrubs would be similar to that shown in Fig. 1A.6, except on a much smaller scale. The interaction of tree root behavior and foundation failure is considered in following sections, especially 7A, 7B, 7C, and 9A.

1A.8.1 Summary: Soil moisture behavior

1. Roots per se provide a benefit to soil (and foundation) stability since their presence increases the soil's resistance to shear [19,23]. Also, the plant canopy (shade) reduces evaporation and, overall, may conserve soil moisture.

2. Tree roots tend to remove soil moisture; hence the net result, if any, is foundation settlement. Settlement is normally slow in developing, limited in overall scope, and can be arrested (or reversed) by a comprehensive maintenance program. (Refer to Sec. 7A.) Chen [20] states, "The end result of shrinkage around or beneath a covered area seldom causes structural damage and therefore is not an important item to be considered by soil engineers." Other noted authors might disagree, at least to some extent. Mike Crilly, of the Building Research Establishment, London (and others within that organization [22]) presents data shown in Fig. 1A.8 [21]. These data were collected by using rods embedded in the ground. Group 1 data, away from trees, suggest negligible soil movement at depths below the surface. (The surface loss was likely due to grass and evaporation. Refer also to item 9, below.) Group 2 data show vertical movement potential at the surface of 100 mm (4 in) and about 60 mm (2.4 in) at 1 m (3.3 ft), but below 2 m (6.6 ft) the movement is on the order of less than 15 mm (0.6 in). The data bring to mind two questions: (1) what would the moisture (and vertical movement) profiles look like if the data were taken from foundation slabs designed with perimeter beams and (2) would the conventional foundation design preclude damage? Others have suggested that surface soil movement can be related to the movement of slab foundations, although it is not always clear how the correlation might be made [2,8,22]. For example, would tests using 1 m^2 (10.89 ft^2) pads poured on the ground surface relate to tests using larger pads, i.e., 400 m^2 (4356 ft^2), or conventional foundations?

3. While some degree of settlement is noted in most light foundations on expansive soils, that specific problem by itself is seldom sufficiently serious to demand repair. In fact, according to a random sampling of over 25,000 repairs performed (principally within the Dallas–Fort Worth area) over a period in excess of 30 years, the incidence of settlement versus upheaval (as the preponderant

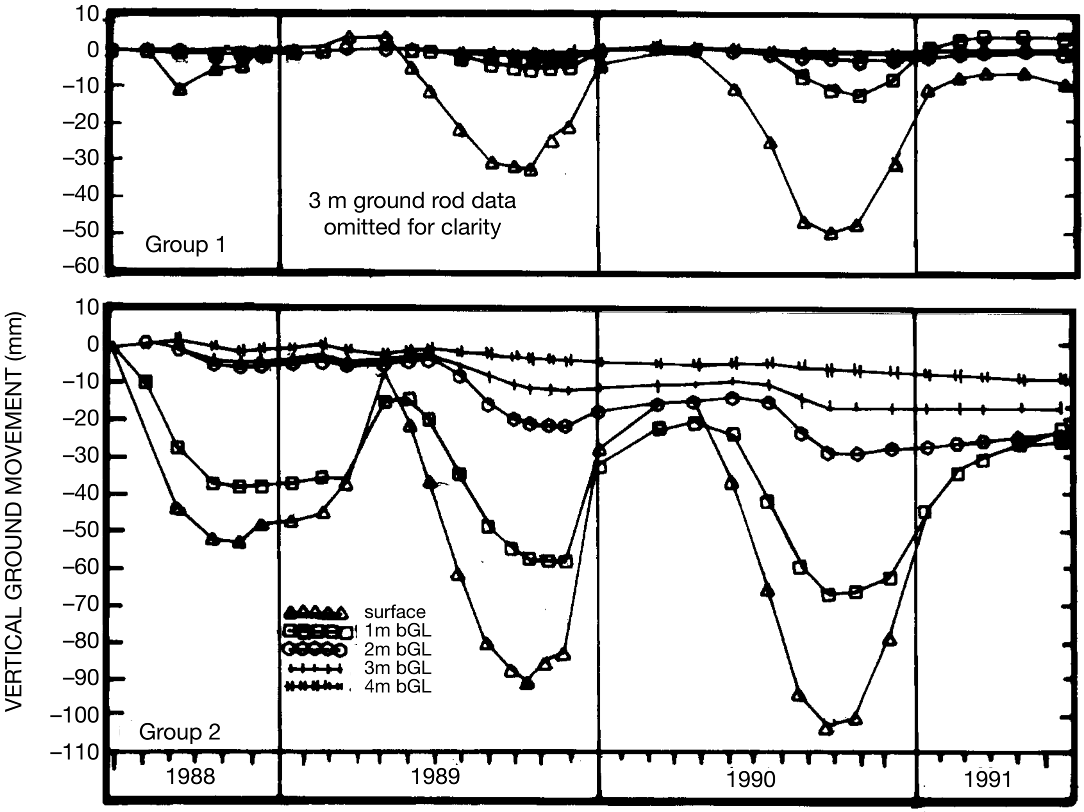

FIGURE 1A.8 Results obtained from ground movement rods: remote from trees (Group 1); and near trees (Group 2).

cause for repair) was about 1.0 to 2.3 (30 to 70%). [Three out of four foundations repaired were of slab construction (as opposed to pier-and-beam) and over 94% of the foundations were of steel-reinforced concrete construction.] Most of the repairs catalogued as "settlement" involved instances of: (1) shimming of interior pier caps (pier-and-beam foundation), (2) underpinning (raising) slab foundation wherein proper mudjacking was not included in the initial repairs and subsequent mudjacking of the interior slab was required, or (3) foundations constructed on uncompacted fill. Delete these from the settlement statistics and the incidence of settlement repairs is reduced to something like 3%.

4. Texas' shallow soils generally exist at moisture levels between the SL and PL with, as a rule, the moisture contents somewhat closer to PL.* In deeper soils, the W% is something higher, between the PL and LL. (For comparative purposes, the C_W rating ≈ 20.)

5. All soil *shrinkage* ceases when W% approaches the SL (by definition) and does not commence until the moisture content is decreased below the LL. *Soil swell* in expansive soils effectively ceases at W% content above or near the PL. (Refer to Chap. 6.) Thus, moisture changes at levels much below the LL or much above the SL do not affect expansive soil volume (or foundation movement) to any appreciable extent.

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*The Atterberg limits (LL, PI, PL, SL, W%) are discussed in detail in Sec. 2A.

6. Expansive soil particles tend to shrink at moisture *reductions* between something below the LL and the SL. Refer to Fig. 1A.8 [23]. Those existing at a *W*% between the SL and PL tend to swell upon access to water. Refer to Figs. 7B.2 and 1A.8. [*Nonexpansive* (or noncohesive) soils are prone to shrink when water is removed from them at or near saturation (or LL). Particle consolidation largely accounts for this volumetric decrease rather than particle shrinkage.]

7. The data depicted in Fig. 1A.9 (McKenn, Ref. 24) suggest a basic relationship between soil volume change and *W*% expressed as pF [pF is the logarithm to base 10 of the pressure in centimeters of water (1 pF = 1 kPa, 2 pF = 10 kPa, 3 pF = 100 kPa, etc.)]. The range of volume change versus pF decreases between the field capacity (2.2 pF) and shrinkage limit (5.5 pF). For more practical concerns, a plant's removal of water (transpiration) is probably limited even further, to that level between field capacity (2.2 pF) and the point of wilt (4.2 to 4.5 pF). Note that the field capacity represents a *W*% less than the LL and the point of plant wilt is well above the SL. Similar conclusions have been published by F. H. Chen [20].

Evapotranspiration, on the other hand, would transcend a wider scope. The combined effect of soil moisture withdrawal could reflect soil volume changes between the field capacity and SL—a wider range than that likely for transpiration alone.

A soil can gain or lose moisture, within specific limits, without a corresponding change in volume [20,23,24].

8. There is definitely a relationship between shrinkage and swell in an expansive soil. A soil that swells will shrink (and vice versa) upon changes in available moisture. However, assume a given specimen where an increase of 4% moisture produces a swell of *X*%. Will removal of 4% moisture cause the soil to shrink *X*%? Not likely [20].

Chen's report, outlining a series of tests using a Denver remolded clay shale, indicates that only at the point of critical dry density does shrinkage equal swell [20]. Figure 1.10 depicts test data showing the shrink and swell resulting from controlled initial moisture contents. In these tests, the dry density was kept reasonably constant (107.0 ± 0.6 lb/ft^3) and the initial moisture content was

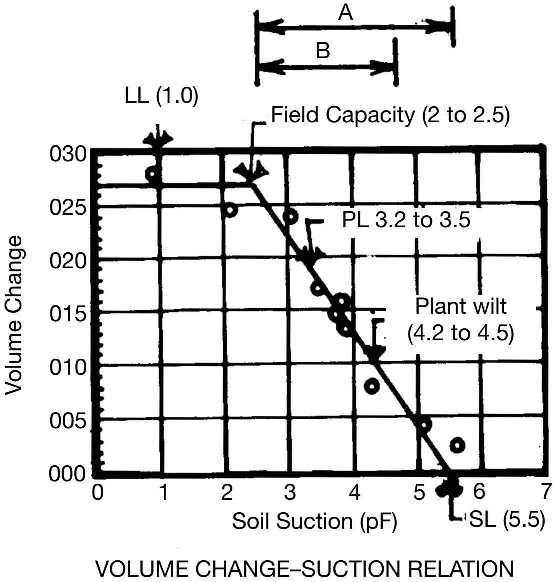

FIGURE 1A.9 Range of relative volume change. A: evaporation and transpiration; B: transpiration.

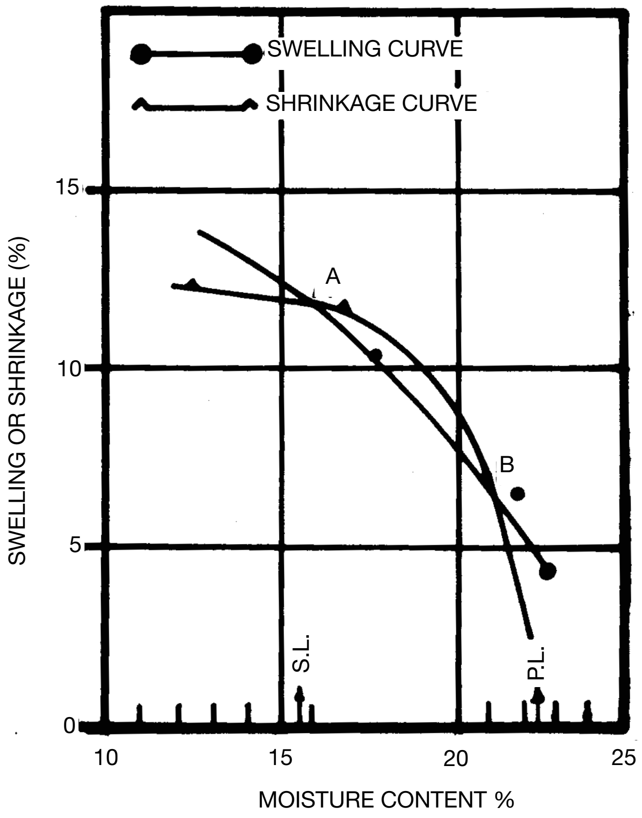

FIGURE 1A.10 Effects of moisture content on swelling and shrinkage.

varied from slightly below the shrinkage limit (15.5% versus 15.1%) to slightly below the plastic limit (22.4% versus 22.3%). The samples were placed under a surcharge pressure of 1 lb/in^2 (7 kPa) and allowed to swell in distilled water. After two or three days the specimens were removed from the water, weighed, and allowed to dry. Once air-dried to initial weight, each specimen was again weighed and the density and moisture content determined. From these data, the percent shrinkage or swell was determined.

As expected, the swell potential decreases as the initial moisture (in situ) increases, approaching zero as the moisture contents nears saturation. Also, shrinkage ceases both at the moisture content referred to as the shrinkage limit (SL) and at or near saturation. Shrinkage is equal to swell at points *A* and *B*. Between the points *A* and *B* shrinkage potential is greater than swelling. Outside this range, the reverse is true.

9. Heave of surface soils occurs mostly within rather confined limits, as noted above (SL to proximity PL). It would seem that removal of surface vegetation in a $C_W \approx 20$ climate would encourage soil desiccation as opposed to net *W*% gain (assuming reasonable drainage). If expansive soils are properly drained, it would seem likely that *W*% variations largely would occur at relatively shallow depths. In climates such as London's [30 in (76 cm) annual rain distributed over about 152 days)], the in situ *W*% in absence of transpiration (lack of evaporation) should, in fact, increase.

However, once again, this effect on soil *movement* begins to cease as the *W*% approaches or somewhat exceeds the PL. It would seem that *W*% in London, for example, would be consistently higher that in the United States. London's rainfall (though roughly equivalent to Dallas–Fort Worth's annual rainfall of 30 in) is distributed rather evenly over 152 days as opposed to the 15 days that account for 80% of the Dallas–Fort Worth precipitation. The considerably more moderate temperature ranges would combine with the extended rain to logically produce both higher and generally more stable *W*%. [The annual average temperature in the Dallas–Fort Worth area is about 65°F (18°C), whereas that for London is about 52°F (11°C). The relative temperature *ranges* are 15° to 105°F (–9° to 40°C) for Dallas–Fort Worth and 38° to 78°F (3° to 25°C) for London.]

10. Vegetation (transpiration) removes soil moisture mostly at very shallow depths [15–17]. The U.S. horticulture community invariably recommends that trees be watered and fed at or near the drip line (extend of canopy). Further, most agree that nutritional roots are classically quite shallow—within 12 to 24 in (30 to 60 cm). The reasons given include: (1) root development favors loosely compacted soil, (2) roots like oxygen, (3) roots like water, (4) roots like sunlight (to some extent), and (5) roots exert only that energy necessary for survival. Under particularly adverse conditions (such as a prolonged draught) feeder roots may develop at deeper depths. Still it is generally agreed that 90% of the tree's moisture needs are taken from 12 to 24 in (30 to 60 cm).

11. It has been well established by many research projects that foundation stability is not influenced by soil behavior below the soil active zone (SAZ). In Dallas, the preponderance (87%) of that influence on foundation stability is limited to about 3 ft (1 m), although the SAZ may extend to depths in excess of 7 ft (2.13 m). [8,11] Other geographical locations report different depths for the active zone. For example: (1) for a Canadian soil, Sowa [14] indicates the depth of the soil active zone to be 1 to 3 ft (0.3 to 1 m); (2) for an Israeli soil, Komornik [13] reports an active soil zone as deep as 11.5 ft (3.5 m) but approximately 71% of the total moisture variation occurrs within the top 3.2 ft (1 m); (3) Holland and Lawrence report data on an Australian soil where soil moisture equilibrium depth is less than 4 ft (1.25 m) [9].

12. Other factors of concern include such issues as: (1) overburden tends to suppress soil expansion; doubling the effective overburden pressure (1000 to 2000 lb/ft^2) can reduce swell by about one-third (F. H. Chen) [20]; (2) the surcharge load on the soil diminishes with depth (for strip footings the effect of load is in the range of only 10% at a depth of twice the width); and (3) low soil permeabilities severely inhibit soil moisture movement, particularly in a vertical direction [expansive (sedimentary) soils in general have much higher lateral than vertical permeability].

13. Without a doubt, the age and proximity of the tree (and the depth of the perimeter beam) are very important factors that affect the amount of water a tree might remove from the foundation-bearing soil. Certainly, younger trees tend to remove moisture at a faster and greater relative rate. Also, trees tend to require much more water during growth periods. Without the leaves or during dormancy, a tree might require as little as 1% of the growth amount of moisture. The influence of transpiration or foundation stability should thus be relative to season. It would seem wise in most cases not to plant new trees in close proximity to the foundation. Nonetheless, concrete evidence available to the author seems to suggest that the impact of vegetation on the stability of foundation is grossly overstated. Any *proof* to the contrary would be welcome.

14. Many engineers in the United States (and probably elsewhere as well) confuse center heave with perimeter settlement. Hence, the influence of trees is often overstated. (Refer to Sects. 7A. and 9A.) Sound evidence and not wishful thinking should be the final criterion for decision making. One source for reliable data offers a history of over 25,000 actual repairs performed over 30 years. Many of these repairs were performed on structures with trees (in some cases multiple trees) located in close proximity to the foundations, sometimes as close as 1 ft (0.3 m). There is no memory of the repair company suggesting or requiring the removal of any tree, bush, or other vegetation. Yet in absence of tree removal, none of the repairs experienced a subsequent failure that could be attributed to the presence of a tree, bush, or vegetation. (These data were collected primarily from the Dallas–Fort Worth area of Texas but data points included other states from Arizona to Illinois and Oklahoma to Florida.) Does this seem to dispute the deleterious influence of trees on foundation stability? If the trees played a predominate part in causing the initial foundation failure, why did not

the same or similar problem recur? Also *many* other foundations within the same areas have a tree (or trees) in close proximity to the foundation, yet never suffer foundation distress. It does not stand to reason that trees are capable of preferentially selecting one address over another.

15. Again with reference to the study mentioned above and item 3, most of the repair causes were attributed to upheaval brought about by the accumulation of water beneath the slab foundation. (Once the source for water was removed, the foundation stabilized.) There seems to be some confusion in terminology in addressing slab heave on expansive soils. An often misused term is *natural center doming*, which allegedly describes the buildup of soil moisture due to capillary and/or osmotic transfer. Proponents believe that this phenomenon occurs in most slab-on-grade foundations, with the net result being a central high or domed area. Research does not verify this conclusion [9,11]. Also, for greater detail, refer to Sections 7A, 7B, 7C, and 8. *Center lift* is another term used in the BRAB and Post Tension Institute (PTI) books (Refs. 25, 26). This is an important design concern that relates more to upheaval than to *center doming*. (Refer to Sec. 9A.)

1A.9 CONCLUSIONS

What factors have become obvious with respect to soil moisture as it influences foundation stability?

1. Soil moisture definitely affects foundation stability, particularly if the soil contains expansive clays.
2. The soil belt is the zone that affects or influences foundation behavior the most.
3. Constant moisture is beneficial to soil (foundation) stability.
4. The water table, in itself, has little, if any, influence on soil moisture or foundation behavior, especially where expansive soils are involved.
5. Vegetation can remove substantial moisture from soil. Roots tend to find moisture. In general, transpiration occurs from relatively shallow depths.
6. Introduction of excessive (differential) amounts of water under a covered area is cumulative and threatens stability of some soils. Sources for excessive water could be subsurface aquifers (e.g., temporary perched groundwater), surface water (poor drainage), and/or domestic water (leaks or improper watering). Slab foundations located on expansive soils are most susceptible to the latter. Refer to Sects. 7A, 7B, 7C, and 9A.
7. Assuming adequate drainage, proper watering (uniformly applied) is absolutely necessary to maintain consistent soil moisture during dry periods—both summer and winter.
8. The detrimental effects on foundations from transpiration appear to be grossly overstated.

The homeowner can do little to affect either the design of an existing foundation or the overall subsurface moisture profile. From a logistical standpoint, about the only control the owner has is to maintain moisture around the foundation perimeter by both watering and drainage control and to preclude the introduction of domestic water under the foundation. Adequate watering will help prevent or arrest settlement of foundations on expansive soils brought about by soil shrinkage resulting from the loss of moisture.

From a careful study of the behavior of water in the aeration zone, it appears that the most significant factor contributing to distress from expansive soils is excessive water beneath a protected surface (foundation), which causes the soil to swell (upheaval). From field data collected in a 30 year study (1964–1994), including some 25,000 repairs, it is an undeniable fact that a wide majority of these instances of soil swell were traceable to domestic water sources as opposed to drainage deficiencies. Further, the numerical comparison of failures due to upheaval versus settlement was estimated to be in the range of about 2 to 1. Refer to Sects. 7A, 7B, 7C, and 8 for more detailed information. Also bear in mind that the data described were accumulated from studies within a C_W rating (climatic rating) of about 20 (refer to Fig. 7.B.8.3). This describes an area with annual rainfall in the range of 30 in (75 cm) and mean temperatures of about 65°F (18°C).

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SECTION 1B
SITE PREPARATION

BARBARA COLLEY

1B.1 INTRODUCTION

The presence of water over and under a building site impacts the way foundations perform. Maintaining a consistent level of soil moisture is desirable. The best way to affect the consistency of the soil moisture is by limiting the incursion of unplanned water onto the building pads. For this reason, civil engineers and others design plans so that surface, and in some cases underground, water will flow away from building foundations.

1B.2 GRADING PLANS

To protect the building pad from surface water, each project must be sculpted and compacted to direct drainage away from buildings and other structures. The activities necessary to accomplish this are called *earthwork*. Before concrete can be poured and structures built, the land must be prepared to provide a strong base. A civil engineer specializing in soils should be assigned to determine the characteristics of the soil, evaluate the potential for groundwater impacts and recommend construction methods to be used to provide the base for the structures. If the site is in a mountainous area or an area subject to earthquakes, a geologist or geologic engineer should also be contracted to evaluate risks and make recommendations for protection against landslides and earthquakes.

1B.3 THE SOILS REPORT

An investigation of the soils should be made for every site and a report made. The investigation should be made by a qualified civil engineer specializing in soils science. The soils engineer will visit the site, take soils samples, and make borings at various locations. The cores resulting from the borings show the underlying strata. A three-dimensional view of the layers of earth and rock can be projected from the cores. Although subsurface conditions cannot be described with absolute certainty, the unknowns are reduced and much useful information is provided.

The different types of soil and rock on the site are identified. A series of tests are performed on the soils to determine their strength, plasticity, potential for liquefaction, and permeability (See Section 2A). The depth of groundwater is also provided. The level of groundwater varies with the time of year and the character of the previous rainy seasons. If the seasons have not been typical or there is historical evidence that groundwater is a problem, further investigation is indicated.

The information provided will be useful to the architect and structural engineer in designing the structures, to the site engineer in designing paved surfaces and slopes, and to the contractor charged with grading the site. If subsurface conditions change abruptly under a proposed structure location, it may be necessary to excavate existing earth to provide a consistent earth foundation beneath that structure, or to design different foundations for different parts of the structure.

The report should describe maximum allowable slopes. The allowable slope is based on the *angle of repose* for the soil on the site. The angle of repose is the angle between horizontal and the slope of a heaped pile of the material. Using a steeper slope could result in slope failure or landslide. The *slope* is described as the unit horizontal distance necessary for each unit of vertical distance (Fig. 1B.1). The slope described as 2:1 indicates two horizontal units to for every vertical unit. (The same slope is defined as 1:2 vertical to horizontal in the metric system). These slopes will be used between areas or pads of different elevations.

The relative compaction requirement should be included in the soils report and is important to the site engineer. Typically, the engineered base for structures in the field must have 90 to 95% relative compaction. That is, the soil must be compacted to 90 to 95% of the maximum dry unit weight from laboratory tests. Compaction testing methods are described later in this book.

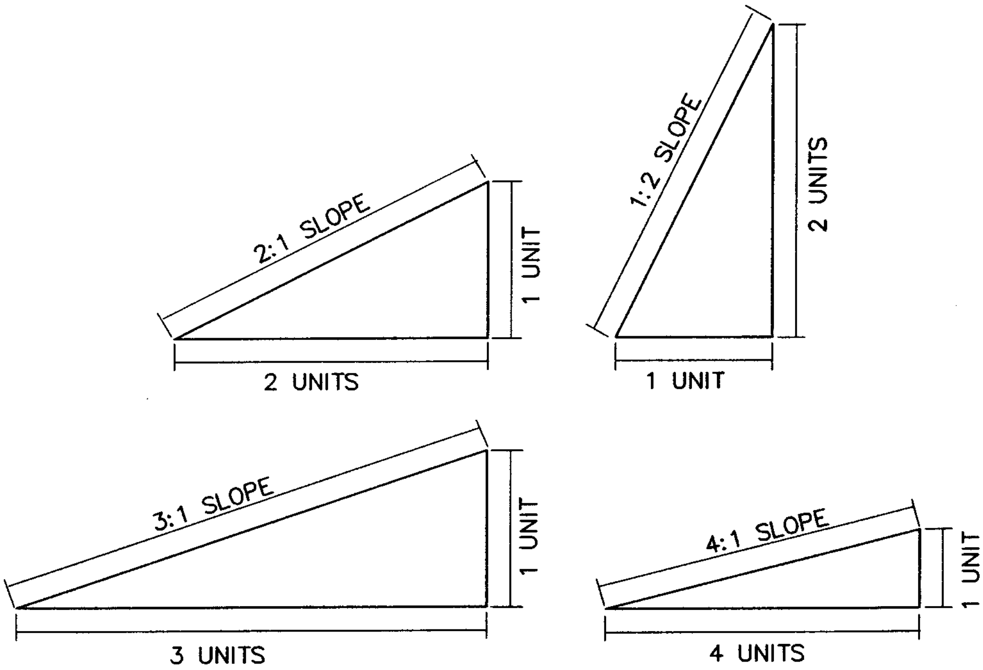


FIGURE 1B.1 Slopes are described by the number of horizontal units for each vertical unit.

TABLE 1B.1 Earthwork Calculation

	Cut (yd³ or m³)	Fill (yd³ or m³)
Pads and parking	4780	4080
Compaction	0	700
Organic material	320	
Stockpile for landscaping		320
TOTAL	5100	5100

The natural earth in place may not be sufficiently compacted, in which case more earth will be required to fill the same space after compaction. A clear demonstration of this can be seen by filling a cup loosely with sand and clearing off the excess sand level with the top of the cup. If you then tap the cup several times, the sand will compact, and the cup will no longer be full. The same is true for earthwork.

All sites require some excavation and some embankment to provide level pads. If the earthwork is measured in cubic yards for design and estimation purposes, more than a cubic yard of excavation will be required for each cubic yard to be filled. The percentage difference, expressed as a portion of 1, is called the *compaction* or *shrinkage factor*. The soils report should give a shrinkage factor and may describe the optimum moisture needed and construction methods and equipment to be used to accomplish the recommended compaction. The relationship used to determine the amount of earth needed to compensate for shrinkage is shown here.

$$V_R = \frac{V}{100} - S \tag{1B.1}$$

where V_R = volume of compacted earth (fill) required, yd³
V = volume of uncompacted earth (excavation), yd³
S = shrinkage factor

Not all soil found on a site will be suitable for construction of the building pad. Humus soil must be removed before construction is begun. The soils report should describe the depth of the unsuitable soil and whether it can be stockpiled and later used for landscaping and on nonstructural areas of the site.

It is desirable to have the grading plan designed so that excavation and fill on a site will *balance*. The earthwork on a site is said to balance when no import or export of material is required to create the building pad. To accomplish a balance, a volume of earth to allow for shrinkage must be included in the calculations (see Table 1B.1). Where the native soils have poor structural qualities or are expansive, the soils report may recommend importation of soils better suited to providing a subbase for structures.

1B.4 THE GEOLOGIC REPORT

Peoples lives and property can be destroyed very quickly by landslides and earthquakes; therefore, hillside areas of existing or potential landslides should be identified. Once a previous or potential landslide area is identified, recommendations can be made to avoid the risky areas. In some cases, areas of potential landslides or of soil creep can be used if certain precautions are taken or the structures are designed to accommodate the problems.

Earthquakes can be a threat to life and can damage or destroy structures. There are two primary ways that earthquakes cause damage. One is through the lateral forces created when the earth moves. This is a structural engineering problem. The other cause is that some soils liquefy during the ground shaking. These soils and their depths must be identified so that foundations can be designed to withstand liquefaction.

The geologist will research the geologic history of the site, study aerial photographs, perform soundings to determine subsurface densities, and dig trenches across suspected earthquake faults and ancient landslides. Earth cores will also be extracted and studied. With this information, recommendations can be made as to areas where structures are at risk and possible mitigation methods must be taken.

The geologic report should also identify groundwater conditions. If the water table is near the surface, it can create problems for structures. The geologist can make recommendations as to the scope of the problem and make suggestions for removing the water so that it will not adversely affect the structures.

1B.5 HILLSIDE SITES

On hillside sites, earthwork is usually significant. Earth is excavated from one area of the site and placed on another in order to create a level pad or pads for the foundations. Where there will be high cut or fill slopes, benches are usually required in the slope. The benches will stop falling rocks and earth and will be used to intercept and redirect overland drainage. Benches are also required in existing sloped ground that will be covered by an embankment (Fig. 1B.2).

The natural slope is first scraped clean of any organic material, then cut into benches. The vertical distances between benches and the width of the benches will be determined by the characteristics of the soil, widths needed to operate equipment, and what the finished slope will be. Benches so employed in fill slopes are usually sloped at 1% into the hillside and have a key in the bottom bench to connect the soil masses.

1B.6 EXISTING TOPOGRAPHY

Of prime importance in understanding the various elements of the grading plans as well as the other aspects of design is the concept of elevations. When the term *elevation* is used, it may refer to an ac-

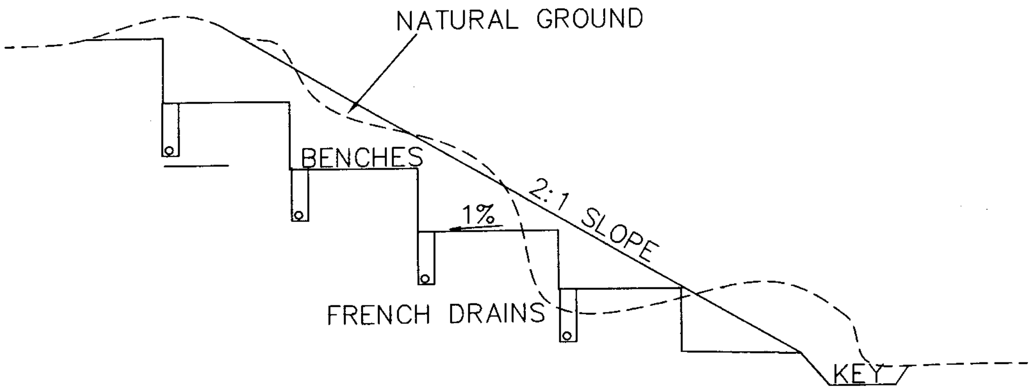


FIGURE 1B.2 Benches.

tual elevation (vertical distance in feet or meters above mean sea level), or it may refer to a vertical distance above an assumed elevation. Although the dimension of the elevation is in feet or meters, it is customary to show elevations without a dimension.

All plans using elevations should have a *benchmark* (BM). The benchmark is a vertical reference point. The benchmark may be a brass disk set in concrete by the U.S. Geological Survey (USGS) or some other agency, and tied to mean sea level, but it can be anything that has a permanent elevation that can be referenced. Some jurisdictions require that all plans be referenced to their standard benchmarks or USGS benchmarks. At this writing, USGS maps and benchmarks are in English units (feet), except for some of the 1:100,000 maps produced in 1991 and 1992. Whether the elevations are in feet or meters will be clear from information provided on the map.

On projects where there is no existing benchmark in the vicinity, the surveyor may establish a benchmark using some permanent feature such as a top of curb or manhole cover and give it an arbitrary elevation high enough so that no point related to the project will have a negative elevation. This point then has an assumed elevation and elevations are given to elements needed to design and build the plan in reference to that benchmark. What is important is that all the vertical relationships among the design elements is established. There are areas where the land is below sea level and will have negative elevations, but when an assumed elevation is to be used for the benchmark, negative elevations should be avoided.

Care should be taken when using elevations from existing plans. The benchmarks used to design different projects are often taken from different sources, so the relation between elevations on the projects will not be true. The elevation for a physical object taken from one benchmark may be different from an elevation for the same object taken from another benchmark, unless the two benchmarks refer to a common benchmark. Even then, there may be some differences due to the degree of precision or errors. Where two or more sets of existing plans are to be tied together, it may be necessary to establish a *benchmark equation*. An example is

Rim elevation for sanitary manhole on Main Street at Spring Street 139.68 from Tract 5555
= 140.03 from Tract 5560

In this case, if elevations for Tract 5560 are to be used on the new project, but ties must be made to objects in Tract 5555, 0.35 (140.03 – 139.68) must be added to all elevations taken from Tract 5555.

Before design is begun on the grading plan, elevations should be shown wherever they must be considered in the design. This includes elevations for existing and proposed:

1. Natural ground
2. Ditch flow lines within project boundaries and outside a sufficient distance to show the limits of the *drainage basin* (described later in this section) contributing drainage flows to the project
3. Tops of curbs at
   a. Property lines
   b. Beginnings and ends of horizontal curves
   c. Beginnings, ends, and high or low points in vertical curves
   d. High and low points in street center line profiles
   e. Points beyond the property line as necessary to show the grade of the street so that smooth transitions can be made.
4. Existing streets being met at connections and as necessary to show the grade of the street so that smooth transitions can be made
5. The bases of trees and other amenities to remain

In most cases, the topographic map will have been produced through the use of photogrammetry, and most of this information will be available on the map. The engineer must determine how far beyond the limits of the project topography is required before ordering the topographic map.

Lines connecting points of equal elevation are called *contours* (Fig. 1B.3). They are usually plotted for even elevations of 1, 2, or 5 feet (0.3, 0.6, or 1.5 m). Where the terrain is very flat, the one foot contour interval is used and intermediate elevations are spotted where the slope between contours is not uniform. In steep terrain, the contour interval may be 5 feet (1.5 m), 10 feet (3 m), or

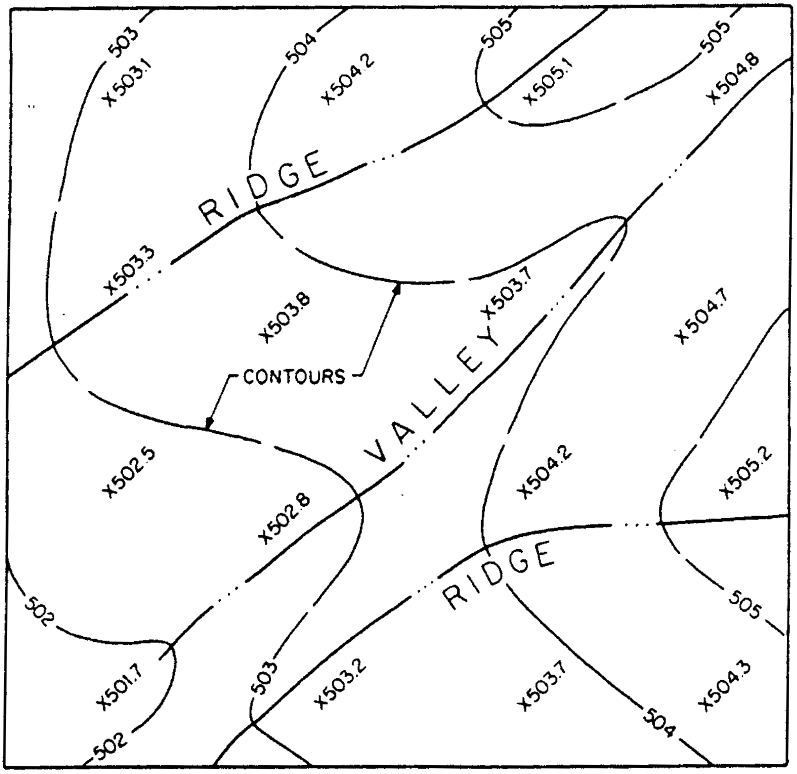


FIGURE 1B.3 Contours.

even greater. The steeper the slope, the closer the contours will be. Therefore, rather than fill the map with contour lines, a greater interval is used.

The surveyor or photogrammetrist should have marked an elevation wherever there is a break in the slope. Therefore, it should be safe to assume that the ground between elevations slopes evenly. Though contours are used primarily to illustrate existing topographic conditions, *contour grading* can be used to show proposed finished contours. During preliminary stages of design, the contours as they will exist when the construction is complete can be drawn as a graphic illustration of the concept. Exact contours can be drawn during the design phase to be used for earthwork calculations and to show drainage patterns.

Cross-sections are used extensively in designing grading plans. Figure 1B.4 shows an example. Elevations on the natural ground are plotted to scale in a line perpendicular to, and measured distances from, some reference line. When the points are connected, they represent the cross-section of the natural ground. Then elevations at break points in the finished plan are plotted along the same line. The elevation at the edge of the finished lot usually does not meet the existing ground but is above or below it. This point is called the *hinge point*. From this point, a slope is designed based on the slope recommended in the soils report. The slope will probably be between 1:1 and 4:1. That slope will be extended until it connects to the natural ground. That point is called the *catch point*.

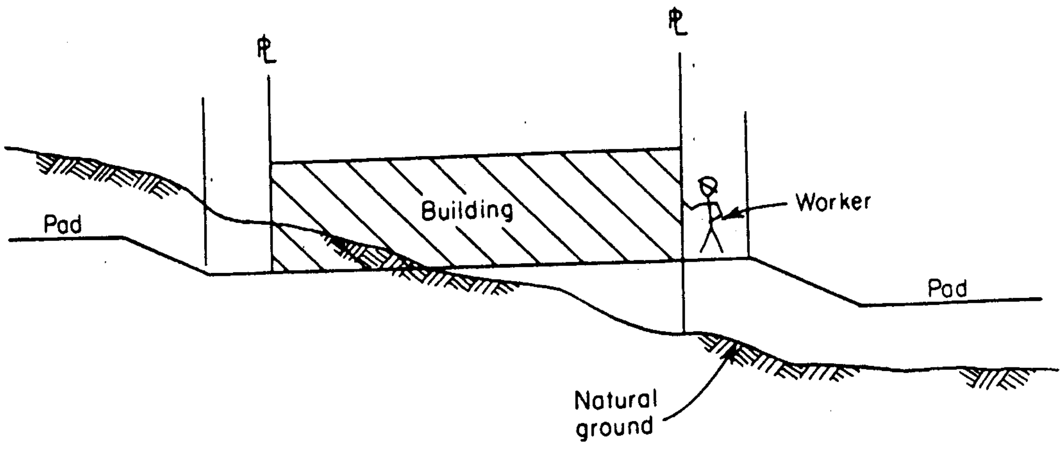


FIGURE 1B.4 Cross-section.

1B.7 DETERMINING THE BUILDING PAD

The grading plan must be designed with an understanding of the drainage criteria. The storm drainage and overall design are coordinated with the grading plan. On hilly or complicated sites, the first step may be a preliminary contour grading plan. Usually, street profiles are existing or have been designed and proposed top-of-curb elevations or edge of pavement elevations calculated and transferred to the grading plan. This information is essential for designing the site grading.

There are three types of residential lot grading plans (Fig.1B.5):

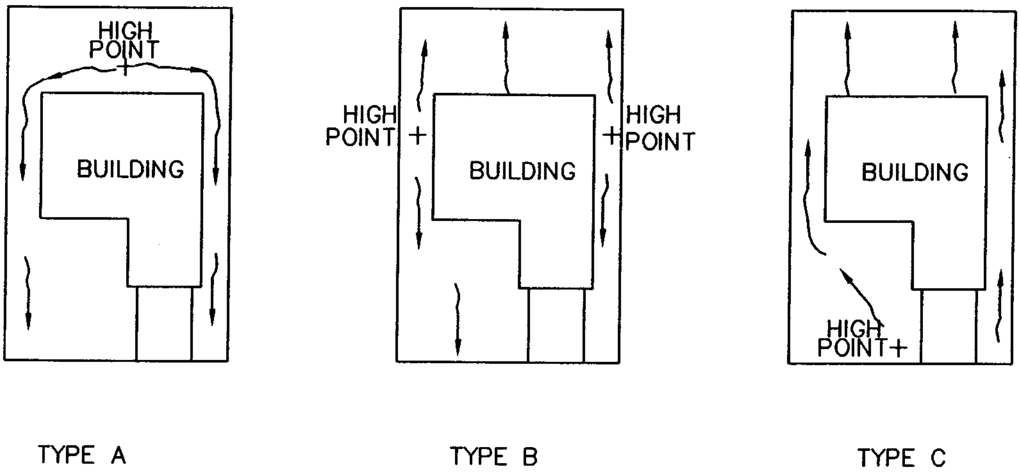


FIGURE 1B.5 Types of drainage.

Type A—All the overland drainage on the lot is directed to the street at the front of the lot.

Type B—Drainage on the front half of the lot is directed to the street in front, and drainage on the back of the lot is directed to a street, alley, or ditch in the back of the lot.

Type C—All drainage is directed to the back of the lot.

Some jurisdictions allow only Type A drainage. Where type B or C is allowed, a ditch or other drainage facility must be designed for the back of the lot. Storm drainage easements must then be acquired to take the drainage across adjacent properties. All lots crossed with a ditch or underground system for storm drainage must be provided with a private storm drainage easement. On hillside sites where much of the site will be left natural, a ditch may be required at or near the property line to prevent storm water that falls on one property from crossing adjacent property.

On residential and simple commercial/industrial sites, the elevations of the pads should be selected so that they will drain to the front of the property. This will save the complications of draining storm water over adjacent properties or the cost of installing storm water inlets.

1B.7.1 Building Pads with No Storm Water Inlets

The criteria for selecting the building pad elevations where there will be no drainage inlets within the lot are:

1. The pad must be high enough above the lowest top-of-curb elevation at the front of the property to accommodate a drainage swale around the building with a slope of at least 1%. Often, the size of the lot and slope in the street are consistent, so a constant amount can be added to the lower top of the curb to establish pad elevations.
2. The pad must be designed so the grade on the driveway does not exceed 15% up or 10% down to the garage floor. Steeper grades may result in the undercarriage of cars scraping and damaging the car or the driveway. Flatter driveway slopes should be used wherever possible. A drainage swale must be provided in the driveway in front of the garage where the garage is below the street.

   When the building setback distance and driveway length are consistent in a subdivision, a consistent maximum elevation difference for a building pad can be calculated. The elevation difference for a driveway up should be calculated using the top of the curb on the lower side of the driveway. The elevation difference for a driveway down should be calculated using the top of the curb on the higher side of the driveway. The driveway slope is a function of the length of the driveway as well as the elevation difference. Where flexibility is allowed for the building setback, the driveway slope can be made less steep by making the driveway longer.
3. The widths of slopes between pads and surrounding features are affected by the vertical distances between them. It is necessary to verify that the slopes do not occupy so much space on adjacent lots that the level pad becomes too small to be useful or whether retaining walls will be required. Typically, building pads on residential sites where fences may be built extend to five feet beyond the property line before sloping down to the adjacent pad.
4. Vertical differences between adjacent pads of less than 0.5 ft (0.15 m) should be avoided. It is simpler to build three adjacent pads at one elevation and a fourth pad 0.6 ft (0.18 m) different, than to build three pads each 0.2 ft (0.06 m) different.

On subdivisions that are fairly level, the high point of the swale will be at or near the center of the back of the building (Fig. 1B.6). On subdivisions that are built on hillsides, the high point of the swale will be moved toward the high side (Fig. 1B.7). On lots with narrow side yards, a system of area drains and underground piping may be needed (Fig. 1B.8).

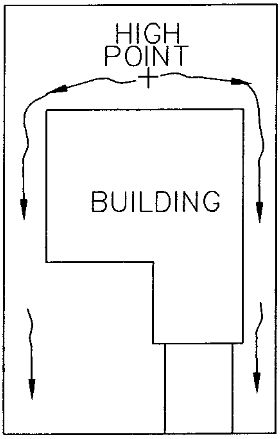


FIGURE 1B.6 Type A drainage on level tract lot.

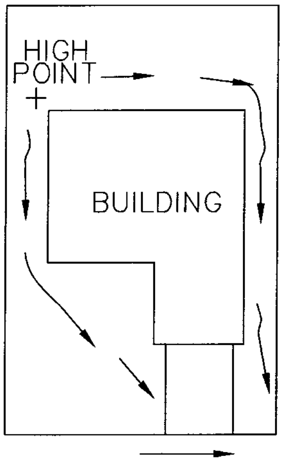


FIGURE 1B.7 Type A drainage on hillside tract lot.

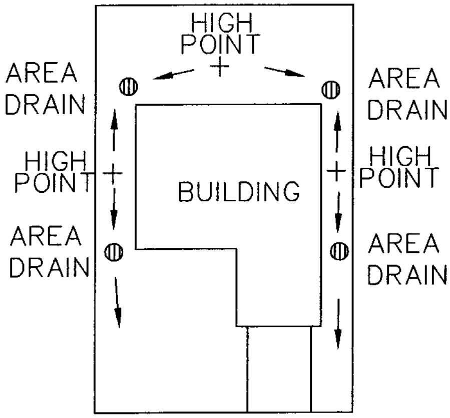


FIGURE 1B.8 Type A drainage with area drains.

1B.7.2 Building Pads with Storm Water Inlets

The elevation of building pads for commercial, industrial, multifamily residential, and single family detached buildings where drainage inlets will be provided is determined as follows:

1. The pad must be determined so that the areas surrounding the pad slope away from the building.
2. Building codes require that the *protective slope*, unless paved, must be at least 0.5 feet below the elevation of the finished floor. The protective slope is the earth against the outside of the foundation
3. The *storm water release point* should not be more than 1.0 ft (0.3 m) above any on-site storm water inlet. The *drainage release point* is that elevation and location where the runoff will leave the property if all the on-site storm water inlets fail to function.
4. The appearance of the building with respect to the street and other surroundings should be considered. If the buildings are much different in elevation from adjacent buildings and improvements, they will look out of place.

The size of the building pad should be designed to extend beyond the building a distance recommended by the soils engineer. Usually, the minimum distance outside the foundation to provide room to work for construction equipment and personnel is 5 ft (1.5 m). A greater distance may be required to provide for foundation support. The pad elevation should be at least 0.2 ft (0.6 m) higher than is necessary to satisfy the other criteria.

1B.8 SITE DRAINAGE DESIGN

On lots within new subdivisions, the runoff for individual lots will be designed to collect and discharge runoff for that lot alone. Normally, collection of off-site runoff reaching the subdivision will be collected and discharged along the boundary of the subdivision. Individual lots must have a swale or ditch within the lot with a drainage flow line around the building to the street or, on very compact lots, to area drains.

1B.9 SURFACE DRAINAGE

Designing storm drainage systems requires an understanding of *hydrology* (the science of the natural occurrence, distribution, and circulation of the water on the earth and in the atmosphere), *hydraulics* (the science of the mechanics of fluids at rest and in motion), and drainage law. Understanding the elements of the design of storm facilities and their coordination with surface improvements and underground utilities is essential. Drainage law varies from location to location and from time to time, so local drainage laws must be investigated and applied.

The purpose and focus of this chapter is for construction and protection of foundations, so hydrology and hydraulics will not be discussed here; however, there is a brief discussion in Section 1B.14. Determining the volume of storm water and subsurface water to be handled on-site should be determined by a qualified civil engineer or hydrologist. The storm water reaching the site is often generated by a very large area outside of the project site. Storm water reaching the site from areas off-site must be intercepted and safely routed away from the structure foundations. This can be accomplished with swales or ditches and storm water inlets. The amount of runoff and location determines the design of ditches.

When the runoff being handled is very small, and the ditch is less than 100 ft long, a simple note, "Grade To Drain," at the flow line of the ditch on the plan, may be sufficient for construction. Where the volume of runoff is low, slopes should be at least 1%. A flatter slope may become uneven in time.

An unlined ditch with a slope that is steep will erode and can threaten the property improvements. The maximum allowable slope depends on the volume of runoff and the type of soil. If the soil is sandy, the maximum limit for the slope of an unlined ditch should be 2.5%. If the soil is compacted clay and the flow is less than one cubic feet per second (cfs), the slope can be as steep as 6%.

Higher volumes of runoff will require lining the ditch. Where erosion will be a problem, the ditch can be lined with any of a number of materials, such as asphalt, concrete, Gunite, or cobblestone. Economics, velocities, and aesthetic will indicate which choice is best. A minimum slope of 0.3% should be used for concrete-paved ditches. Successful construction of a flatter slope is doubtful.

The cross-section of the ditch must be designed to fit the circumstances and accommodate the flow (see Fig. 1B.9). A "V" ditch is most economical to build. If the ditch is located where people

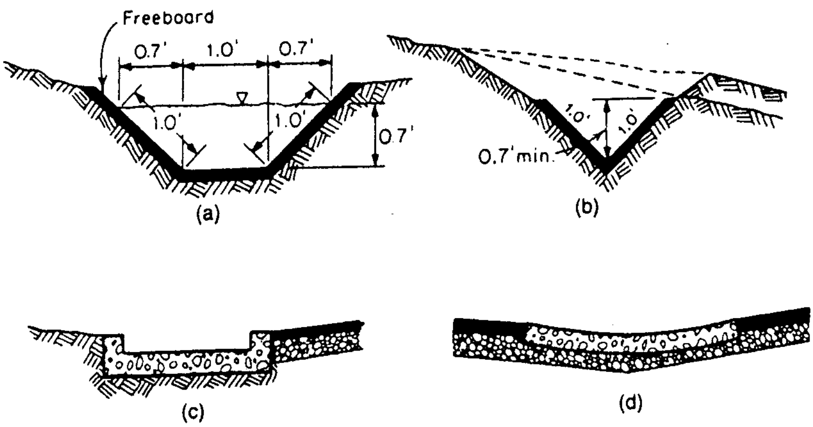


FIGURE 1B.9 Types of ditches: (a) trapezoidal; (b) V ditch; (c) flat-bottomed; (d) curved-bottomed.

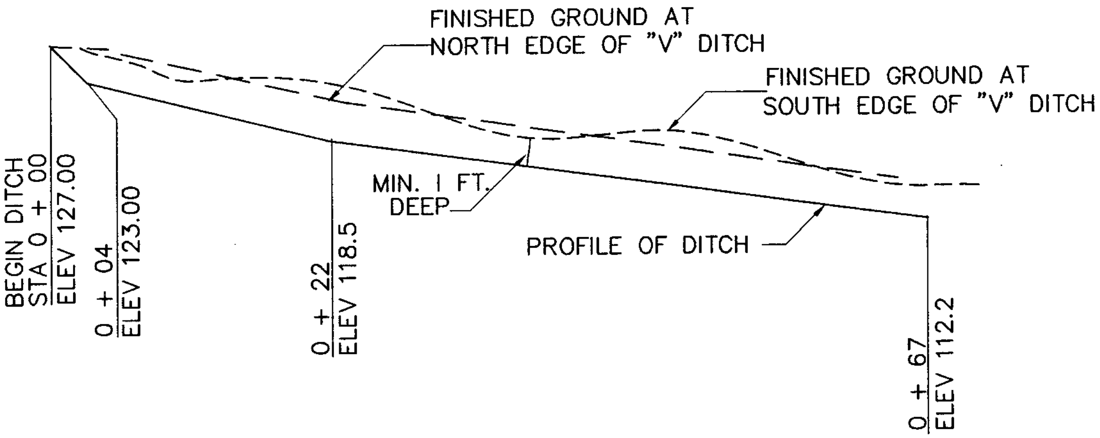


FIGURE 1B.10 Ditch profile.

are likely to step into it, a shallow, flat-bottomed, or curved ditch is better. If the ditch is to carry a large volume of runoff, a trapezoidal ditch is more efficient.

The design of the ditch may be shown entirely on the cross section by showing a minimum depth below existing ground for the flow line of the ditch. The grading contractor can then cut the ditch without needing survey stakes for vertical control. If the design requires more exact vertical control, the flow line profile elevations should be shown on the grading plan or the plan view of the construction plans at grade breaks. The engineer should draw the existing ground and proposed flow line profile and perform the necessary calculation to verify that the ditch will perform as needed.

To design the ditch profile, the existing ground line profile at the centerline or finished ground line profiles at the edges of the ditch are first drawn. A line roughly parallel with and below the lowest ground line profile (Fig. 1B.10) is drawn. The ditch profile must be below the ground at least as much as the ditch is deep. That is, if the ditch is one foot deep, the flow line profile must be at least one foot below the natural ground everywhere at the edge. Otherwise, the ditch will come out of the ground. There should be no more breaks in the profile than are necessary to accommodate the changes in the ground line profile. If the cross slope is steep or erratic, it may be necessary to draw cross-sections at critical points to verify that catch points will be within the property or within a reasonable distance. When the ditch profile is drawn, the slopes must be calculated all along its length.

For each section of the profile, the difference in elevations at the beginning and end of the section is divided by the length of that section. These calculations are continued until the grades all along the profile have been established.

Designing the shape and slope of the ditch is an iterative process. A slope and cross-sectional area including some freeboard for possible wave action or hydraulic jumps is designed. The capacity is determined using the *continuity equation* and *Manning's equation* (described in Section 1B.14) and that cross-sectional area and slope. After comparing the designed capacity to the required capacity, the ditch is redesigned to provide greater capacity or a more economical design.

1B.10 STORM WATER INLETS

At the low point in the ditch, the runoff is collected and routed underground or discharged into an approved waterway. To collect the runoff for removal in an underground system, *storm water inlets* (SWI) are used. Inlets are also referred to as drop inlets (DI), flat grate inlets (FGI), catch basins

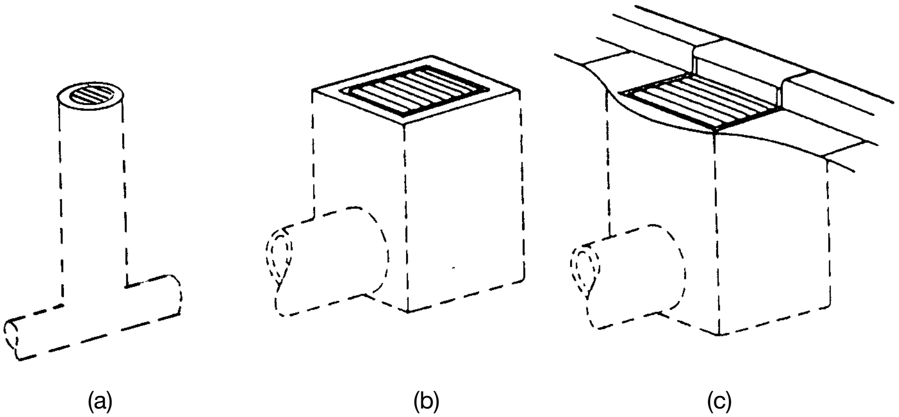


FIGURE 1B.11 Storm water inlets. (a) Area drain; (b) field inlet; (c) catch basin.

(CB), or area drains (AD). The terms "drop inlet" and "flat grate inlet" usually refer to inlets in a large open area such as in a field or parking area. The term "catch basin" usually refers to a storm water inlet located in a street or other area in conjunction with a curb. Area drains usually are small inlets placed in landscape areas.

1B.11 SUBSURFACE DRAINAGE STRUCTURES

A common method for removing ground water is the use of *French drains* (Fig. 1B.12). French drains are ditches in which permeable material is placed then covered with earth or improvements. The permeable material is wrapped in a geotextile to keep silt out. Groundwater moves into the

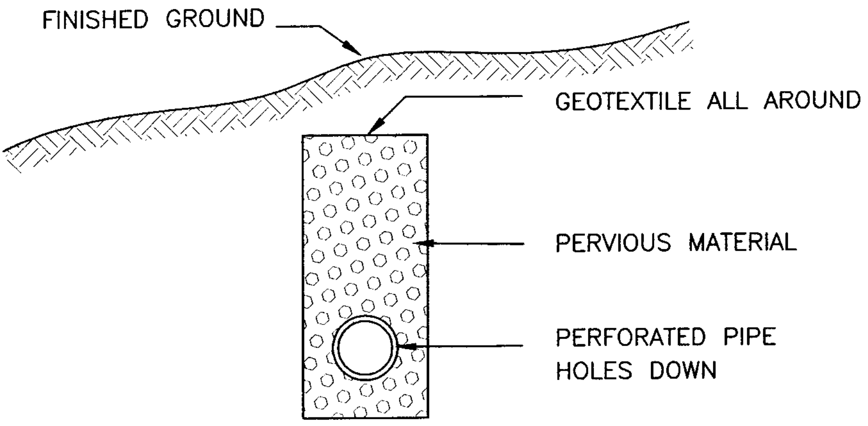


FIGURE 1B.12 French drain.

ditch because the water can move through the permeable material more easily than through the surrounding soil. A perforated pipe (PP) is placed in the ditch with the holes down at a slope designed for the expected flow. Hydrostatic pressure forces the water into the perforated pipe. The perforated pipe is then connected to the underground drainage system at a pipe or storm water inlet.

These French drains are constructed behind and flush with retaining walls to safely remove ground water to protect against damaging hydrostatic pressure, and basement walls to protect against the incursion of moisture. They are also constructed in excavation and embankment benches (Fig. 1B.2) to intercept, collect and remove ground water. The buildup of subsurface water can lubricate interfaces of soils, causing landslides or mud flows if the ground becomes saturated.

1B.12 HIGH WATER TABLES

Treatment of high water tables that can impact the stability of the building pads and thus the foundations is a more complicated matter. In some cases, a thick permeable base may be sufficient, but design responsibility for this condition should be given to soils and structural engineers. A simple, French drain type of mitigation would not be sufficient. In some cases, a matrix of French drains or wells may be recommended to cause draw-down (Fig. 1B.13) of the groundwater.

1B.13 LANDSCAPING PROBLEMS

Consideration must be given to the landscaping design. In clay soils, uneven soil moisture caused by sequential irrigation can cause uneven pressure and heaving. For this reason, if landscaping is to be placed next to the foundation, the clay soil should be removed and replaced with more permeable soil. An alternative might be to provide for simultaneous, even irrigation, but that approach is risky because of inevitable breakdowns of irrigation systems.

Existing trees are often an amenity to be preserved and protected. Where buildings and other structures are to be built in close proximity, an arborist should be consulted. The arborist will make recommendations so that the construction does not damage the tree so much that it has to be re-

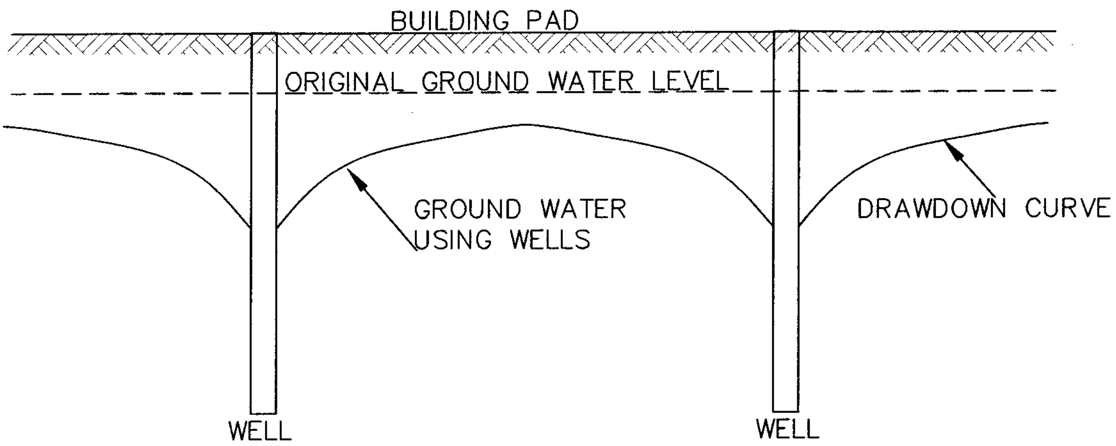


FIGURE 1B.13 Drawdown curves.

moved after the construction has taken place and to determine if the roots of the trees are likely to reach into and damage the foundation substructures (refer to Section 7B).

1B.14 DRAINAGE FLOWS

Understanding the elements of the design of storm facilities and their coordination with surface improvements and underground utilities is essential. Determination of drainage impacts on a site should be prepared only by a civil engineer specializing in land development or a hydrologist. For inexperienced persons to make these determinations can be worse than ignoring the impacts. Describing the formulas and techniques used are described here for information only.

1B.14.1 Hydrology

A formal study of hydrology includes complicated concepts of weather forecasting, storm water runoff, and stream flow routing, as well as the determination of groundwater characteristics. Fortunately, however, for the small areas that are ordinarily involved in land development, the rational formula provides a conservative flow rate that can be used for designing storm water facilities. It is questionable whether areas as small as city lots or any area less than a square mile can be accurately determined using the rational formula but it is the method most commonly used for lack of something better. For projects where the drainage basin affecting the project is larger than 320 acres (120 ha), hydrologists should be brought in and other methods used. The rational formula is given in Eq. 1B.2.

$$Q = kCIA \tag{1B.2}$$

where Q = flow rate (cfs or m³/s)
k = is a factor to account for units:
Imperial 1.008 cfs, per ac, in/h
S.I. 0 .0286 m^3/s, per ha, mm/h
I = rainfall intensity (in/hr or mm/hr)
A = area (acres or hectares)

The *intensity* factor (I) in the rational formula is the rate of rainfall over the area in inches per hour. This factor can be taken from an intensity–duration–frequency (IDF) chart (Fig. 1B.14) supplied by the responsible agency or the weather bureau for the area specific to your project. To find the intensity from the chart, you must know the return period. If the return period is 100 years, the rate of rainfall given is of the most intense storm expected during a 100 year period. This is called a "100 year storm" or a "100 year event." If the return period is 10 years, the rate of rainfall is of the most intense storm expected to occur during a 10-year period and is called a "10 year storm." The larger the interval, the greater the intensity. The jurisdiction responsible for flood control will dictate what return period to use.

The *duration* (D) is the amount of time it takes for a drop of rain to travel from the most distant point in the drainage basin (described below) to the point (ditch, swale, or storm water inlet) for which the drainage quantity is being calculated. This is called the *time of concentration* (t_c). There are complicated formulas to determine the time of concentration. One that is used to approximate the time of concentration for a pear-shaped basin is the Kirpich equation, which can be used for the time of concentration for overland plus channel flow.

$$t_c = 0.0078\left(\frac{L}{S^{0.5}}\right)^{0.77} \tag{1B.3}$$

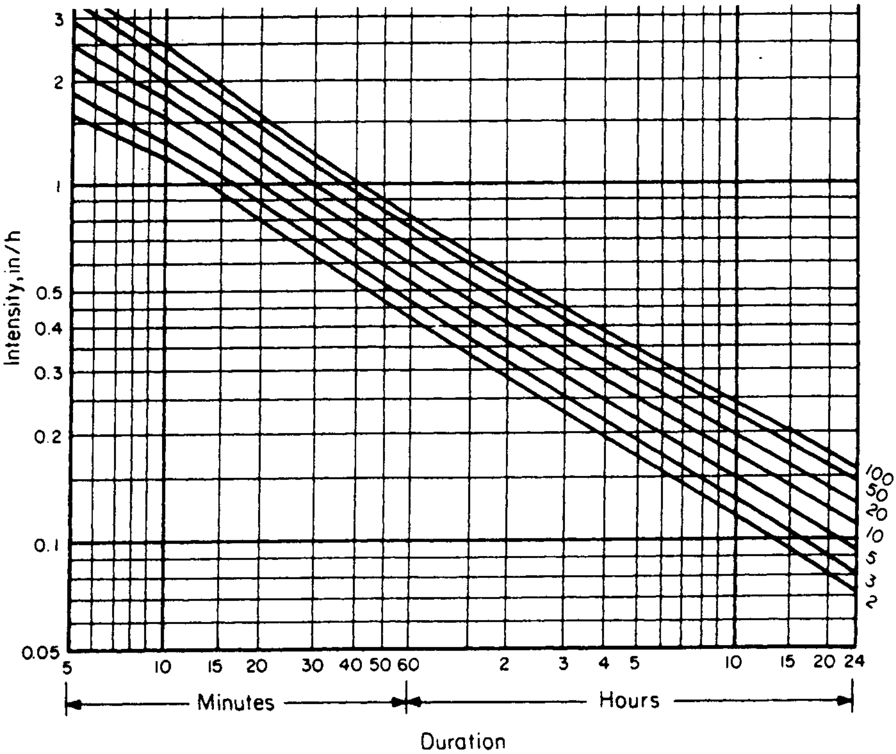


FIGURE 1B.14 IDF Chart for San Jose, CA.

where t_c = time of concentration (min)
L = horizontal projected length of the watershed (feet or meters)
$S = H/L$, where H is the difference in elevation between the most remote location in the watershed and the point of concentration (feet or meters) and L is the horizontal length between those same two points

This formula can be used when determining the time of concentration for existing large, off-site, pear-shaped drainage basins adjacent to the subdivision.

For *small-scale* hydrologic problems, an estimate of the duration is adequate. If the distance or the runoff travel time is from the rooftop to the swale, ditch, or storm water inlet, a duration of 10 to 15 min can be used. Here the most distant point in the drainage basin is judged to be the rooftop. The swale, ditch or storm water inlet is the drainage structure. The time of concentration increases as the drop of rain continues downstream.

When you have the return period and duration, the intensity can be read from an IDF chart (Fig. 1B.14). For example, if the return period is 5 years, find it on the right-hand side of the chart. The 5 is at the end of a diagonal line. Now, find the duration of 15 min at the bottom of the chart. Follow the vertical line representing 15 min until it intersects the diagonal line for the 5 year return period. The intersection falls about halfway between the horizontal lines for 1 and 1.5 in/hr (38 mm/hr). The resulting intensity is 1.25 in/hr (32 mm/hr) as read from the left side of the chart. Notice that as the duration becomes longer, the intensity diminishes. The reason for the decrease in intensity is that peak intensity is seldom sustained for long. The average intensity is less for longer periods of time.

TABLE 1B.2 Typical Runoff Coefficient (C) Values for 5- to 10-Year Frequency Design

Description of area	Runoff coefficients
Business	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.85
Lawns, sandy soil	
Flat, 2%	0.05–0.10
Average, 2 to 7%	0.10–0.15
Steep, 7%	0.15–0.20
Lawns, heavy soil	
Flat, 2%	0.13–0.17
Average, 2 to 7%	0.18–0.22
Steep, 7%	0.25–0.35

Source: Warren Viessman, Jr., Terrence E. Harhaugh, and John W. Knapp, *Introduction to Hydrology,* Intext, New York, 1972, p. 306.

The runoff coefficient (C) in the rational formula (Eq. 1B.2) represents the amount of water running off as a proportion of the total amount of precipitation falling on the area. Of the precipitation that reaches the ground, some will percolate into the soil, some will be taken up by the vegetative cover, some will evaporate, and the remainder will run off. For buildings, runoff coefficients range from 0.70 to 0.95. That is, 70 to 95% of the precipitation falling on the building will run off. The responsible agency may provide a table of coefficients to use. The coefficient reflects the type of soil, type of vegetative cover, and the evenness and degree of slope. Typically, the area will consist of more than one type of cover. In that case, a weighted average should be used.

The area (A) in the rational formula (Eq. 1B.2) is the area of the drainage basin. A drainage basin or watershed is that area of land from which drainage contributes to a particular waterway. Several drainage basins are illustrated in Fig. 1B.15. Ridges W, X, Y, and Z and swales A, B, and C are shown. Drainage basin A is bounded by ridges W, X, and Y and contributes storm water to swale A. Drainage basin B is bounded by ridges Y and Z; water falling there contributes to swale B.

To determine the amount of runoff reaching the point of concentration at A, the drainage basin

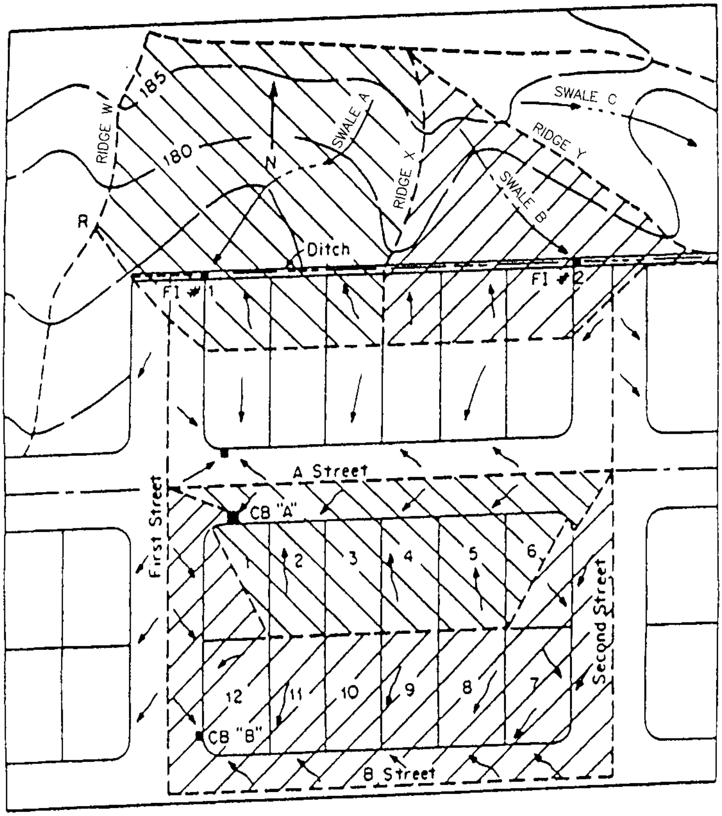


FIGURE 1B.15 Drainage basins in a developed area.

contributing to that point in the swale (waterway) is delineated. Water flowing overland follows the steepest route. The flow line of the steepest route will always be perpendicular to the contours. The land is steepest where the contours are closest. To delineate the drainage basin contributing to a particular point, trace the flow line from point A up the contours at right angles (Fig 1B.15) to the ridge lines. For most projects, the specific project topography will not cover a sufficient area. USGS maps are typically used to determine the drainage basins that will impact the project.

Drainage basins in a developed area are shown in Fig. 1B.15. One drainage area is bounded on the north by the crown on "A" Street, on the south by the lot line between lots 2 through 5 and 8 through 11, and on the east and west by ridges through lots 1 and 6. This drainage area is collected at catch basin "A" (CB "A"). Catch basin "B" collects water from the area bounded by the ridges described above through lots 1 to 11 and by the crowns on First Street, Second Street, and B Street.

The first step in drainage system design is to develop the grading plan. On-site surface drainage basins are created to direct runoff to ditches and storm water inlets. Six of the lots shown in Fig. 1B.15 will interface with existing drainage basins along the northerly tract boundary. In this case, the lots will be graded so that the northern half of each lot drains north and the southern half of each lot drains to A Street. First and Second Streets slope south. Two of the drainage basins established when the lots are constructed this way are delineated in Fig. 1B.15. The storm water falling on the basins will collect in the ditch along the northerly tract boundary and be picked up by fields (FI) #1 and #2.

The ditch here must be designed to accommodate the off-site drainage basins to the north as well. At the northwest corner of the tract, the basin is limited by the point from which the water flows. Water falling south of point R flows south and will not reach the site. Therefore, the limit of the basin is as shown. Once the boundaries of the drainage basins have been defined, their areas can be calculated. If the drainage basin is irregular, use of a planimeter may be the quickest way to determine the area. Convert the area to acres before putting it into the rational formula. Once the quantity of runoff (Q) has been established, the size and type of drainage facilities can be designed. Drainage will run from the high point in the ditch or swale profile to the low point, where an area drain or other drainage facility must be located.

1B14.2 Hydraulics

The understanding of hydraulics for flow in ditches and pipes for the simple situations covered in this section relies only upon the rational formula, the continuity equation, and Manning's equation. The continuity equation simply says the quantity passing a particular location in a pipe or other channel depends upon the cross-sectional area of the flow at that location and the velocity. In other words, the bigger the pipe and the faster the flow, the more flow will pass. The equation is:

$$Q = VA \tag{1B.4}$$

where Q = quantity of flow (cubic feet per second, cfs)
A = area of the cross-section of the flow, square feet, ft^2)
V = velocity, feet per second, fps)

Manning's equation gives the velocity (V). Manning's equation is:

$$V = \frac{0.486}{n} R_H^{0.67} S^{0.5} \tag{1B.5}$$

where n = coefficient of friction
R_H = hydraulic radius
S = slope (ft/ft or m/m)

The slope is taken from the ditch or pipe profile. The n is referred to as Manning's n and is a friction factor for the roughness of the pipe or channel. The value of n to use may be dictated by the responsible jurisdiction. Otherwise, 0.010 can be used for PVC (polyvinyl chloride) pipe and 0.013 can be used for concrete, reinforced concrete, or vitrified clay pipe (CP, RCP, or VCP). Values of n for other materials are given in Table 1B.3. The smoother the conduit, the smaller the value of n is. From Manning's equation (Eq. 1B.5), we see that the smoother the pipe used, the greater is the velocity, thus capacity, produced.

R_H in the equation is the *hydraulic radius*. It accounts for the effect of friction on the flow. The value of R_H is expressed in Eq. 1B.6.

$$R_H = \frac{a}{p} \tag{1B.6}$$

where R_H = hydraulic radius
a = cross-sectional area of the flow (ft^2)
p = wetted perimeter (ft)

The wetted perimeter (Fig. 1B.16) is the length measured on the cross-section that will be wet when the ditch or pipe is flowing at the design capacity.

TABLE 1B.3 Values of n to Be Used with Manning's Equation

Surface	Best	Good	Fair	Bad
Uncoated cast-iron pipe	0.012	0.013	0.014	0.015
Coated cast-iron pipe	0.011	0.012*	0.013*	
Commercial wrought-iron pipe, black	0.012	0.013	0.014	0.015
Commercial wrought-iron pipe, galvanized	0.013	0.014	0.015	0.017
Polyvinyl chloride (PVC) pipe	0.009	0.0 10	0.011	
Smooth brass and glass pipe	0.009	0.010	0.011	0.013
Smooth, lockbar and welded "OD" pipe	0.010	0.011*	0.013*	
Riveted and spiral steel pipe	0.013	0.015*	0.017*	
Vitrified sewer pipe	0.010			
	0.011	0.013*	0.015	0.017
Common clay drainage tile	0.011	0.012*	0.014*	0.017
Glazed brickwork	0.011	0.012	0.013*	0.015
Brick in cement mortar; brick sewers	0.012	0.013	0.015*	0.017
Canals and ditches				
Earth, straight and uniform	0.017	0.020	0.0225*	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.033*	0.035
Rock cuts, jagged and irregular	0.035	0.040	0.045	
Winding sluggish canals	0.0225	0.025*	0.0275	0.030
Dredged earth channels	0.025	0.0275	0.030	0.033
Canals with rough stony beds, weeds on earth banks	0.025	0.030	0.035*	0.040
Earth bottom, rubber sides	0.028	0.030*	0.033*	0.035
Natural stream channels				
1. Clean, straight bank, full stage no rifts or deep pools	0.025	0.0275	0.030	0.033
2. Same as 1, but some weeds and stones	0.030	0.033	0.035	0.040
3. Winding, some pools and shoals, clean	0.033	0.035	0.040	0.045
4. Same as 3, lower stages, more ineffective slopes and sections	0.040	0.045	0.050	0.055
Neat cement surfaces	0.010	0.011	0.012	0.013
Cement mortar surfaces	0.011	0.012	0.013*	0.015
Concrete pipe	0.012	0.013	0.015*	0.016
Corrugated metal pipe	0.025*	0.025*	0.025*	0.025*
Wood stave pipe	0.010	0.011*	0.012	0.013
Plank flumes				
Planed	0.010	0.012*	0.013	0.014
Unplaned	0.011	0.013*	0.014	0.015
With battens	0.012	0.015*	0.016	
Concrete-lined channels	0.012	0.014*	0.016*	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry-rubble surface	0.025	0.030	0.033	0.035
Dressed-ashlar surface	0.013	0.014	0.015	0.017
Semicircular metal flumes, smooth	0.011	0.012	0.013	0.015
Semicircular metal flumes, corrugated	0.0225	0.025	0.0275	0.030
5. Same as 3, some weeds and stones	0.035	0.040	0.045	0.050
6. Same as 4, stony sections	0.045	0.050	0.055	0.060
7. Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
8. Very weedy reaches	0.075	0.100	0.125	0.150

*Values commonly used in designing.

Source: Adapted from E. F. Brater and Horace King, *Handbook of Hydraulics,* McGraw-Hill, New York, 1976, pp. 7–22.

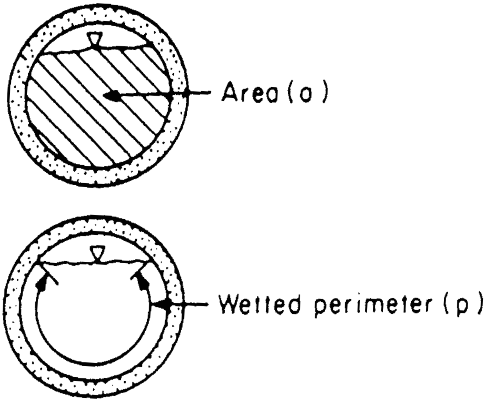


FIGURE 1B.16 The wetted perimeter, $R_h = a/p$.

Using the continuity equation and Manning's equation, the quantity can be calculated. For design purposes on simple projects, pipes can be assumed to be flowing full. Assuming that the pipe is flowing full yields a conservative capacity because when a pipe is full, the increased fiction offsets the increased cross section. Maximum capacity occurs when the height of the flow is at 0.8 the diameter of the pipe.

The sectional area required is compared with the area the pipe ditch section designed provides. If the cross-sectional area of the ditch or pipe is larger, the design will work; if not, using a larger cross section, or a steeper or smoother slope will provide greater capacity. This procedure is repeated using the new V, R_H, and/or S (Fig. 1B.17). If the ditch is long or lined and the section is much larger than necessary, the cross-section should be made smaller, and thus the ditch made cheaper. Here again, if the design of the cross-section is changed, a new cross-sectional area and a new wetted perimeter must be determined and R_H recalculated.

1B.15 DRAINAGE SYSTEMS

Simple residential lots should not require drainage *systems* involving swales, ditches, storm water inlets, and pipes. For larger or more complex sites, a system must be designed. The design involves use of a sophisticated hydraulics software or a time-consuming design procedure.

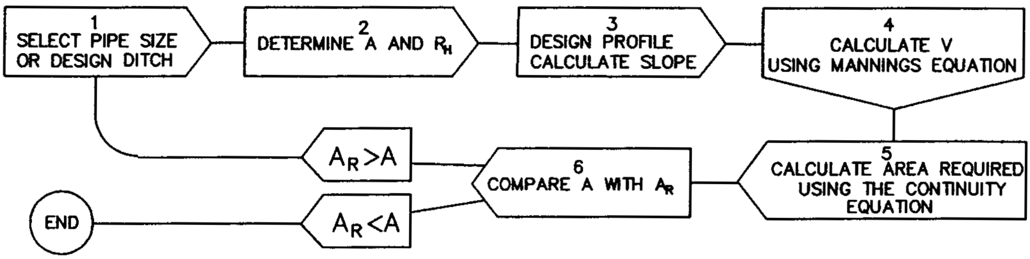


FIGURE 1B.17 Drainage design flow chart.

1B.15.1 Small Sites

On small lots with area drains or other facilities, pipe sizing can be determined easily. In most cases, the pipe size is determined by what makes sense for maintenance—eight to twelve inch diameter pipes will be minimum acceptable for large commercial/industrial sites and four to six inch diameter PVC pipe for single family detached sites. This information can then be used to determine R_H and n The minimum slope as determined by the elevation of the *outfall* (location where the runoff is released into a storm drain or other watercourse) and the finished slope of the land. The value of V can be determined with this information. That value is then used in the continuity equation (1B.4). The quantity of runoff (Q) can be determined using the area of the entire site and the minimum slope to be used. That gives the minimum cross-sectional area for ditches and pipes. Any ditch or pipe that has a greater cross-sectional area or steeper slope than the one used in this exercise will be sufficient to handle the runoff. On small sites, the minimum pipe sizes and slopes usually have much more capacity, as determined by maintenance concerns that are determined by hydraulic considerations.

1B.15.2 Large Sites

The choice of size and slope of ditches and pipes is made based on hydraulic factors, the criteria of the responsible agency or client, criteria affecting the design of the profile as described earlier, and the cost of material and trenching. Although providing for a single drainage basin is simple, providing for multiple basins served by a piping network is complicated. A storm drainage system calculations form is included here (Fig. 1B.18). When filled out correctly, the pipe sizes, slopes, and flow velocities are shown.

The calculations can be made with the use of a spread sheet, as illustrated in Fig. 1B.18, and a computer spread sheet. Once the form has been prepared, it can be used repeatedly by simply replacing data in the appropriate cells while being careful to not affect the formulas. Additional columns can be added for pipe inverts and cover. Hydraulic grade-line calculations can also be included in additional columns. The only problem with making your calculations in this way is that it is easy to accidentally corrupt one of the formulas without realizing it, so unusual care must be taken. Extra care must be taken as well to analyze the results visually to catch any errors.

Filling out the form must be coordinated with design of the profile and may have to be done more than once to coordinate with other criteria. Filling out the form is complicated. Normally the first t_c to use will have additional flow time between where the drop of rain enters the gutter and where it enters the inlet. Flow time can be calculated using the equation

$$t = \frac{l}{60\ V} \tag{1B.7}$$

where t = flow time (min)
l = length (ft) (or m)
V = velocity (fps) (or m/s)

Once the runoff reaches the storm water inlet or ditch, time becomes cumulative. That is, you must calculate the amount of time it takes the flow to travel from the first point of concentration to the second point of concentration and then add that time to the beginning time. Time is added with each leg of the system. This is important because the intensity decreases with time and the IDF chart must be revisited and the intensity determined for each section of the system. Where more than one pipe comes into a point of concentration, the longest time of travel is used.

The area (A) and value for the runoff factor (C) must also be recalculated if the terrain being crossed is inconsistent. With the use of a computer spread sheet or hydraulics software, several alternative slopes and pipe sizes can be examined to determine the most efficient ones.

Point of concen-tration	t_c (min.)	C	I (in/h)	A (ac.)	Total Q (Ft^3/s)	Diameter D (Ft)	Circum-ference Wetted Perimeter	Pipe area (Ft^2)	Hydraulic Radius (R_H)	Slope (Ft/100Ft)	Velocity (Ft/s)	Length (Ft)	Q=VA Capacity (Ft^3/s)
A	B	C	D	E	F	G	H	I	J	K	L	M	N

FIGURE 1B.18 Storm drainage calculations form.

Once the pipe sizes and slopes are determined, the profiles can be designed. Again, the design is an iterative process as shown in Figure 1B.17. The design is further complicated by the constraints of the outfall elevation (invert) and other pipes or underground obstacles that must be crossed. The profile calculation should be begun at the outfall. Of course each down-stream section of pipe must be the same size or larger than the section before, and if there is an abrupt change of slope from steep to shallow, an inlet or manhole must be constructed to account for possible hydraulic jumps.

1B.16 CONCLUSION

Site design should be prepared by a qualified civil engineer experienced in land development, with the help of a soils engineer, and information provided in following sections. In some cases, geologic engineers and, where existing trees are in close proximity to foundations, an arborist should be consulted. The surface of the land is sculpted to direct overland and underground flows away from foundations. The building pads are constructed to certain minimum standards of compaction and structural strength before foundations can be constructed. When the building pad has been constructed to meet specifications, the design of the foundations becomes the responsibility of the structural engineer.

P • A • R • T • 2

SOIL MECHANICS AND FOUNDATION DESIGN PARAMETERS

SECTION 2A
SOIL MECHANICS

RICHARD W. STEPHENSON

2A.1 INTRODUCTION

Rock consists of an aggregate of natural minerals joined by strong and permanent cohesive bonds. *Rock mechanics* is the engineering study of rock.

Soil is defined as natural materials consisting of individual mineral grains not joined by strong and permanent cohesive forces. Natural soils are products of the weathering of rock. *Soil mechanics* is the study of the engineering properties of soil.

2A.2 PHYSICAL CONDITION

2A.2.1 Introduction

A natural soil consists of three separate components: solids, liquids, and gases. The solids are normally natural mineral grains, although they can be human-made materials such as furnace slag or mine tailings. The liquid is usually water, and the gas is usually air. The relative amounts of each of the components in a particular soil may be expressed as a series of ratios. These ratios may be based on relative masses or weights, relative volumes, or relative mass or weight densities. These weight–volume ratios are fundamental to soil mechanics and geotechnical engineering.

2A.2.2 Two-Phase Soil (Dry or Saturated)

If a soil consists of only solids (mineral particles) and voids (either gas- or liquid-filled), then it is a two-phase soil system. Although the void spaces are interspersed throughout the mineral particles, a unit volume of the soil may be viewed as in Fig. 2A.1. Using this figure, some terms can be defined. The unit weight of soil solids (γ_s) is

$$\gamma_s = \frac{W_s}{V_s} \tag{2A.1}$$

where W_s = weight of solid phase
V_s = volume of solid phase

The unit weight of the total soil system (γ_t) is

$$\gamma_t = \frac{W_t}{V_t} \tag{2A.2}$$

where W_t = total weight
V_t = total volume

If the voids are filled with gas (air), then

$$W_v = 0$$
$$\gamma_t = \gamma_{\text{dry}} = \frac{W_s}{V_t} \tag{2A.3}$$

where γ_{dry} = dry unit weight

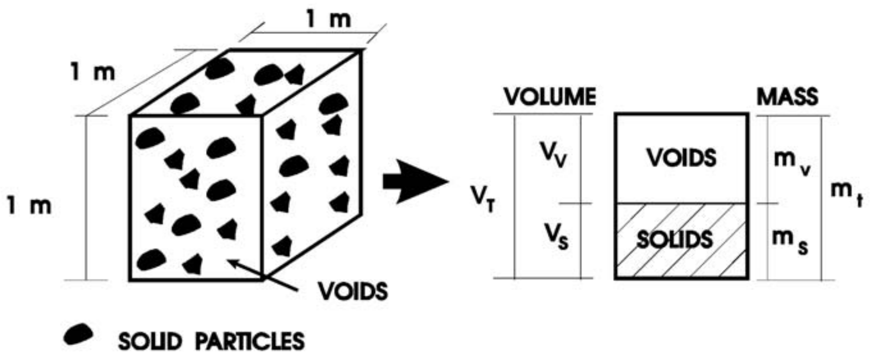


FIGURE 2A.1 Two-Phase representation of soil.

If the voids are filled with liquid (water)

$$W_v = W_w$$

$$\gamma_t = \gamma_{\text{sat}} = \frac{W_s + W_w}{V_t} \tag{2A.4}$$

where γ_{sat} = saturated unit weight
W_w = weight of water

The buoyant unit weight is defined as:

$$\gamma' = \gamma_{\text{sat}} - \gamma_0 \tag{2A.5}$$

where γ_0 = unit weight of water
= 62.4 pcf = 9.807 kN/m^3

The voids ratio (e) is

$$e = \frac{V_v}{V_s} \tag{2A.6}$$

where V_v = volume of voids

Porosity (n%)is defined as

$$n\ (\%) = \frac{V_v}{V_T} \times 100 \tag{2A.7}$$

Water content (w%) is

$$w\ (\%) = \frac{m_w}{m_s} \tag{2A.8}$$

Specific gravity of soil solids (G_s):

$$G_s = \frac{\gamma_s}{\gamma_0} \tag{2A.9}$$

2A.2.3 Three-Phase Soil

Soils are not always either dry or saturated. Often, the voids are partly filled with water. The soil block then consists of a three-phase system, as shown in Fig. 2A.2. Using Fig. 2A.2, some weight-volume relationships can be defined. Moist unit weight is

$$\gamma_{\text{moist}} = \frac{W_t}{V_T} \tag{2A.10}$$

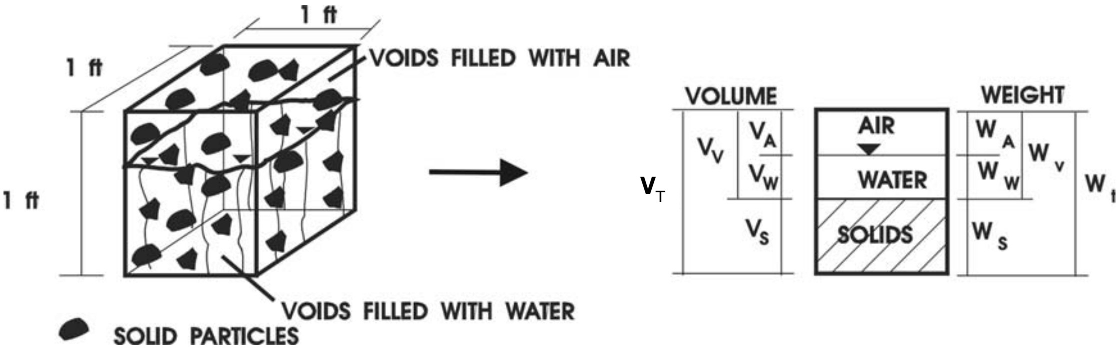


FIGURE 2A.2 Three-phase soil system.

Water content (w%) is

$$w\,(\%) = \frac{W_w}{W_s} \tag{2A.11}$$

Degree of saturation is

$$S_r = \frac{V_w}{V_V} \times 100 \tag{2A.12}$$

2A.3 SOIL IDENTIFICATION AND CLASSIFICATION

2A.3.1 Introduction

Soil is identified and classified using several systems. These systems include (a) the method by which the solid particles are formed; (b) the size of the individual particles; and (c) the engineering properties of the soil.

2A.3.1.1 Soil Formation

Soil can be classified based on the origin of their constituents. The major origins or soil are rock weathering and organic decomposition.

Rock Weathering. Rock weathering can be either mechanical or chemical. Either will result in a large rock mass being broken into smaller particles.

Mechanical weathering. Mechanical weathering can be caused by exfoliation of large rock masses, differential thermal expansion and contraction of minerals within a rock mass, or the freezing and subsequent thawing of water in minute fissures in the rock mass. In addition, mechanical weathering can be caused by the impact by running water on a rock mass, scouring of a rock mass by glacial movement, or breakdown of the rock mass by the impact of wind-blown particles. Rocks are also weathered by the expansion of the roots of vegetation growing in the minute cracks and fissures in the rock.

Chemical Weathering. Chemical weathering occurs when hard rock minerals are transformed into soft, easily erodible matter. The primary chemical processes that weather hard rock minerals include oxidation, carbonation, hydration, and leaching. When a rock containing iron comes into contact with moist air, $2Fe_2O_3 \cdot H_2O$ ("rust") is formed. This rust is very soft and easily eroded from the rock mass.

When water is added to CO_2 in the atmosphere, a weak carbonic acid is formed. This acid decomposes minerals containing iron, calcium, magnesium, sodium, or phosphates.

Hydration. Hydration is the taking up of water, which is then bound chemically to form new minerals. Not only does hydration alter the mineral, often to a softer state, but it also causes a volume increase that in turn increases the decomposition of the rock mass.

When water comes into contact with salt, gypsum, feldspars, or limestone, the minerals will dissolve. The dissolved minerals are then transported and redeposited elsewhere. This is called *leaching*.

Organic Decomposition. Because organic soils are so unique and difficult to deal with, only a relatively small fraction of the soil solids needs to be organic for the organic constituent to control its engineering behavior.

Peat. Peat is formed by the growth and subsequent decay of plants. These soils tend to be fibrous, black in color, and smell like rotten eggs.

Coral. Coral is the accumulation of fragments of inorganic skeletons or shells of organisms. These soils are easily identified visually.

2A.3.1.2 Mineralogy

A mineral is an inorganic compound found in nature. For engineering purposes, the minerals are separated into rock and soil minerals.

Quartz. Quartz is the principal mineral in granite, sands, and rock flour. The mineral is colorless, transparent, and quite hard. Quartz is very resistant to chemical weathering. Quartz has a specific gravity of 2.66.

Feldspar (Silicates of Aluminum). Feldspars are important because these minerals chemically weather into clay minerals. Their specific gravities are about 2.7.

Micas. Micas are the primary mineral in granites and gneiss. Mica can be split into thin, elastic sheets. It is colorless and transparent and has a specific gravity of 2.8.

Carbonates. Carbonates are derived from the chemical weathering of calcium-bearing feldspars and other calcium-bearing rocks and have been reformed into new rock masses. Principal carbonates include calcite, dolomite, and the various limestones. The carbonates and quite soft and white to colorless and are highly susceptible to chemical weathering. Their specific gravities vary from 2.7 to 2.8.

Principal Clay Minerals. The principal clay minerals are listed below.

Kaolinite. Kaolinite is derived from the chemical weathering of feldspars and other aluminum-bearing rocks. Its primary structure consists of a single sheet of gibbsite bound to a sheet of silicon (Fig. 2A.3). Successive two-layer sheets are bound together by weaker hydrogen bonds. These hydrogen bonds can be relatively easily broken, yielding a basic two-layer sheet structure.

Illite. Illite takes the structure of kaolinite a further step. The hydroxyls of the octahedral layer are stripped of their hydrogen ions on both sides, and the oxygen ion is the tip of a tetrahedral layer on both sides of the octahedral layer. The octahedral layer is electrically neutral. However, the tetrahedra are not neutral. Approximately one tetrahedron in seven contains an aluminum (+3) in place of the usual silicon (+4) because of isomorphous substitution. This results in an overall charge deficiency in each of the tetrahedral layers. This charge deficiency draws potassium ions (+1) into the structure in that octagonal void that exists, as discussed previously. Thus, a potassium ion fits into this void and the void of the next sheet of illite. It is drawn by the negative charge of the tetrahedral layer and thereby holds the sheets together. This bond is obviously not as strong as the hydrogen bond of kaolinite, but it is not weak either.

Montmorillonite (Smectite). The make-up of montmorillonite is very similar to that of illite. Montmorillonite is a three-layer mineral, except the tetrahedral layer is relatively neutral with almost no substitution of aluminum for silicon. The octahedral layer has charge deficiencies with alu-

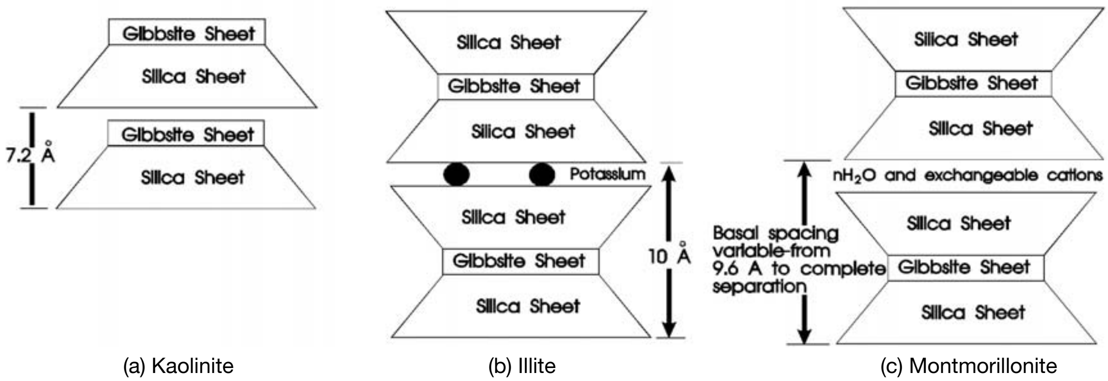


FIGURE 2A.3 Structure of principal clay minerals: (a) kaolinite, (b) illite, (c) montmorillonite.

minum in the vacated positions. Since the seat of the charge deficiency is in the center of the sheet, there is a weaker attraction by the negative charge to outside positive charges. Instead of strongly attracting potassium, hydrated ions, such as sodium, are weakly attracted. The sodium ions are weakly attracted to the potassium ions and to the negative faces of the sheets of montmorillonite because of its dipole nature. Thus montmorillonite surrounds its sheets with oriented water and hydrated cations. Soils with montmorillonite are known for their propensity to swell in the presence of water.

2A.3.1.3 Grain (Particle) Size Classification

One method of classifying soils is by the size of the individual particles. The size and distribution of soil particles are determined by performing a grain size analysis.

Coarse-Grained Soils. Coarse-grained soils behave in nature as individual particles. They are subdivided into gravel and sand. *Gravel soils* have a particle sizes coarser (larger) than about the #4 or the #10 mesh sieve opening, depending upon which particular classification system is used. *Sands* have particle sizes finer than gravel (#4 or #10 mesh) and coarser than the #200 mesh sieve. Coarse sand particles pass the #4 sieve and are retained on the #10 mesh sieve. Medium sand has a particle size that is smaller than the #10 and larger than the #40 mesh. Fine sand has particles in the #40 to the #200 mesh size.

Fine-Grained Soils. Fine-grained soils largely behave as a mass and not as individual particles. Their particle sizes can be divided, however, into silt and clay. *Silt* particles are smaller than the #200 mesh (0.0074 mm) but larger than 2 μm. Silts are derived by the mechanical weathering of rock. *Clay* particles are smaller than 2 μm. They are developed by the chemical weathering of rock minerals.

2A.3.1.4 Grain (Particle) Size Distribution

The particle size distribution analysis of a soil involves determining the relative amounts of particles within given size ranges in a soil mass. Different test methods are used for coarse-grained soils and fine-grained soils.

The particle size distribution of a coarse-grained soil is determined by a sieve analysis (ASTM D-422). The test uses a set of calibrated sieves, stacked in descending opening size, through which the soil is passed. The largest screen opening is several inches and the smallest size commonly used is #200 mesh (0.074 mm). Intermediate size screens are used to separate various sizes of particles down to #200 mesh. Larger particles are retained on the upper sieves, whereas the smaller particles pass through onto the lower sieves. The grain size distribution of coarse-grained soils influences their density, permeability, shear strength, and compressibility.

The grain size distribution of fine-grained soil is determined by sedimentation. The method is based on Stoke's Law:

$$D = \sqrt{\frac{18\eta v}{\gamma_s - \gamma_f}} \tag{2A.13}$$

where: D = diameter of sphere
η = viscosity
v = velocity of fall of sphere
γ_s = unit weight of sphere
γ_f = unit weight of fluid

A sample of the soils is mixed into a suspension in water and the suspension placed in a sedimentation cylinder. Using Stoke's Law, it is possible to calculate the time t for particles of diameter D to settle a specified depth in the suspension. A hydrometer is used to measure the specific gravity of the suspension. Details of the test are given in ASTM D 422.

Although the grain size distribution of fine-grained soils is often performed, their properties are more affected by structure, shape, and geologic origin than particle size distribution.

The distribution of particle sizes in a soil is represented by a *grain size distribution curve*. This curve is a semilogarithmic plot with the ordinates being the percentage by weight of particles smaller than the size given by the abscissa (Fig. 2A.4). The flatter the distribution curve, the larger the range of particle sizes in the soil. The steeper the distribution curve, the smaller the size range. A soil that has a relatively even distribution of particle sizes is called a *well-graded soil*. A soil that consists primarily of particles of one size is called *uniform*. Descriptive coefficients are used to quantify various characteristics of the grain size distribution curve.

The particle size corresponding to any specified value on the ordinate (percent smaller) curve can be read on the size distribution curve For example, the diameter of particle that corresponds to

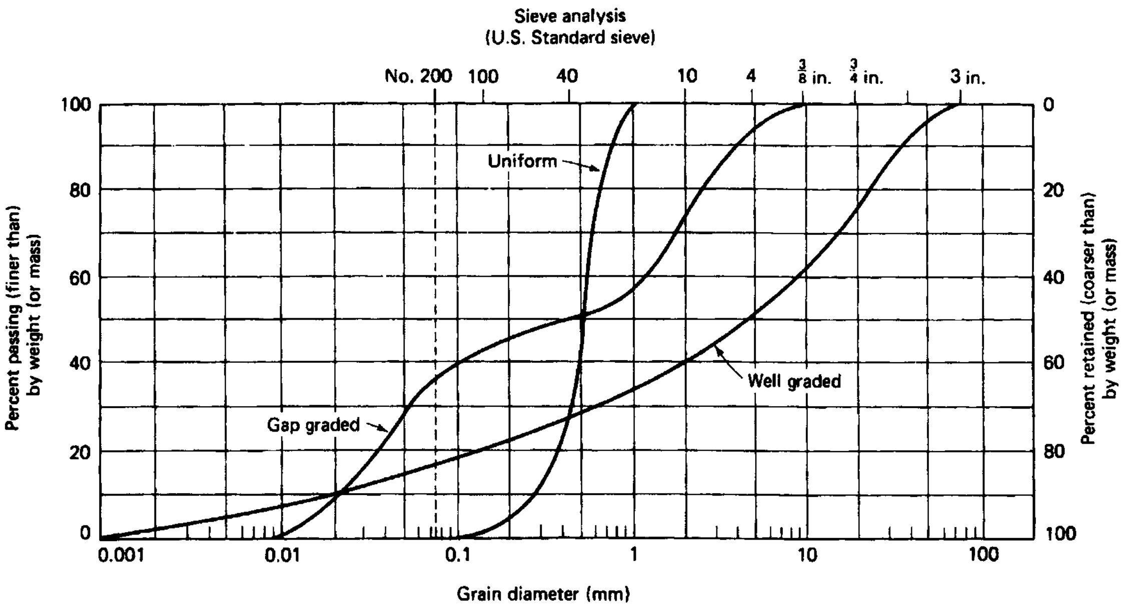


FIGURE 2A.4 Typical grain size distribution curves.

the 50% smaller ordinate is the D_{50} size. Similarly, the D_{10} size is the diameter of grains that only 10% of the soil is finer than. The D_{10} is known as the effective diameter of the soil.

The uniformity coefficient is defined as:

$$C_u = \frac{D_{60}}{D_{10}} \tag{2A.14}$$

and is a measure of the general slope of the grain size distribution curve. A C_u near unity indicates a one-size (uniform) soil. On the other hand, a $C_u = 6$ indicates a well-graded soil.

The coefficient of curvature (C_z) indicates the constancy of the slope of the grain size distribution curve.

$$C_z = \frac{D_{10}^2}{D_{60} \times D_{10}} \tag{2A.15}$$

A C_z of between 1 and 3 indicates a well-graded soil. C_z values outside these limits indicate either uniform or gap-graded soils.

2A.3.1.5 Grain Shape

Coarse-Grained Soils. The shape of individual coarse-grained soil particles (larger than the #200 mesh) can be classified as being bulky, platey, or needle-like in shape. *Bulky-grained* particles are roughly equidimensional, i.e., the length ≈ width ≈ height. This is true of most gravels, sands, and silts. Bulky-grained soils are strong and relatively incompressible. Sands that are derived from mica have shapes that are *plate shaped*, i.e. length ≈ width ≫ thickness. Platey particles are more elastic and prone to breakage than are bulky-grained particles. Some coral and some clays have particle shapes that are *needle shaped*. These particles are resilient under low static loads but tend to break under higher loads.

Particle Angularity. Under load, angular corners crush and break but resist displacement. Smoother particles are less resistant to displacement but are less likely to crush. Particles are classified as angular, subangular, subrounded, rounded, or well rounded.

Fine-Grained Soils. Silts are products of the mechanical weathering of rock, and therefore their shape is primarily bulky. Because the clay minerals are crystalline with an orderly, sheet-like molecular arrangement, the clay particles break down into very small (<2μm) sheets where the length ≈ width ≫ thickness.

2A.3.1.6 Soil Plasticity

Soil plasticity is defined as the ability to undergo deformation without rupture. Plasticity of a soil is caused by presence of plate-shaped clay particles in the soil. The negative charges on the surfaces of the clay platelets attract and bind polar water molecules to their surfaces. The negative end of the water dipole similarly attracts other water molecules. This phenomenon continues with the attractive forces decreasing at larger distances from the clay surface. This layer of tightly bound water around the clay particle is known as absorbed water. The absorbed water thickness will vary depending upon the strength of the surface charge as well as the presence of cations in the water phase (Fig. 2A.5).

Atterberg Limits. The clay particles are held together by a number of different forces including electrical attraction, hydrogen bonding, cation sharing, and Van Der Wall's forces. When the particles are relatively far apart, there is relatively little particle-to-particle attraction. Therefore, it is relatively easy for particles to slip past each other. However, as the particle spacing reduces due to an applied force or the removal of water, the interparticle attraction will increase and the slip potential between the particles will reduce.

As a clay slurry is reduced in volume by desiccation, it passes from a liquid phase through a vis-

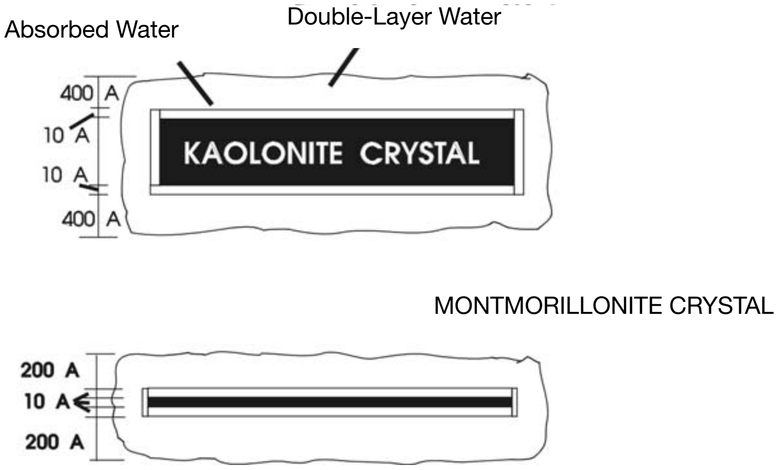


FIGURE 2A.5 Clay particle.

cous fluid stage, through a plastic stage, and finally to a solid state as the particle-to-particle attraction forces increase and the slip potential decreases. Atterberg[1] divided these ranges arbitrarily into five ranges as defined by the water content of the slurry. Two important ranges for engineering work are defined by three limits listed below.

Liquid Limit. The liquid limit (LL) is defined as the lower limit of viscous flow. Casagrande[2] defined the LL as the water content at which a 2 mm wide trapezoidal groove cut in moist soil held in a special cup would close 0.5 inch along the bottom after 25 taps on a hard rubber plate (ASTM D-4318).

Plastic Limit. The plastic limit (PL) is defined as the lower limit of the plastic range, and is the water content at which a sample of soil begins to crumble when rolled into a thread ⅛ inch in diameter (ASTM D-4318).

Shrinkage Limit. The shrinkage limit is the lower limit of volume change upon drying, and is defined as the water content at which the soil is at a minimum volume as it dries out from saturation (ASTM D-4943).

Index properties are used to provide relative measures of the plasticity of a soil. The *plasticity index* (PI) is defined as

$$\mathrm{PI} = \mathrm{LL} - \mathrm{PL} \tag{2A.16}$$

The *liquidity index* is defined as:

$$I_L = \frac{w_n - PL}{PI} \tag{2A.17}$$

The *activity number* (A) is applied to plastic soils in reference to their propensity for undergoing volume change in the presence of varying moisture conditions:

$$A = \frac{PI}{\% < 0.002\ \mathrm{mm}} \tag{2A.18}$$

TABLE 2A.1 Typical Values of Activity

Mineral	Activity
Kaolinite	0.2–0.4
Illite	0.5–0.9
Calcium montmorillonite	1.0–2.0
Sodium montmorillonite	4 or more

Uses of the Atterberg Limits. In general, the index properties are indicative of remolded soil properties. The liquid limit indicates compressibility, geologic history of the deposit, and the undrained shear strength, among other things.

Clay mineralogy. Since the clay content (fraction smaller than 0.002 mm) governs the Atterberg limits of a soil, the Atterberg limits are thus an indicator of the type of clay mineral. The activity (A) number is used to estimate the type of clay mineral in a soil (Table 2A-1).

Volume change potential. The propensity for a clay soil to undergo expansion or shrinkage with increases or decreases in moisture content can be estimated using the Atterberg limits (Table 2A.2). As the plasticity index increases and the shrinkage limit decreases, volume change potential increases (Table 2A.3).

Compressibility. Skempton[3] developed a statistical relationship between the liquid limit and the compression index for remolded soils:

$$C_c = 0.007(LL - 10) \tag{2A.19}$$

Research at Cornell[4] has justified the Terzaghi expression for undisturbed, normally consolidated soils:

$$C_c = 0.007(LL - 10) \tag{2A.20}$$

These expressions apply only to normally consolidated soils and are valid to about ±30%.

Geologic History. A plot of liquidity index versus depth will tend to smooth out depositional differences between soils. Deposits with a common depositional history show a smooth curve. A normally consolidated deposit will show a continuous decrease in liquidity index with depth.[5] A plot of liquidity index versus vertical effective consolidation pressure provides a means of estimating whether or not the soil has been overconsolidated.

Undrained Shear Strength. For normally consolidated soils, the curve of water content versus logarithm of effective vertical consolidation pressure, $\bar{p}$, and water content versus logarithm of undrained shear strength, s_u, are parallel. Thus, for a normally consolidated soil, the ratio of $s_u/\bar{p}$ is a constant. Skempton developed a statistical relationship between plasticity index and $s_u/\bar{p}$:

$$\frac{s_u}{\bar{p}} = 0.11 + 0.0037PI \tag{2A.21}$$

TABLE 2A.2 Swell Potential

LL	Swell Potential
0–30	Slight to low
31–50	Moderate to intermediate
>50	High

TABLE 2A.3 Estimation of volume change potential from Atterberg Limits

Plasticity index	Shrinkage limit	Probable expansion, % total volume change dry to saturated	Degree of expansion
>35	<11	>30	Very high
25–41	7–12	20–30	High
15–28	10–16	10–20	Medium
<18	>15	<10	Low

This expression is valid for normally consolidated soils tested in situ by vane shear and for soils tested in unconfined compression.

2A.3.1.7 Soil Aggregate Properties

Primary Structure. Soil structure is the arrangement of individual soil grains in relation to each other. Terzaghi classified soils into three broad classes: cohesionless, cohesive, and composite soils.

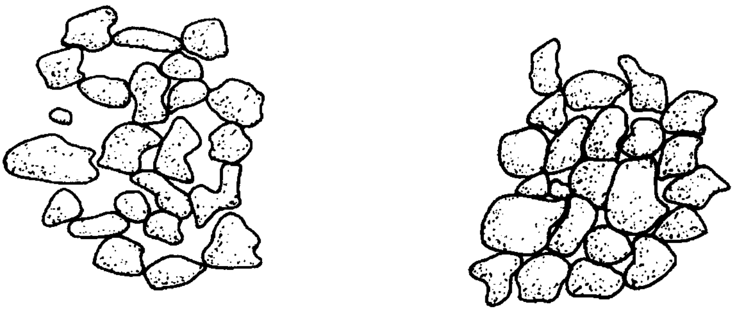
Cohesionless soils. Cohesionless soils consist of particles of gravel, sand, or silt, depending upon the size of their individual particles. The structure of these soils can take two forms: single-grained or honeycombed.

Single-grained. Gravel, sand, or silt particles greater than about 0.02 mm settle out of suspension in water as individual grains independent of other grains. Their weight causes the grains to settle and roll to equilibrium positions practically independent of other forces. The particles may come to equilibrium in a loose condition, a dense condition, or anywhere in between (Figure 2A.6). Relative density is used to measure the compactness of a single-grained soil.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \tag{2A.22}$$

A soil that is in its most dense condition will have a relative density of 100%. Vibration can cause rapid reduction of the volume of a loose single-grained soil structure.

Honeycombed. When silt grains with diameters between 0.0002 and 0.02 mm settle out of suspension, molecular forces at the contact areas between particles may be large enough compared to



(a) Loose (b) Dense

FIGURE 2A.6 Typical single-grained soil structure.

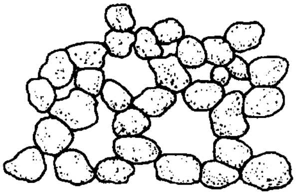


FIGURE 2A.7 Honeycombed structure.

the submerged unit weight of the grains to prevent the grains from rolling down immediately to positions of equilibrium among other grains already deposited. Electrostatic and other forces can cause miniature arches to form, bridging over large void spaces (Figure 2A.7). Because the particles themselves are strong, these honeycombs are capable of carrying relatively large static loads without excessive volume change. However, if the load increases beyond the soil's critical value, large and rapid volume decrease will occur.

Cohesive Soils. Cohesive soils are fine-grained soils whose particles form either a flocculated or dispersed structure.

Particles smaller than 0.0002 mm will not settle out of solution individually due to the Brownian motion. However, because of the electrical charges that exist on the surfaces and edges of clay particles, the negatively charged surfaces are attracted to the positively charged edges and relatively large edge-to-face aggregates or flocs are formed. These flocs grow to masses great enough to settle out of suspension and form *flocculated soils* (Figure 2A.8(a)).

If a flocculated soil is remolded, then the edge-to-face structure collapses and the particles slip into nearly parallel positions. In this configuration, there is very little particle-to-particle contact. This structure is called an *oriented or dispersed structure* (Figure 2A-8(b)).

Composites. In natural clays that contain a significant proportion of larger particles, the structural arrangement of the particles can be highly complex. Single grains of silt and/or sand can be interspersed within a clay platelet matrix.

Consistency and sensitivity of clays. Consistency is a measure of the degree with which a clay soil will resist deformation when loaded. Consistency of clay is measured by the unconfined compression test, which will be described later. Table 2A.4 is often used to describe the consistency of clay.

Sensitivity describes the loss of strength of a soil upon remolding. Numerically, it is defined as the unconfined compressive strength of the undisturbed soil (q_u) divided by the unconfined compressive strength of the same soil remolded at an identical water content (q_r). Table 2A.5 classifies soils according to their sensitivity.

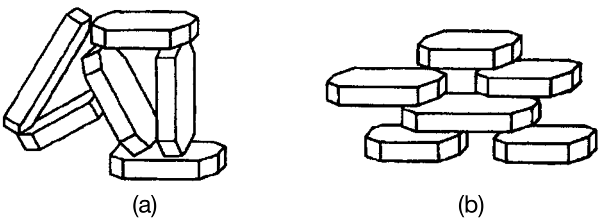


FIGURE 2A.8 Soils that are (a) flocculate; (b) dispersed.

TABLE 2A.4 Consistency of Clay Soils

Consistency	Unconfined compressive strength (tsf)
Very soft	<1/4
Soft	1/4 to 1/2
Medium	1/2 to 1
Stiff	1 to 2
Very stiff	2 to 4
Hard	>4

2A.3.2 Soil Classification

Soil classification is the placing of a soil into a group of soils, all of which exhibit similar characteristics.

2A.3.2.1 Classification According to Origin of Natural Deposits

All soils are either residual or transported.

Residual Soils. Residual soils are formed when rock weathers faster than erosion can carry the soil particles away. In these soils, all soluble materials have been leached out. The chemical disintegration becomes less active with depth, and the alteration becomes less and less with depth until the parent rock is reached. These soils tend to be highly mixed-grained, with gravel or cobble-sized remnants of chemically resistant rock intermixed with clay particles. The particles tend to be very angular.

Transported Soils. Transported soils consist of soil particles that have been moved from their original location by various agents and redeposited. The types of deposits are classified according to their erosion and transportation methods.

Aeolian deposits are wind-transported and deposited. Aeolian deposits are characterized by their high degree of sorting (all-one-size particles) and the uniformity of the deposits. Principal aeolian soils include sand dunes and loess (wind-deposited silt).

Gravitational deposits are soil deposits that have collected at the base of mountains. Chief among the gravitational deposits is talus, which is the accumulation of rock and soil that builds up at the base of cliffs.

2A.3.2.2 Engineering Soil Classification

AASHTO Soil Classification. The American Association of State Highway and Transportation Officials (AASHTO) soil classification identifies kinds of soil in terms of their suitability to serve as a highway base course. This system classifies soils into one of eight groups: A-1 through A-8 (Table 2A-6). A-1 soils consist of well-graded gravels and sands and are the best soils for high-

TABLE 2A.5 Sensitivity of Clay Soils

Sensitivity	q_u/q_r
Insensitive	<1.0
Low	1.0 to 2.0
Medium	2.0 to 4.0
Sensitive	4.0 to 8.0
Extra	8.0 to 10.0
Quick	>10.0

way subgrades. Organic soils that are highly unsuitable for use as a subgrade are classified as A-8. Some groupings are further subdivided. The classifications are based on the soil's grain size distribution and plasticity of the fraction passing the #40 sieve.

The Unified Soil Classification System. This is based on the work of Casagrande.[6] This system classifies soils into three major groups based on their predominant particle size and plasticity. Soils are coarse-grained (sand or gravel) if 50% or more of the soil particles by weight are

TABLE 2A.6 AASHTO soil classification system

TABLE 1 Classification of Soils and Soil-Aggregate Mixtures

General Classification	Grandular Materials (35% or less passing No. 200)			Silt-Clay Materials (More than 35% passing No. 200)			
Group Classification	A-1	A-3[A]	A-2	A-4	A-5	A-6	A-7
Sieve analysis, % passing:							
No. 10 (2.00 mm)	...	...	...	...	...	...	...
No. 40 (425 μm)	50 max	51 mm	...	...	...	...	...
No. 200 (75 μm)	25 max	10 max	35 max	36 mm	36 mm	36 mm	36 mm
Characteristics of fraction passing No. 40 (425 μm):							
Liquid limit	...	...	[B]	40 max	41 mm	40 max	41 mm
Plasticity index	6 max	N.P.	[B]	10 max	10 max	11 mm	11 mm
General rating as subgrade	Excellent to Good			Fair to Poor			

[A]The piecing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

[B]See Table 2A.6B for values.

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TABLE 2 Classification of Soils and Soil-Aggregate Mixtures

General Classification	Grandular Materials (35% or less passing No. 200)							Silt-Clay Materials (More than 35% passing No. 200)			
	A-1		A-3	A-3				A-4	A-5	A-6	A-7
Group Classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, % passing:											
No. 10 (2.00 mm)	50 max	...	...	...	...	...	...	...	...	...	...
No. 40 (425 μm)	30 max	50 max	51 mm	...	...	...	...	...	...	...	...
No. 200 (75 μm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 mm	36 mm	36 mm	36 mm
Characteristics of fraction passing No. 40 (425 μm):											
Liquid limit	...	...	40 max	41 mm	40 max	41 mm	40 max	41 mm	40 max	41 mm	
Plasticity index	6 max		N.P.	10 max	10 max	11 mm	11 mm	10 max	10 max	11 mm	11 min[A]
Usual types of significant constituent materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General rating as subgrade	Excellent to Good							Fair to Poor			

[A]Plasticity index of A-7-5 subgroup is equal to or less than *LL* minus 30. Plasticity index of A-7-6 subgroup is greater than *LL* minus 30 (see Fig. 1).

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larger than the #200 mesh sieve. Soil are fine-grained (silt or clay) if 50% or more of the soil particles by weight are smaller than the #200 mesh sieve. If the soil contains organic matter, the soil is designated either organic or peat, no matter what the size of the mineral grains. Each class of soil is further divided into subclasses depending upon either its grain size distribution (coarse-grained soils) or its plasticity (fine-grained soils). All soils are given a two-letter designation descriptive of the soil's primary and secondary classification.

TABLE 2A.7 Unified soil classification chart

Criteria for assigning group symbols and group names using laboratory tests[A]				Soil classification: Group symbol	Soil classification: Group name[B]
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines	Fines classify as ML or MH	GM	Silty gravel[F,G,M]
		More than 12% fines[C]	Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean sands Less than 5% fines	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			Cu < 6 and/or 1 > C~> 3"	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	Silts and clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$	OL	Organic clay[K,L,M,N] / Organic silt[K,L,M,O]
	Silts and clays Liquid limit 50 or more	inorganic	P1 plots on or above "A" line	CH	Fat clay[K,L,M]
			P1 plots below "A" line	MH	Elastic silt[K,L,M]
		organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$	OH	Organic clay[K,L,M,P] / Organic silt[K,L,M,O]
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A]Based on the material passing the 3-in (75 mm) sieve.
[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay.
[D]SandS with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay.

[E]$Cu = D_{60}/D_{10} \quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F]If soil contains ≥ 15% sand, add "with sand" to group name.
[G]If fines classify as CL-CL, use dual symbol GC-CM, or SC-SM.
[H]If fines are organic, add "with organic fines" to group name.
[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J]If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[K]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L]If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
[M]If soil contains ≥ 30% plus #200, predominantly gravel, add "gravelly" to group name.
[N]P1 ≥ 4 and plots on or above A line.
[O]P1 < 4 or plots below A line.
[P]P1 plots on or above A line.
[Q]P1 plots below A line.

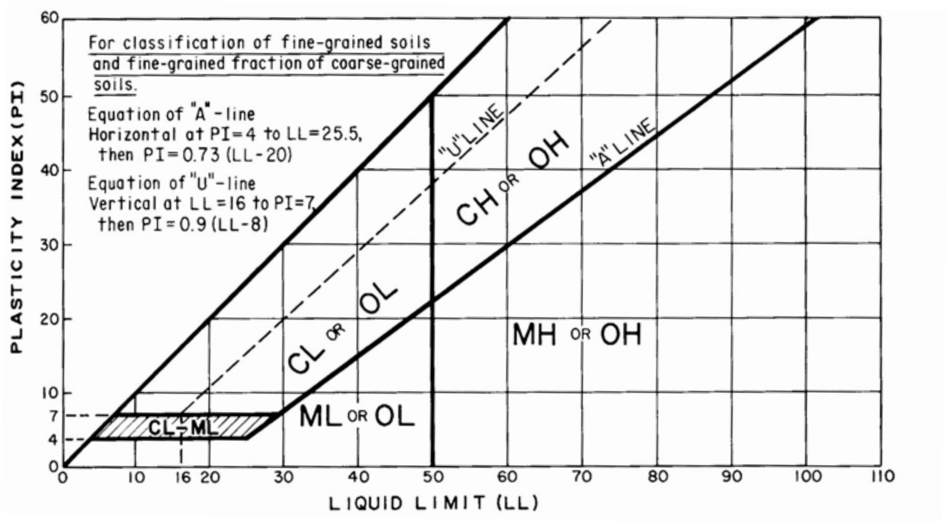

FIGURE 2A.9 Plasticity chart.

Coarse-Grained Soils. Coarse-grained soils are classified as gravel (G) if 50% or more of the coarse fraction is larger than the #4 mesh sieve. The coarse-grained soil is classified as sand (S) if 50% or more of the coarse fraction lies between the #4 and the #200 mesh sieve. Subclasses are well-graded (W), poorly graded (P), silty (M), or clayey (C) based on the amount, distribution, and plasticity of their particles. Details of the classification are given in Table 2A.7.

Fine-Grained Soils. Fine grained soils are classified as either silt (M) or clay (C) depending upon their plasticity as measured by the Atterberg Limits. In order to make this determination, the plasticity chart (Figure 2A.9) is utilized. Once the liquid limit and plasticity indexes are known for the soil, plotting the point on the chart makes the appropriate soil identification. Soils are high plasticity (H) if their liquid limits exceed 50. Soils are low plasticity (L) if their liquid limit is less than 50. The A line separates the clays from the silts and the organic soils. Soils whose liquid limit–plasticity indexes plot above the A line are classified as clays, whereas soils whose liquid limit–plasticity indexes plot below the A line are classified as silts. Organic soils that plot below the A line have visible organic material.

2A.4 WATER FLOW IN SOILS

Many geotechnical engineering problems involve the flow of water through soils. The study of water flow through soils has received much attention due to its importance in seepage and soil consolidation problems. Chemical transport through soils is receiving much more attention due to increased interest in groundwater pollution, waste disposal and storage, and so on.

2A.4.1 Basic Principles of One-Dimensional Fluid Flow in Soils

2A.4.1.1 Darcy's Law

Water flow is related to the corresponding driving forces according to Darcy's law:

$$\text{Water Flow : } q = kiA \tag{2A.23}$$

where q = flow rate (flux) of water

i = hydraulic gradient
k = hydraulic conductivity
A = cross-sectional area of flow

Two general assumptions govern the analysis of fluid flow through soil and rock. The first is that all of the voids are interconnected. The second is that water can flow through even the densest of natural soils. Although the second assumption appears to be valid, it is generally accepted that not all of the void spaces provide passageways for pore fluids.

Darcy's law can be presented as follows for the condition of Figure 2A.10:

$$Q = kiA\Delta t \tag{2A.24}$$

where Q = quantity of flow (volumetric fluid flux, volume units)
i = a forcing function (gradient)
k = fluid conductivity, a coefficient of proportionality related to the properties of the flow medium and the fluid (often called permeability or, more correctly, hydraulic conductivity).
A = cross-sectional area of flow
Δt = time length of flow duration.

Referring to Figure 2A.10, we get

$$q = \frac{Q}{\Delta t} = k\left[\frac{(h_A - h_B)}{L}\right]A = -kiA \tag{2A-25}$$

where h_A = the upstream head = $(h_p + h_z)_A$
h_B = the downstream head = $(h_p + h_z)_B$
h_p = pressure head
h_z = elevation head
L = the length of flow
q = specific fluid flux (flow rate)

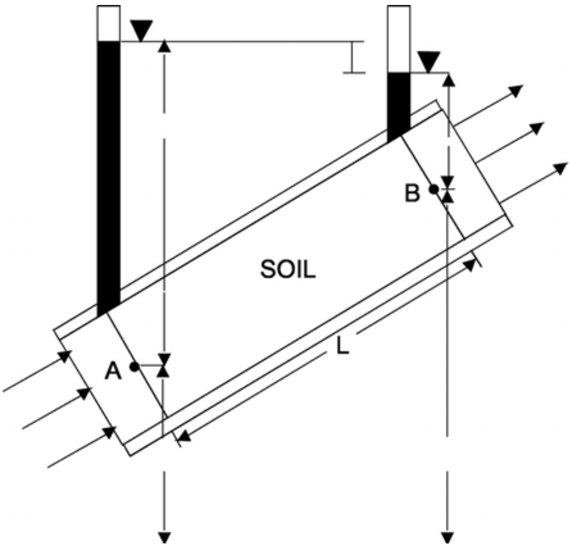

FIGURE 2A.10 Darcy's law.

2A.4.1.2 Flow Velocity

Since flow can only occur through the soil voids,

$$q = VA = v_s A_v \tag{2A.26}$$

where v = approach, apparent or superficial velocity
$= q/A = ki$
v_s = seepage velocity through soil (assumed to be in a straight line)
A = total cross-sectional area of soil volume
A_v = volume of soil voids

$$v_s = v\left[\frac{A}{A_v}\right]\left[\frac{L}{L}\right] = v\left[\frac{V}{V_v}\right] = \frac{v}{n_e} \tag{2A.27}$$

$$v_s = \frac{ki}{n_e} \tag{2A.28}$$

n_e = effective porosity of the soil (pores available for flow)

2A.4.2 Determination of Permeability

2A.4.2.1 Laboratory Methods

It is obvious that the hydraulic conductivity of a soil is, in many cases, the controlling factor in subsurface migration of hazardous wastes. Estimates of contaminant flow quantities and patterns can only be as accurate as the values of hydraulic conductivity used to make them. Subsurface fluid flow may occur under either partially saturated or fully saturated conditions. Numerous methods are available for measurement of hydraulic conductivity of soils either in situ or in the laboratory.

Soils are generally nonhomogeneous and anisotropic. Fine-grained (clay) soils are often stratified, and contain root holes, fissures, and cracks. Therefore, it is desirable to test as large a volume of soil as possible. This usually means that testing should be done in the field. Field methods can usually provide more representative values than laboratory methods because they test a larger volume of material, thus integrating the effects of macrostructure and heterogeneities. However, there are many difficulties in controlling the boundary conditions of field tests.

The most common laboratory methods of measuring the coefficient of hydraulic conductivity in the laboratory are the constant head test and the falling head test.

Constant Head Tests. In the constant head test, the hydraulic gradient, i, is maintained constant at h_w/L (Figure 2A.11). From Darcy's law,

$$k = \frac{q}{iA} = \frac{QL}{(\Delta t h A)} \tag{2A.29}$$

where q = specific fluid flow (flow rate)
Q = flow volume in elapsed time, t
L = Length of soil specimen
h = head of water
A = Cross-sectional area of specimen
Δt = elapsed time of test

The main advantages of the constant head test are the simplicity of interpretation of the data and the fact that the use of a constant head minimizes confusion due to changing volume of air bubbles when the soil is not saturated.

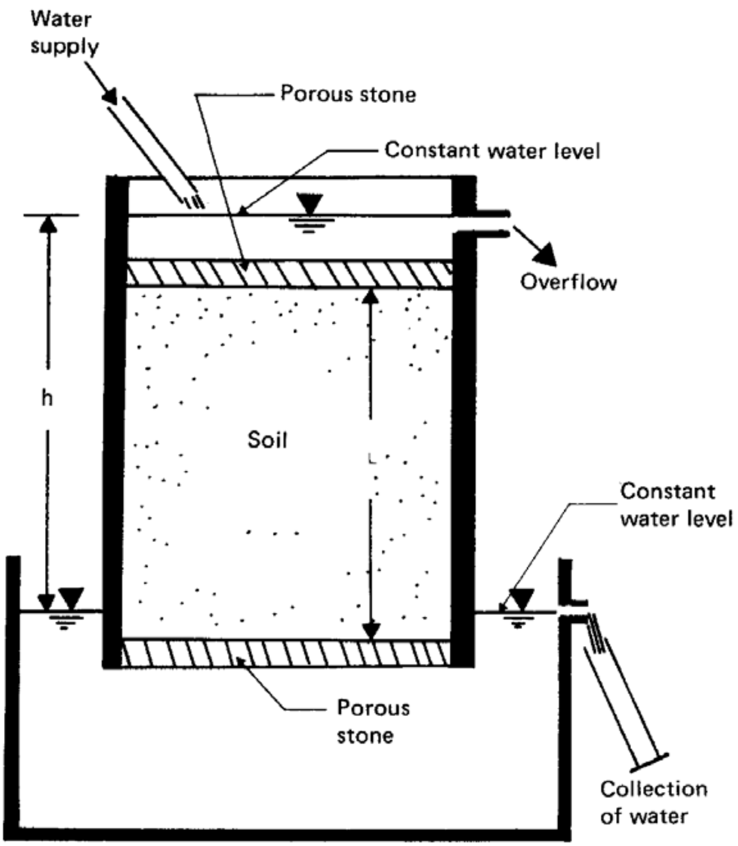

FIGURE 2A.11 Constant head test.

Falling Head Test. In this test,

$$q = a\frac{dh}{dt} \tag{2A.30}$$

where a = the cross sectional area of the burette (Fig. 2A.12). Therefore

$$k = \left[\frac{al}{At}\right]\log_e\left[\frac{h_1}{h_2}\right] \tag{2A.31}$$

The main advantage of this procedure is that small flows are easily measured using the burette. The observation time may be long, in which case corrections for water losses due to evaporation or leakage may be needed.

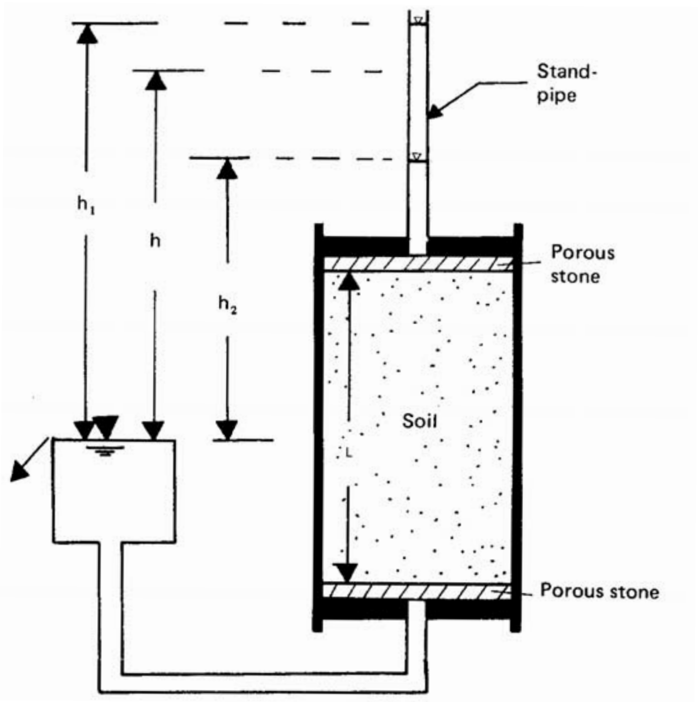

FIGURE 2A.12 Schematic of falling head permeability test.

Triaxial Flexible Wall Permeability Tests. Two techniques are used to confine the soil sample during either the constant or falling head tests: rigid wall confining ring or a flexible membrane.

A flexible wall confined specimen placed in a triaxial permeameter allows us to:[7]

1. Back-pressure saturate
2. Reapply isotropic stresses to simulate field conditions
3. Insure against short-circuiting of permeant
4. Measure independently soil sample volume change

Sources of Error. The following are sources of error in testing:

1. Use of nonrepresentative samples (the overriding source of error)
2. Voids formed during sample preparation
3. Smear zones
4. Alteration in clay chemistry
5. Air in sample
6. Growth of microorganisms
7. Menisci problems in capillary tubes
8. Temperature
9. Volume change due to stress change

2A.4.2.2 Field Methods

The use of field methods for the measurement of hydraulic conductivity had at least two advantages over laboratory techniques: 1) less disturbance of soil, and 2) larger, more representative volume of soil can be tested. The disadvantages of field methods, which are, of course, the advantages of laboratory tests are: 1) less expensive, 2) can saturate the soil fully, 3) can vary the stress, and 4) can test with waste liquids.

The hydraulic conductivity of in situ field soil is usually measured by monitoring the rate of fluid flow into or out of a boring. The coefficient of permeability is then calculated by any one of several available formulas. Heads may vary or be kept constant. Both pump-in and pump-out tests are used.

Auger Method. This method involves the drilling of a hole below the water table, pumping the water in the hole down, and measuring the rate that the water refills the hole (recovery). The coefficient of hydraulic conductivity is calculated using:

$$k = \left[\frac{\pi^2}{16}\right]\left[\frac{r}{Sd}\right]\left[\frac{\Delta h}{\Delta t}\right] \tag{2A.32}$$

where k = hydraulic conductivity
r = radius of the boring
S = Shape factor
d = depth of the bottom of the hole below the water table
h = height of water in the hole
t = time elapsed since the cessation of pumping

This equation is limited to an incompressible soil, a hole drilled to an impervious base, and no drawdown of the water table. Shape factors are given in Fig. 2.13.

Tests Using Cased Holes. In these tests, casings are inserted in the bore holes to the depths where the permeability is to be measured. The coefficient of hydraulic conductivity is calculated by:

$$k = \frac{q}{5.5rh} \tag{2A.33}$$

where r = inside radius of the casing
h = differential head of water
q = rate of supply of water to maintain constant head

Pumping From Wells. The coefficient of hydraulic conductivity can be determined by pumping from a well at a constant rate and observing the steady-state water table in nearby observation wells.

$$q = k\left[\frac{dh}{d}r\right]2\pi rh \tag{2A.34}$$

Solving:

$$k = \left[\frac{(2.303q)}{\pi(h_2^2 - h_1^2)}\right]\log\left[\frac{r_2}{r_1}\right] \tag{2A.35}$$

The equation above has been developed on the assumption that he well fully penetrates the permeable layer.

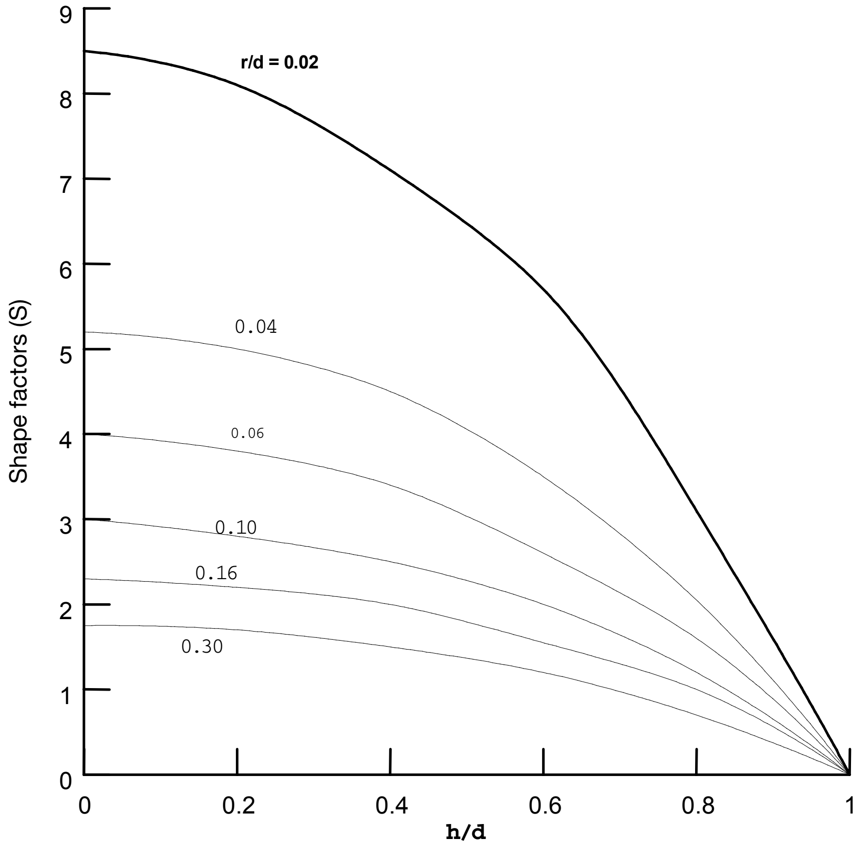

FIGURE 2A.13 Shape factors for auger method.

Tests on Compacted Clay Fills

Sealed Double-Ring Infiltrometer Test. The sealed double-ring infiltrometer (SDRI) is a device designed for making determinations of hydraulic conductivities in the range of 10^{-7} cm/sec to 10^{-10} cm/sec. It is comprised of two concentric rings. To facilitate installation, the ring shape is rectangular rather than circular. Seepage interior to the inner ring is used for the determination of hydraulic conductivity. Water in the annular space between the inner and outer rings prevents lateral seepage from the soil column underlying the inner ring. Although a uniform hydraulic head is maintained over the entire test area, the inner ring is sealed to eliminate effects of evaporation. Fig. 2A.14 shows a schematic of the SDRI installation.

Tensiometers are used to track the wetting front during the test. A tensiometer is a tube with a porous cup at one end and a vacuum gage at the other. First, a hole is augered to the desired depth in the material to be monitored. After being filled with water and ensuring that the porous tip is saturated, the tensiometer is placed in the auger hole. The annular space is then back filled in compacted layers with the soil originally removed from the hole. The tensiometer is then allowed to come into equilibrium with the surrounding soil. Because the surrounding soil is unsaturated and the porous cup is in hydraulic contact with the soil, a vacuum develops in the tensiometer and can be read by the gage

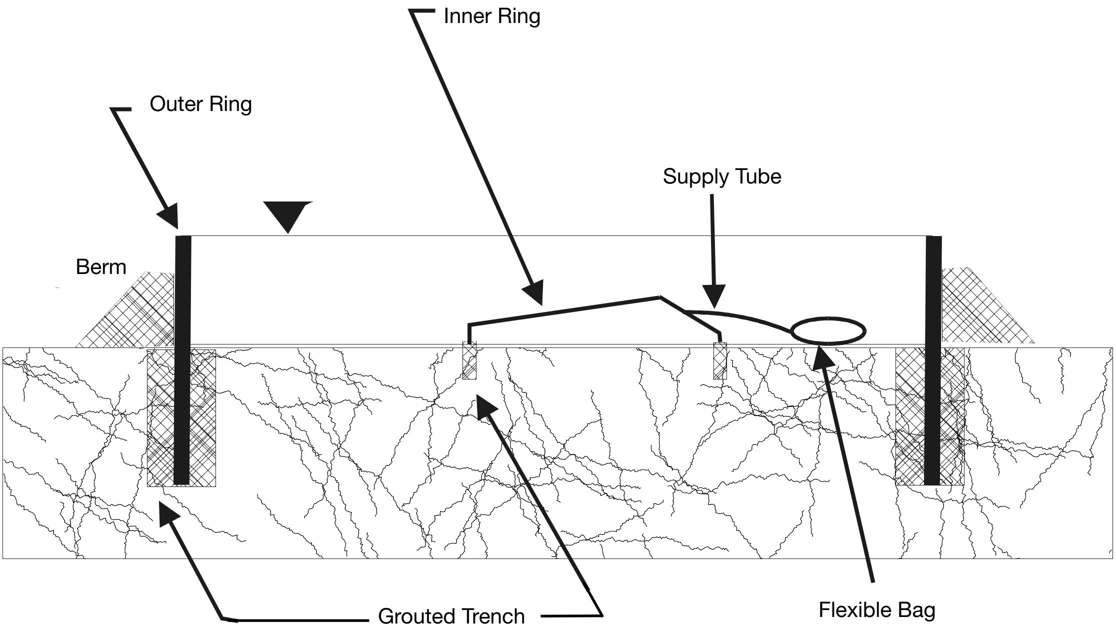

FIGURE 2A.14 Schematic of sealed double ring infiltrometer.

at the top. As the wetting front passes, the pressure will go to zero. Typically, nine tensiometers are installed in the soil in the annular space between the inner and outer rings—three each at depths of 15 cm, 30 cm, and 45 cm. The time at which the wetting front passes each tensiometer is recorded.

Prior to commencing the test, water is placed into the inner ring. The outer ring is filled to a depth of about 280 mm (at least 100 mm over the apex of the dome of the inner ring). This depth is maintained within 25 mm throughout the test and a record of the volume of water added is maintained. A clear flexible bag with a capacity of 1000 ml to 3000 ml (such as an intravenous bag available from medical supply houses) is used to measure the volumetric rate of seepage from the inner ring. It is filled with water, weighed, and attached to the inlet tube. Periodically during the test, the bag is removed and weighed. The record of bag weight change with time is used to compute the infiltration rate.

The test is complete when the infiltration rate becomes constant or when it becomes less than some predetermined value. At this time, the tubing is disconnected from the inner ring, water is drained from the rings, and grout is removed from around the rings to allow the rings to be extracted. The soil in the test area circumscribed by the inner ring is then sampled for a soil moisture profile determination.

The actual determination of hydraulic conductivity entails straightforward application of Darcy's Law. Vertical percolation rate (seepage velocity) can be calculated as the volumetric flow rate divided by the area of exposed soil surface circumscribed by the inner ring. Volumetric flow rate is the mass change over a known period of time, as determined from weight measurements in the bag divided by the density of water:

$$q = \frac{\Delta M}{\Delta t} \cdot \frac{1}{\rho} \tag{2A.36}$$

where q = volumetric flow rate (cm^3/sec)
ΔM = change in mass (gm)
Δt = incremental time over which ΔM is observed (sec)
ρ = density of water (gm/cm^3)

It then follows that the seepage velocity can be computed as

$$v_s = \frac{Q}{D_i^2} \tag{2A.37}$$

where v_s = seepage velocity (cm/sec)
D_i = length of side of inner ring (cm)

The hydraulic gradient is

$$i = \frac{H + L_f h_{we}}{L_f} \tag{2A.38}$$

where H = depth of ponding in the SDRI (cm)
L_f = distance from wetting front to the soil surface (cm)
h_{we} = water-entry suction head (cm), a negative number

There is no universally accepted way of determining h_{we} under field conditions. Therefore, it is usually assumed to be zero.

By rearranging the common expression for Darcy's Law, hydraulic conductivity, k, can be calculated as

$$k = \frac{v_s}{i} = \frac{v_s}{\left(\dfrac{H + L_f}{L_f}\right)} \tag{2A.39}$$

where k = hydraulic conductivity (cm/sec)

Boutwell Two-Stage Borehole Test. Boutwell developed a two-stage hydraulic conductivity test. The apparatus is shown schematically in Fig. 2A.15. The device is installed by drilling a hole to depth Z. The depth Z must be at least five times larger than D to avoid ambiguities in the interpretation of test results. Furthermore, the depth from the base of the borehole to the bottom of the liner should never be less than about $5D$ for the same reason. After the hole is drilled, a casing is placed inside the hole and the annular space between the casing and borehole is sealed with grout. A cap is placed on the permeameter and a reservoir is used to fill the casing and the standpipe. Once the permeameter has been assembled and filled with water, the Stage I tests are started. The elevation of zero porewater pressure in the soil is assumed to be at the base of the casing such that the head driving the flow is H, as shown in Fig. 2A.15.

A series of falling-head tests are performed, and the hydraulic conductivity from Stage I (k_1) is computed as follows:

$$k_1 = \frac{\pi d^2}{11 D \Delta t} \ln\left(\frac{H_1}{H_2}\right) \tag{2A.40}$$

The values of k_1 are plotted as a function of time until steady conditions are reached. This typically takes from a few days to as much as 2 to 3 weeks. The steady-state value of k_1 is used for later calculations. Next, the top of the permeameter is removed and the hole is deepened with an auger or by pushing in a thin-walled sampling tube. The ratio of length to diameter (L/D) in the uncased zone (Fig. 2A.15) should be between 1.0 and 1.5. The permeameter is reassembled and a series of falling-

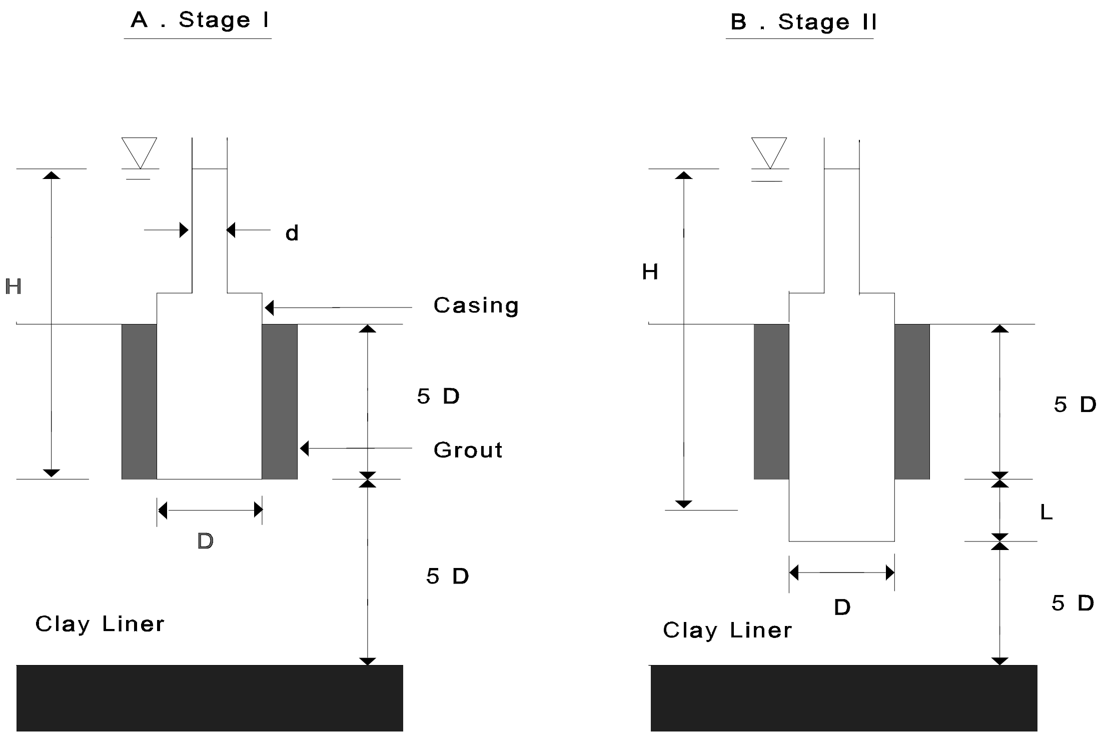

FIGURE 2A.15 Boutwell two-stage hydraulic conductivity test.

head tests are again performed. The head loss (H) is assumed to be as shown in Fig. 2A.15. The hydraulic conductivity from Stage II (k_2) is calculated as follows:

$$k_2 = \left[\frac{A}{B}\right] \ln\left(\frac{H_1}{H_2}\right) \tag{2A.41}$$

where

$$A = d^2 \left\{ \ln\left[\left(\frac{L}{D}\right) + \left(1 + \left(\frac{L}{D}\right)^2\right)\right]^{0.5} \right\} \tag{2A.42}$$

$$B = 8D\left(\frac{L}{D}\right)(dt)\left\{1.0 - 0.562 \exp\left[-1.57\left(\frac{L}{D}\right)\right]\right\} \tag{2A.43}$$

The values of k_2 are plotted as a function of time until k_2 ceases to change significantly. Next, arbitrary values of m are selected, where m is defined as

$$m = \left(\frac{k_h}{k_v}\right)^{0.5} \tag{2A.44}$$

k_h and k_v are the hydraulic conductivities in the horizontal and vertical directions, respectively. The corresponding values of k_2/k_1 are calculated from the expression

$$\left(\frac{k_2}{k_1}\right) = \frac{m \ln\left[\left(\frac{L}{D}\right) + \left(1 + \left(\frac{L}{D}\right)^2\right)\right]^{0.5}}{\ln\left[\left(\frac{mL}{D}\right) + \left(1 + \left(\frac{mL}{D}\right)^2\right)^{0.5}\right]} \tag{2A.45}$$

Typically, values of m ranging from 1 to as much as 10 might be used to compute k_2/k_1. The resulting data are plotted as shown in Fig. 2A.16 for $L/D = 1.0$ and $L/D = 1.5$, and a smooth curve is fitted visually through the data for each L/D ratio. The plot of k_2/k_1 versus m is then entered with the actual value of k_2/k_1 as determined from equations 2A.44 and 2A.45 on the basis of data collected during Stage I and Stage II tests. The actual value of m that corresponds to the actual value of k_2/k_1 is found. The hydraulic conductivities in the vertical and horizontal directions are computed as follows:

$$k_h = mk_1 \tag{2A.46}$$

$$k_v = \left(\frac{1}{m}\right)k_1 \tag{2A.47}$$

The assumptions that were made are that the soil is homogeneous, pore water pressure in the surrounding soil is zero at the base of the permeameter, the soil that is permeated is sufficiently far removed from the boundaries of the liner that the test results are unaffected by boundary conditions, the degree of saturation of soil through which the water flows is uniform, and that soil suction is negligible.

The advantages of borehole tests are that the devices are relatively easy to install, they can be installed at great depth, the cost is relatively low, the hydraulic conductivity in both the vertical and horizontal directions can be measured, and relatively low hydraulic conductivities (10^{-9} cm/sec) can be measured. The disadvantages are that the effects of incomplete and variable saturation are un-

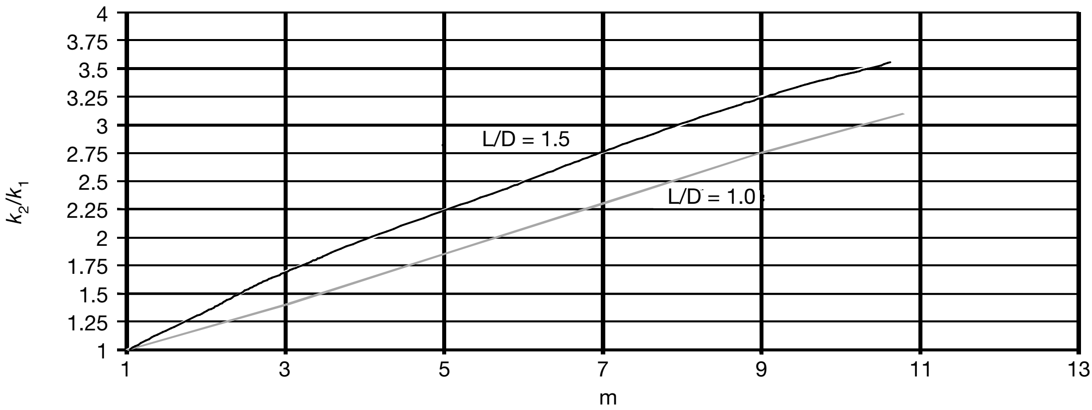

FIGURE 2A.16 k_2/k_1 versus m.

known, the influence of soil suction upon the results is ill defined, the test cannot be used near the top or bottom of a liner, and the volume of soil that is permeated is relatively small.

2A.5 GEOSTATIC STRESSES IN SOIL

The three phases of soil (solid, water, and gas) will react differently to applied stresses and thus a relationship between the phases must be established. The solid particles and water are relatively incompressible and the gaseous phase is highly compressible. The following definitions and relationship between the various phases have been proposed by Terzaghi.

2A.5.1 Total Stresses

Total stress, σ, is the stress acting at a point in a soil mass with a horizontal top surface. The total stress is computed as the total weight of a column of unit area above the point, i.e.,

$$\sigma = \gamma z \tag{2A.48}$$

2A.5.2 Pore Stresses

The pore or neutral stress (u_w) is the stress within the water voids. Since this stress is hydrostatic, it acts equally in all directions. Under no flow conditions (static),

$$u_w = \gamma_w h_w \tag{2A.49}$$

where γ_w = the unit weight of water (9.8 kN/M^3 = 62.4 lbs/ft^3)
h_w = head of water

2A.5.3 Effective Stresses

The intergranular force acting between points of contact of the solid constituents per unit area is termed the effective stress ($\overline{\sigma}$). Effective stress cannot be measured but can be calculated from the general relationship for saturated soils:

$$\overline{\sigma} = \sigma - u \tag{2A.50}$$

Effective stress, not total stress, governs the shear and compressibility behavior of soils.

2A.6 DISTRIBUTION OF APPLIED STRESSES IN SOIL

The stress in a soil mass due to an applied load can most easily be computed from elastic theory. The soil mass is generally assumed to be semi-infinite, homogeneous, and isotropic.

2A.6.1 Point Load

Boussinesq[8] published a relationship for the stress at any point with coordinates (x,y,z) beneath the location of a point load on the surface of the mass in 1885. His equation was:

$$\sigma_z = \frac{3P}{2\pi} \frac{z^3}{(x^2 + y^2 + z^2)^{5/2}}$$

if

$$x^2 + y^2 + z^2 = R^2 \tag{2A.51}$$

$$\sigma_z = \frac{3P}{2\pi} \frac{z^3}{R^5}$$

If $r^2 = x^2 + y^2$

$$\sigma_z = \frac{P}{z^2} \frac{3}{2\pi} \left[\frac{1}{\left(\frac{r}{z}\right)^2 + 1} \right]^{5/2}$$

or

$$\sigma_z = \frac{P}{z^2} I \tag{2A.52}$$

where I = influence factor = $f(r/z)$

If equation 2A.52 is plotted versus depth z for $r = 0$ (center line), Fig. 2A.17 results. As can be seen, the stress decreases rapidly as the depth beneath the point of load application increases. If

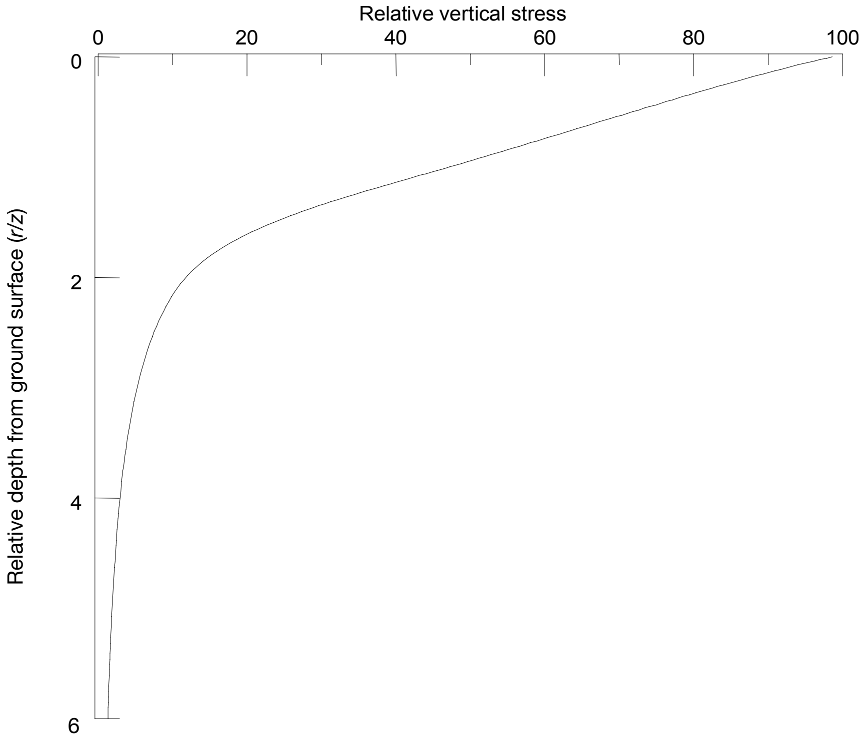

FIGURE 2A.17 Vertical stress versus depth.

equation 2A.52 is plotted versus r for a constant depth z, Fig. 2A.18 results. The stress decreases rapidly as the distance from the axis of the load increases.

2A.6.2 Uniformly Loaded Strip

Equation 2A.52 can be used to determine the stress beneath a flexible strip load of width B (Fig. 2A.19). Using the terms defined in Fig. 2A.19, the following equation can be developed:

$$\sigma_v = \frac{q}{\pi}[\beta + \sin\beta\cos(\beta + 2\sigma)] \tag{2A.53}$$

where $q = P/A$ = load/unit area

Table 2A.8 shows the variation of σ/q with $2z/B$ for various values of $2x/B$.

2A.6.3 Uniformly Loaded Circular Area

The Boussinesq point load equation can be used to develop an expression for the vertical stress below the center of a uniformly loaded flexible circular area:

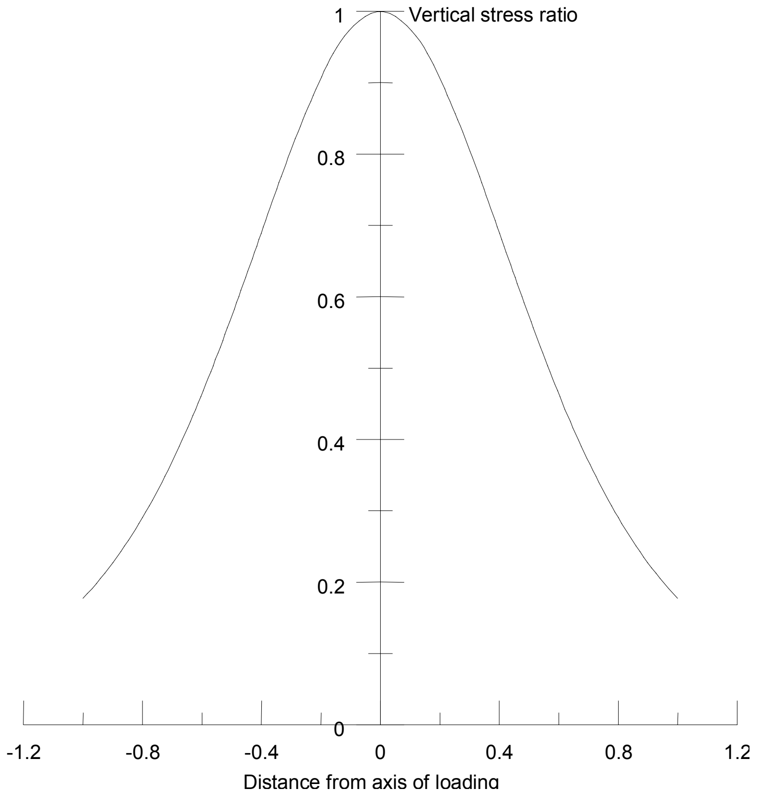

FIGURE 2A.18 Vertical stress versus r.

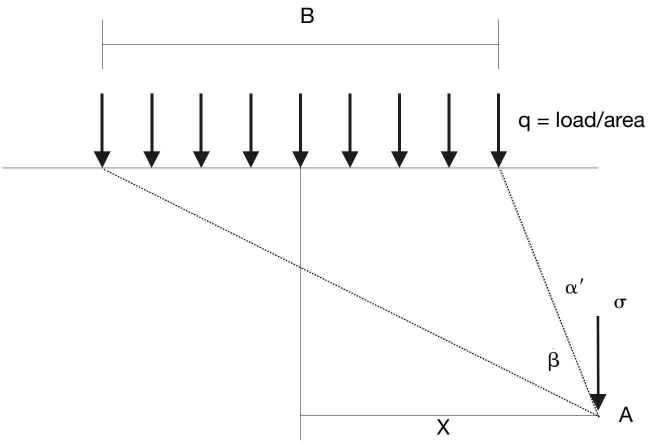

FIGURE 2A.19 Vertical stress due to a flexible strip load.

TABLE 2A.9 Variation of σ/q with $2z/B$ and $2x/B$

$2x/B$	$2z/B$	σ/q	$2x/B$	$2z/B$	σ/q
0	0.0	1.0000	1.5	1.0	0.2488
	0.5	0.9594		1.5	0.2704
	1.0	0.8183		2.0	0.2876
	1.5	0.6678		2.5	0.2851
	2.0	0.5508	2.0	0.25	0.0027
	2.5	0.4617		0.5	0.0194
	3.0	0.3954		1.0	0.0776
	3.5	0.3457		1.5	0.1458
	4.0	0.3050		2.0	0.1847
0.5	0.0	1.0000		2.5	0.2045
	0.25	0.9787	2.5	0.5	0.0068
	0.5	0.9028		1.0	0.0357
	1.0	0.7352		1.5	0.0771
	1.5	0.6078		2.0	0.1139
	2.0	0.5107		2.5	0.1409
	2.5	0.4372	3.0	0.5	0.0026
1.0	0.25	0.4996		1.0	0.0171
	0.5	0.4969		1.5	0.0427
	1.0	0.4797		2.0	0.0705
	1.5	0.4480		2.5	0.0952
	2.0	0.4095		3.0	0.1139
	2.5	0.3701			
1.5	0.25	0.0177			
	0.5	0.0892			

$$\sigma_v = q\left[1 - \frac{1}{\left[\left(\frac{R}{z}\right)^2 + 1\right]}\right]^{3/2} = qI \tag{2A.54}$$

where R = radius of loaded area
I = influence factor

Table 2A.9 gives I for various values of z/R.

2A.6.4 Uniformly Loaded Rectangular Area

The vertical stress below a corner of a flexible rectangular loaded of width B and length L can be computed from

$$\sigma_v = qI_3 \tag{2A.55}$$

where

$$I_3 = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right)\right] + \frac{1}{4\pi}\left[\tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right]$$

$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$

The variation of I_3 is shown in Fig. 2A.20. The special case of the stress beneath the center of a square loaded area with side B is given in Table 2A.10.

TABLE 2A.9 I vs z/R for circular loaded area

z/R	I
0.0	1.0
0.02	0.9999
0.05	0.9998
0.10	0.9990
0.2	0.9925
0.4	0.9488
0.5	0.9106
0.8	0.7562
1.0	0.6765
1.5	0.4240
2.0	0.2845
2.5	0.1996
3.0	0.1436
4.0	0.0869
5.0	0.0571

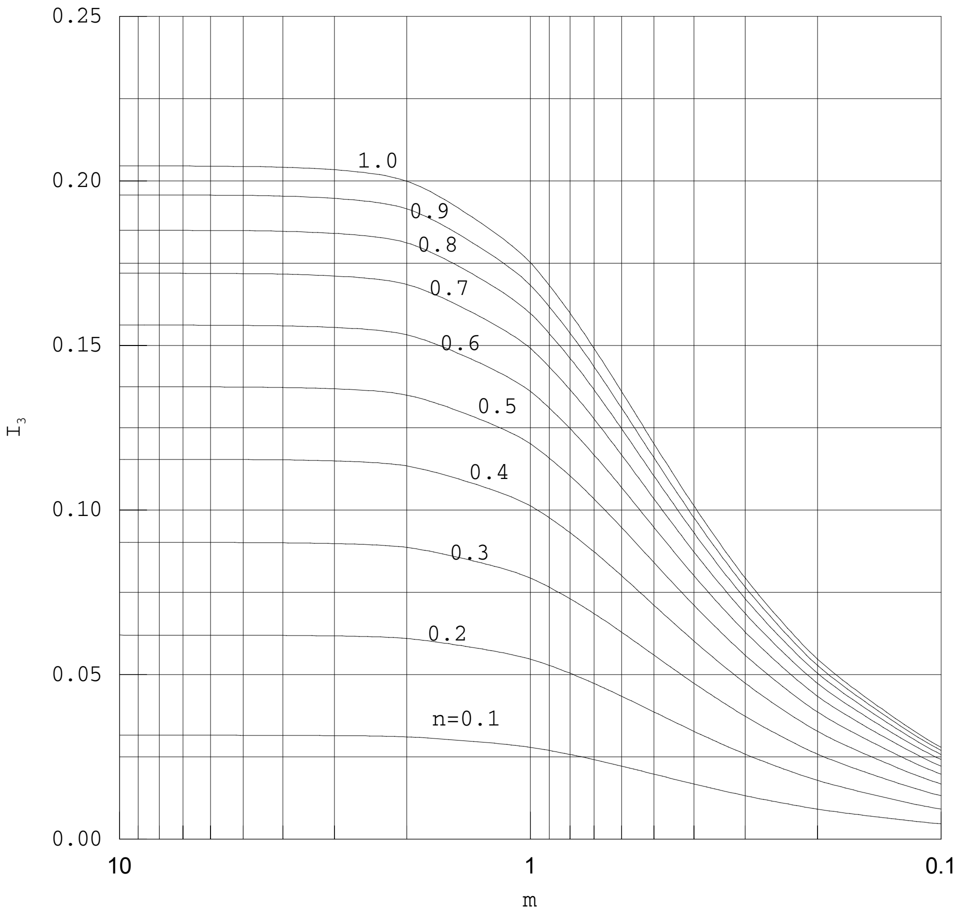

FIGURE 2A.20 Variation of I_3 with m and n.

TABLE 2A.10 Influence factors for stress beneath the center of a square load

B/z	I	B/z	I
0.0	1.0000	2.4	0.7832
20.0	0.9992	2.0	0.7008
16.0	0.9984	1.8	0.6476
12.0	0.9968	1.6	0.5844
10.0	0.9944	1.4	0.5108
8.0	0.9892	1.2	0.4276
6.0	0.9756	1.0	0.3360
5.0	0.9604	0.8	0.2410
4.0	0.9300	0.6	0.1494
3.6	0.9096	0.4	0.0716
3.2	0.8812	0.2	0.0188
2.8	0.8408	0.0	0.0000

2A.6.4 Average Stress Influence Chart

The point load equation has been used to compute average stresses beneath a of circular, square and rectangular uniformly loaded footings. The results are presented as Fig. 2A.21. To use this chart enter the chart with the appropriate depth ratio (z/B) with B being the least dimension. Using the curve for the appropriate shape, read the influence factor on the abscissa. The average stress is the product of the influence factor times the applied stress.

2A.7 ONE-DIMENSIONAL CONSOLIDATION

Assume that the soil stratum shown in Fig. 2A.22 has been formed by sedimentation. If we assume that the element of soil is in equilibrium with its overburden, then the effective overburden

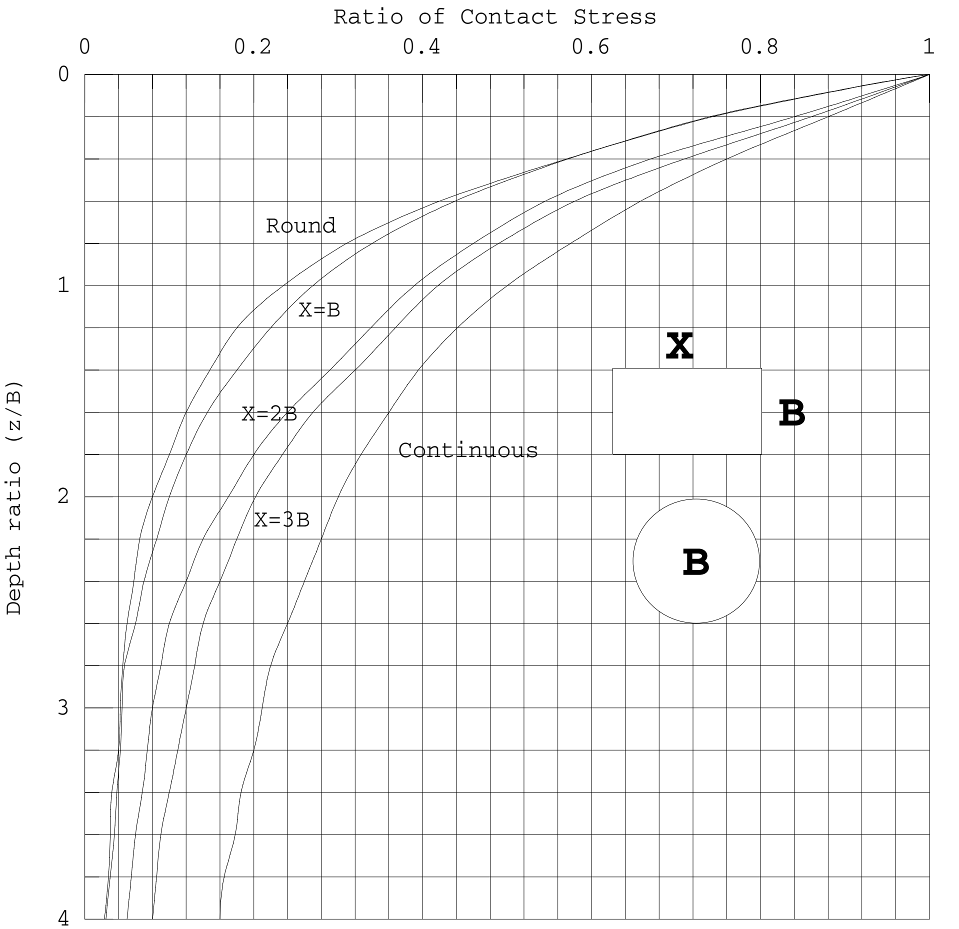

FIGURE 2A.21 Average stress beneath a loaded area.

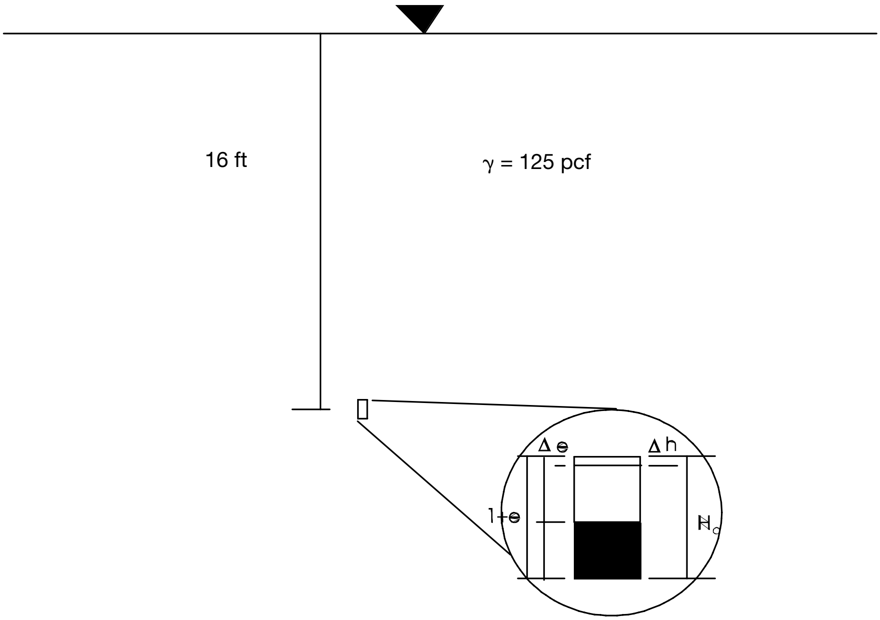

FIGURE 2A.22 Consolidation of a soil element.

$$\frac{\Delta e}{1+e} = \frac{\Delta h}{H_0}$$

$$\Delta h = \frac{H_0 \Delta e}{1+e} \tag{2A.56}$$

stress on that soil element is $\overline{\sigma} = \gamma z = (125 - 62.4)16 = 1000$ psf. In this state, the soil has a void ratio of e ($wG_s = 1.17$) and an element height of H_0. If a very wide fill is placed on top of the soil strata, the stress on the soil element is increased by an amount Δp ($\gamma_{\text{fill}} \cdot h_{\text{fill}}$). This increase in stress causes the void ratio (and the height) of the soil to decrease (settle), as you can see from the figure.

Therefore, if we can determine the relationship between the change in void ratio (Δe) and the applied stress, we can compute the settlement of the soil. This relationship is usually determined from a laboratory one-dimensional consolidation test.

2A.7.1 One-Dimensional Laboratory Consolidation Test

The one-dimensional consolidation test is performed using a disc-shaped sample of soil that is confined around its periphery by a rigid, impervious ring. The specimen is loaded on its flat surfaces through porous stones that allow the water from the soil voids to escape but restrain the soil particles from moving. All deformation occurs parallel to the axis of the specimen (Fig. 2A.23)

2A.7.1.1 Performance of a Consolidation Test

The procedures used in performing one-dimensional consolidation tests vary widely, depending on such factors as soil type, sampling method, and nature of the problem in the field (foundation of

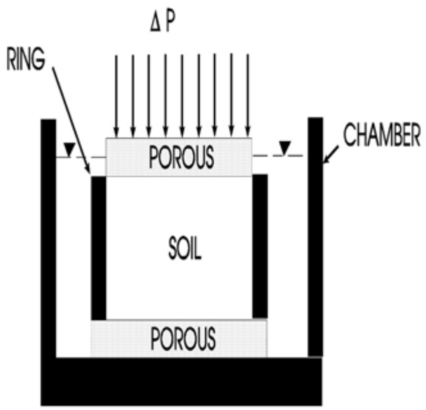

FIGURE 2A.23 Fixed ring consolidometer.

building, embankment, etc.). The procedures recommended in ASTM D2435 are recommended for normal situations.

The soil sample is removed from the sampling tube in such a manner as to minimize further disturbance. A suitable length of soil is removed and is carefully trimmed into the ring. The cell is placed in the loading frame and a seating load is applied, the size of which depends on the strength of the soil. The dial indicator used to measure the axial deformation of the soil is mounted in the frame and an initial reading is taken. The consolidation cell is then filled with water. The applied pressure is adjusted continuously until the soil comes to equilibrium with the pressure at constant volume.

The first increment of consolidation pressure is then applied and the soil begins to consolidate. A series of readings of axial deformation are made at preselected times. The consolidation pressure is maintained constant until consolidation has essentially ceased. The standard load increment duration for each pressure is 24 hours (ASTM D2435 Method A), although shorter or longer times may be used depending on the coefficient of consolidation of the soil and the thickness of the sample. Sufficient time-deformation readings are taken to insure that consolidation is complete. For some soils, a period of more than 24 hours may be required to reach end-of-primary consolidation. Usually, the load increment duration is some multiple of 24 hours and should be the standard duration for *all load intervals*.

When consolidation is essentially complete under the first pressure, the next increment of pressure is applied and the deformation reading taken again. The process is repeated until some preselected maximum consolidation pressure is attained. It is usual practice to double the pressure for each successive increment. Thus, for a typical consolidation test, the sequence of pressures might be 125, 250, 500, 1000, 2000, 4000, 8000, 16000, 32000, and 64000 psf. The highest pressure is controlled by the capacity of the frame, economics, and time limitations, or a variety of other factors.

After the deformation has ceased under the highest load, the loads are removed in a series of decrements such that the pressure is usually reduced by four times for each decrement. The final pressure is usually of the order of 125 psf. When the specimen has equilibrated under the final pressure, the apparatus is dismantled rapidly to prevent the soil from imbibing a significant amount of water after the final pressure has been removed.

2A.7.1.2 Calculation of the Laboratory Consolidation Curve

The results of the consolidation test are normally presented as plots of void ratio, e, versus the logarithm of applied pressure, $\bar{p}$. A plot of the pressure-void ratio curve from the test is shown in Fig.

2A.24. As you can see, the curve is highly nonlinear. If that same data is plotted on a semilogarithmic plot e vs $\log_{10} p$, Fig. 2A.25 results. The e–log p plot has an initially shallow slope that transitions into a steeper slope.

As you can see from the plot, the deformations are relatively small until the load approaches the in situ ($\overline{p}_0$) pressure. When the load exceeds the in situ pressure, the deformation increases dramatically. The pressure where the e–log p curve increases slope is called the *maximum past consolidation pressure* (*preconsolidation pressure,* $\overline{p}_c$) and is the greatest stress that has ever been on the soil. For the case studied here, $\overline{p}_c = 1000$ psf $= \overline{p}_0$). The portion of the e–log p curve from the first load to $\overline{p}_c$ is called the *reload* or *recompression* curve, since the soil is being reloaded to its maximum past value. The portion of the e–log p curve for loads greater than $\overline{p}_c$ is called the *virgin consolidation* curve because each additional load is greater than any load that has ever been on the soil previously. Soils where $\overline{p}_c \approx \overline{p}_0$ are called *normally consolidated soils*.

In some (in fact, in most) cases, $\overline{p}_c \ll \overline{p}_0$. This implies that the soil has been more heavily loaded in its past than it is now. Soils with this stress history are called *overconsolidated soils*. Compared to normally consolidated soils, overconsolidated soils are stiffer and less compressible. As before, that portion of the e–log p curve to the $\overline{p}_c$ is the reloading branch, whereas that portion beyond $\overline{p}_c$ is the virgin consolidation branch.

2A.7.1.3 Reconstruction of the Field Consolidation Curve

The consolidation curve shown in Fig. 2A.25 is assumed to be typical of field consolidation curves. The recompression curves merge smoothly with the virgin consolidation curve and have a region of

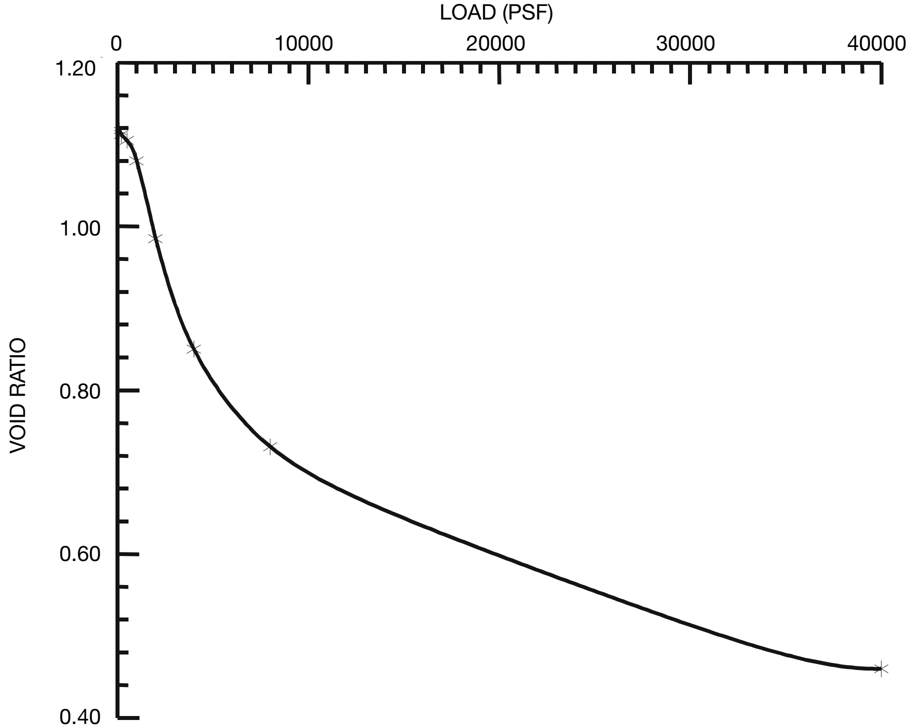

FIGURE 2A.24 *e–p* plot.

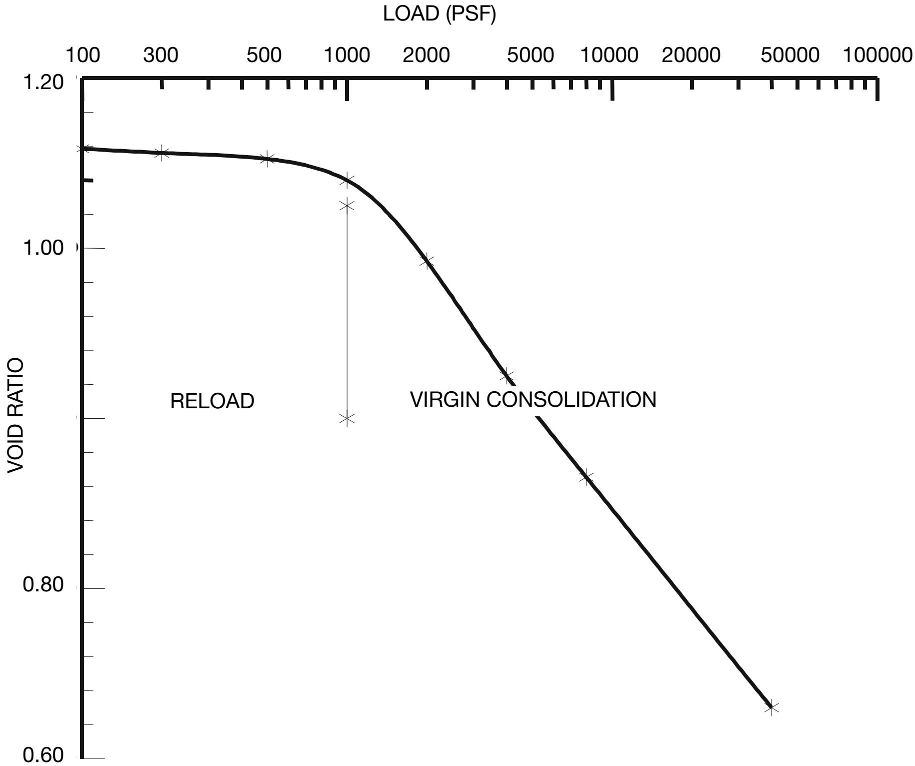

FIGURE 2A.25 *e*–log *p*.

sharpest curvature in the vicinity of the maximum previous consolidation pressure, $\bar{p}_c$. Disturbance of soil samples during sampling, transportation, storage, and trimming causes the laboratory curve to be displaced to lower void ratios and to have less pronounced curvature in the vicinity of $\bar{p}_c$.[9,10] A procedure is needed for reconstruction of the field consolidation curve from a slightly disturbed laboratory curve. No exact method for reconstruction of the field curve exists. The available methods are based on laboratory tests and field experience.

The first step is to obtain an estimate of the maximum previous stress under which the soil was consolidated, $\bar{p}_c$. Although several procedures have been proposed, the one suggested by Casagrande[9] seems to be the most satisfactory and most widely used. The procedure is shown in Fig. 2A.26. At the point of sharpest curvature of the laboratory curve, two lines are drawn, one tangent to the laboratory curve and the other horizontal. The angle between these lines is bisected by a third line. The intersection of the bisecting line and the extended laboratory virgin consolidation curve is taken as an approximation of the maximum previous consolidation pressure. If this pressure is approximately equal to the calculated effective overburden pressure in the field, $\bar{p}_0$, then the soil is assumed to be normally consolidated. One point on the field curve is then e_0, $\bar{p}_c$, where e_0 is the initial void ratio of the sample. The field consolidation curve is drawn to pass through the point e_0, $\bar{p}_c$, and to be asymptotic to the laboratory curve at high pressures (Fig. 2A.27).

If the samples used in the laboratory are normally consolidated and have a maximum of disturbance, the e_0, $\bar{p}_0$ point is so near the backwards extension of the laboratory virgin curve that the field

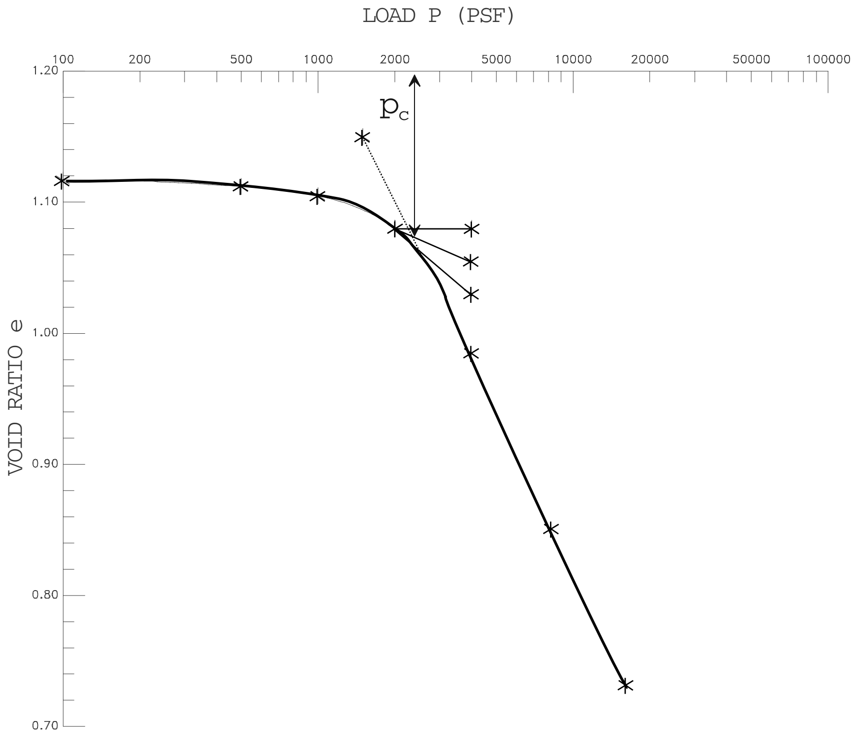

FIGURE 2A.26 Casagrande construction for preconsolidation pressure.

curve can be drawn without use of Casagrande's construction. If the soil is badly disturbed, the laboratory consolidation curve will not have an obvious point of sharpest curvature and no known construction, will give a reasonable approximation of the field curve. The construction, then, is of greatest value for slightly disturbed or overconsolidated specimens.

If Casagrande's construction indicates that p_c exceeds p_0 by a significant amount, then the soil is overconsolidated. The reconstruction of the field curve then is based on the procedure recommended by Schmertmann (1955). A laboratory curve for a sample of highly plastic, overconsolidated, clay is shown in Fig. 2A.27. Casagrande's construction is used to estimate the maximum previous consolidation pressure, the effective overburden pressure in the field is calculated, and the point e_0, $\overline{p}_0$ is plotted. For the moment it will be assumed that the soil is simply overconsolidated, i.e., that it was consolidated to some maximum effective stress in the field and then rebounded directly to the point e_0, $\overline{p}_0$. Furthermore, the field rebound curve is assumed to be parallel to the laboratory rebound curve. Thus, a curve may be drawn parallel to the laboratory rebound curve back from a point on the $\overline{p}_c$ line such that the rebound curve passes through the e_0, $\overline{p}_0$ point. The point on the $\overline{p}_c$ pressure line is then one point on the field virgin consolidation curve. The actual field curve must pass through the point e_0, $\overline{p}_0$, must pass through the pressure $\overline{p}_c$ lower than the "known" point on the field virgin curve, but must remain above the laboratory curve. The reconstructed field curve is drawn in by eye and merged gradually with the laboratory virgin curve (Fig. 2A.27). Schmertmann[11] suggested certain refinements to the method just suggested.

A number of problems arise when attempts are made to apply the foregoing method. First, for many highly overconsolidated soils, the reloading curve of even hand-carved samples appears to be

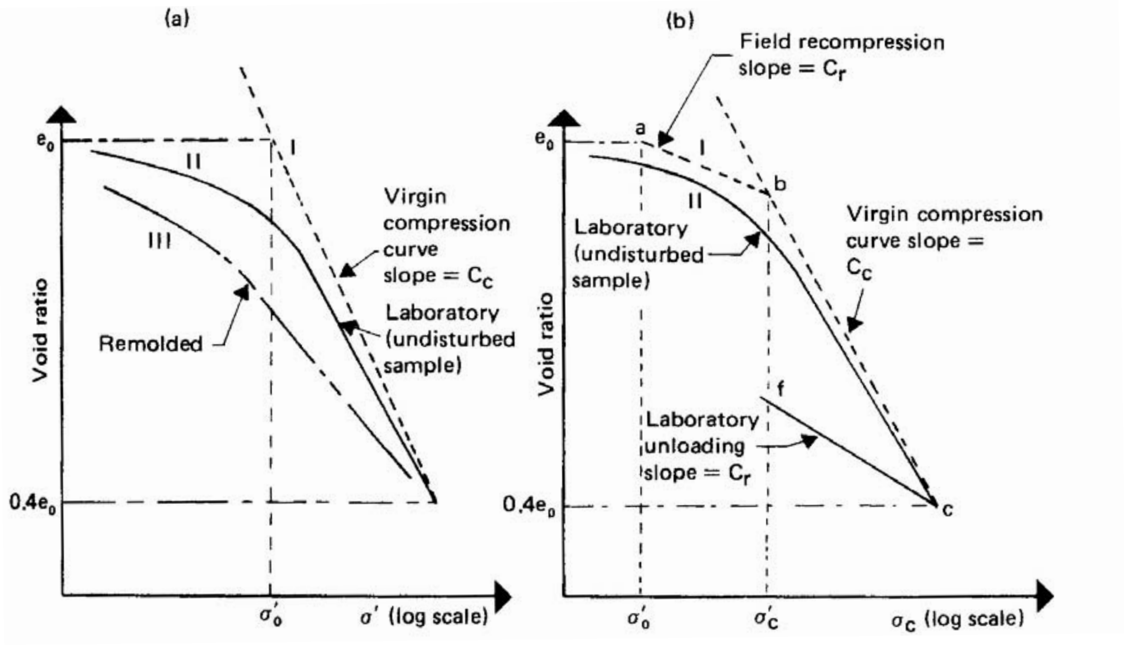

FIGURE 2A.27 Construction of field consolidation curve.

a continuous smooth curve and there is no apparent method for estimating the maximum previous consolidation pressure. Such curves are common for clayey glacial tills. It is also possible that the soil has been rebounded to a pressure much less than the existing overburden pressure and is now on a reloading, rather than rebounding, curve.

Based on the foregoing comments, it seems apparent that reconstruction of the field curve is based largely on the judgment of the soils engineer aided by certain constructions. The uncertainty in estimating the position of the field curve will be reduced if relatively undisturbed samples are used.

2A.7.1.4 Calculation of the Coefficient of Consolidation

The time required for consolidation of a soil can be computed from

$$T = \frac{c_v t}{H_d^2} \tag{2A.57}$$

where T = time Factor (see Table 2A.11)
H_d = maximum drainage distance

For the case of a soil layer loaded very quickly with drainage allowed on both sides, the time–settlement curve can be calculated, provided that the ultimate settlement, S_u, and the coefficient of consolidation, c_v, are known. The ultimate settlement can be calculated from the reconstructed field consolidation curve using the methods discussed previously. The coefficient of consolidation for use in field analyses is also usually estimated from laboratory consolidation tests.

In the laboratory, there is a nearly instantaneous initial settlement, which may be caused by elastic compression of the experimental apparatus, seating of the porous stones against improperly trimmed faces of the soil specimen, or compression of gas bubbles in the soil. This rapid settlement, termed *initial compression*, obviously cannot be taken into account by the theory. Thus, an adjustment must be made to the laboratory curve to remove the effects of initial compression.

There are two procedures in common use for estimating the appropriate values of S_0 and S_{100}. They are designated Taylor's method[12] and Casagrande's method.[13]

2A.7.1.4.1 Taylor's Method of Finding c_v. When Taylor's method is used, the settlement is plotted versus the square root of time. A square root versus time curve from a one-dimensional consolidation test is shown in Fig. 2A.28.

The corrected initial point for the theoretical curve is found just by extending the linear portion of the laboratory square root of time curve (Fig. 2A.28) back to time zero.

Taylor found that there was no distinct change in the square root of time curve to show where primary and secondary compression merged. In an attempt to find S_{100}, Taylor made the assumption that secondary effects could be ignored of U less than or equal to 90%. Further, he noted that at 90% ultimate consolidation ($U = 90\%$), the abscissa of the laboratory curve would be

$$F\sqrt{t_{90}} = F\left[\frac{H^2}{c_v}\right]^{0.5}(T_{90})^{0.5} \qquad \text{(2A.58)}$$

where F is just the scale factor originally used to lay out the time scaler on the graph paper. If the linear portion of the square root of time laboratory curve is extended as a straight line to higher values of U, at $U - 90\%$ the abscissa of this straight line is:

$$\left(\frac{9}{5}\right)F\left(\frac{H^2}{c_v}\right)^{0.5}(T_{50})^{0.5} \qquad \text{(2A.59)}$$

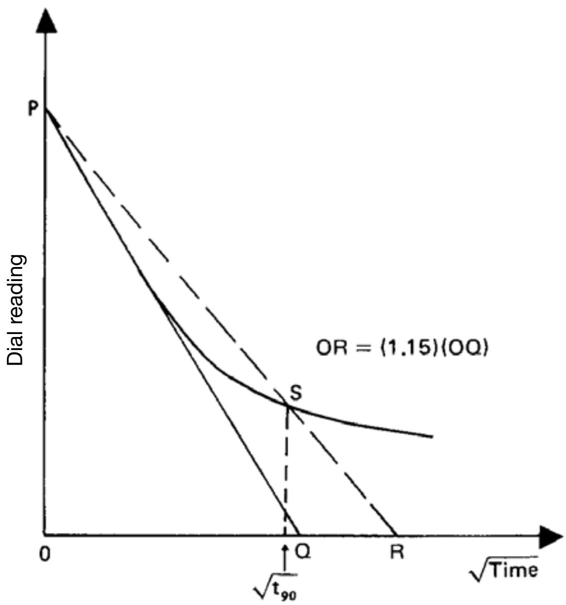

FIGURE 2A.28 Taylor's square root of time plot.

This abscissa was found by linear extrapolation from the point at which $U = 50\%$. But the same abscissa is found if the extrapolation is from some other point on the linear portion of the laboratory curve. At the settlement corresponding to $U = 90\%$, the ratio of the abscissa of the laboratory curve to that of the extended linear curve is

$$\frac{\left[F\left(\frac{H^2}{C_v}\right)^{0.5}(0.848)^{0.5}\right]}{\left[\left(\frac{9}{5}\right)F\left(\frac{H^2}{C_v}\right)^{0.5}(0.197)^{0.5}\right]} \tag{2A.60}$$

Secondary effects are in fact negligible for U less than 90%. At the point $U - 90\%$, $\Delta S_{90} = 0.9\Delta S_{100}$; since ΔS_{90} is known, ΔS_{100} is easily calculated.

It is convenient to select the point at which U is 45% to calculate c_v because the settlement at this point is the average of the known settlements S_0 and S_{90}, thus simplifying the construction used to find the point, and because selection of a point on the linear portion of the laboratory curve ensures that the theoretical and experimental curves will coincide in the region of greatest practical interest. The time factor at 45% consolidation is 0.159 (Table 2A.11). It may be noted that Taylor (1948) recommended use of the U = 90% point. The coefficient of consolidation is then calculated from:

$$C_v = \frac{0.159H_d^2}{t_{45}} \tag{2A.61}$$

where H_d is the average drainage distance during the consolidation period (half the average total thickness for double drainage) and t_{45} is the time corresponding to $U = 45\%$.

2A.7.1.4.2 Casagrande's Method of Calculating c_v. When Casagrande's method is used, the settlement is plotted versus the logarithm of time. Curves such as the one shown in Fig. 2A.29 (plotted using the same data previously shown in Fig. 2A.28) are typically obtained. An even spacing of points along the curve is obtained by using a geometric progression of times at which the deformations of the specimen are recorded. Typical times are 6, 15, and 30 seconds, 1, 2, 4, 8, 15, and 30 minutes, and 1, 2, 4, 8, and 24 hours, all measured from the instant of load application.

The value of S_0 is again obtained as shown on Fig. 2A.29, based on the parabolic approximation of the early part of the theoretical S–t curve.

TABLE 2A.11 Time Factors

U(%)	T	U(%)	T
10	0.008	55	0.238
15	0.018	60	0.287
20	0.031	65	0.342
25	0.049	70	0.405
30	0.071	75	0.477
35	0.096	80	0.565
40	0.126	85	0.684
45	0.159	60	0.848
50	0.197	95	1.127

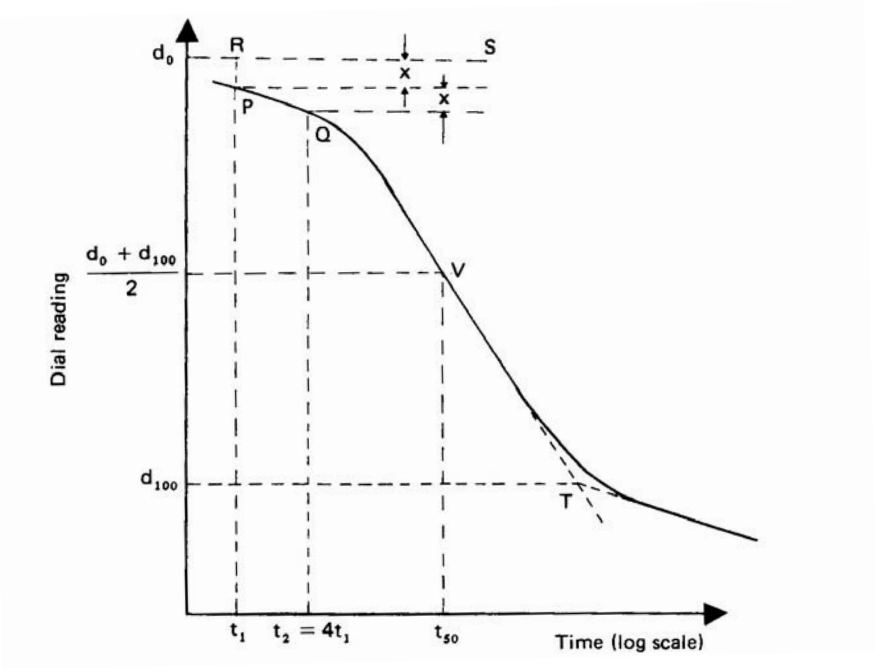

FIGURE 2A.29 Casagrande's log time plot.

$$\Delta S_2 = \Delta S_1 \sqrt{\frac{t_2}{t_1}} \tag{2A.62}$$

If $t_2 = 4t_1$, then, $\Delta S_2 = 2\Delta S_1$ and $S_0 = S_1 - (\Delta S_2 - \Delta S_1$. Because both Casagrande's method and Taylor's method for finding the corrected zero point are based on the assumption that the early part of the S–t curve is a parabola they should yield the same corrected zero point. They differ mainly in the fact that the parabolic part of the curve is clearly visible in the square root plot but is masked in the logarithmic plot. Thus, if Casagrande's method is used, t_2 should be chosen at about t_{50} to maximize the possibility that both points will be on the parabolic part of the curve.

The construction used to locate the maximum theoretical settlement, S_{100}, is shown in Fig. 2A.29. On the semilogarithmic plot, the experimental curves do not become asymptotic to a horizontal line, as required by Terzaghi's theory, but, instead, often become linear, or nearly linear, with a finite slope. The settlement S_{100} is estimated to be the settlement at the intersection of two straight lines, one drawn tangent to the sloping part of the laboratory curve and the other drawn tangent to the laboratory curve at the point of inflection.

To simplify construction and to make the theoretical and experimental curves coincide in the range of greatest field interest, c_v is calculated using t at $U = 50\%$.

It is interesting to note that the two methods give quite different values of S_{100} and thus different values of c_v. Since Taylor's method of finding S_{100} is based on the invalid assumption that secondary effects are negligible prior to 90% consolidation and Casagrande's method has no theoretical justification at all, it is not surprising that they sometimes yield different results.

2A.8 SHEAR STRENGTH OF SOIL

2A.8.1 Introduction

The shear strength of soil depends upon the consolidation pressure, the drainage during shear, the volumetric history (initial relative density of sands or stress history for clays), and other factors such as disturbance, strain rate, stress path, etc.

2A.8.2 Laboratory Tests for Shear Strength of Soil

Numerous types of devices have been developed to measure the shear properties of soil. By far, the most popular and widely used methods are the direct shear, triaxial shear, and unconfined compression devices.

2A.8.2.1 Direct Shear Testing of Soil

A schematic section through a direct shear apparatus is shown in Fig. 2A.30. A vertical or normal stress, σ_n, is applied. The horizontal shear stress, τ, is then increased until failure occurs. If the shear stress is plotted against the horizontal (shear), a curve similar to Fig. 2A.31 results.

Now a *series* of direct shear tests are performed on three different soil samples. However, each test is performed with a different vertical normal stress, $\sigma_n = N/A$ (Fig. 2A.32). The maximum shear stress for each test, τ_f, is called the shear strength.

Plotting the shear strengths from each test against the normal (vertical) stresses, and connecting these failure points results in a failure line, as shown in Fig. 2A.33. This failure line insects the τ axis at c, and is inclined at an angle ϕ to the horizontal. The equation of the failure line (failure envelope) is:

$$\tau_{max} = s = c + \sigma_n \tan \varphi \tag{2A.63}$$

where τ_{max} = shear stress at failure = s, shear strength
σ_n = normal stress on failure plane
φ = angle of internal friction
c = cohesion intercept

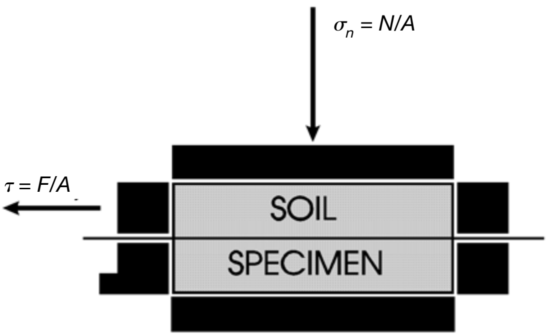

FIGURE 2A.30 Direct shear apparatus.

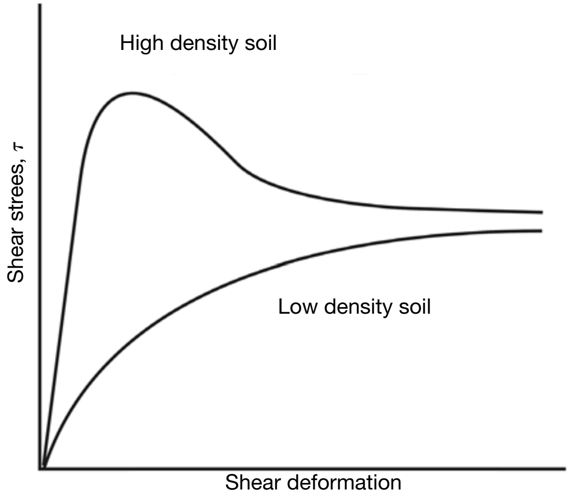

FIGURE 2A.31 Stress deformation for a soil.

2A.8.2.2 Triaxial Testing of Soil

With clays, the time to achieve drainage is important. In the direct shear test, the soil is initially loaded with a total vertical stress. The pore pressure generated by this load may be allowed to dissipate, permitting consolidation to occur. With time, substantially all of the pore pressure is dissipated; the soil reaches an equilibrium volume under the total stress, which is now an effective stress since no pore pressures exist.

The triaxial shear apparatus has proven to be superior to the direct shear box for the study of the

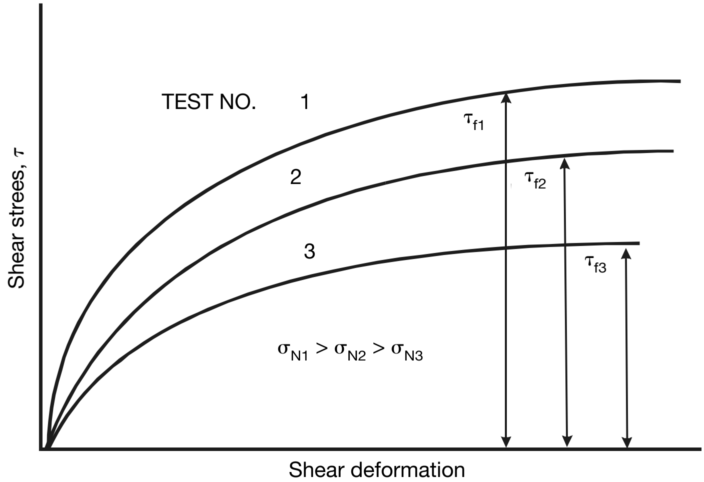

FIGURE 2A.32 Stress-deformation curves for direct shear test at differing vertical stresses.

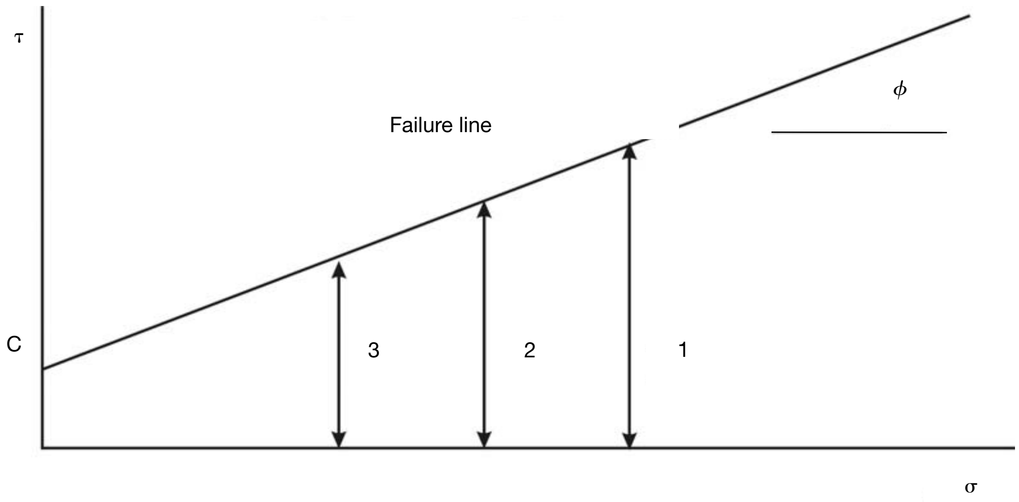

FIGURE 2A.33 Failure diagram for a soil test in direct shear.

consolidated undrained shear strength of a clay primarily because of its ability to control and monitor internal pore water pressures. Preceding the discussion of consolidated undrained shear strength, Mohr's stress theory and the mechanics of the triaxial test must be discussed.

2A.8.2.2.1 Stresses at a Point. Mohr's theory of stresses is fully explained in texts on mechanics of materials. However, that part of the theory pertinent to soil mechanics will be reviewed. Compressive stress will be considered positive, since stresses in soils generally are compressive and not tensile. Shear stresses tending to cause counterclockwise rotation will be defined as positive as well. Consider the stresses on a small two-dimensional element shown as Fig. 2A.34(A). If a plane at an angle (is passed through the element, there exist two resulting stresses on

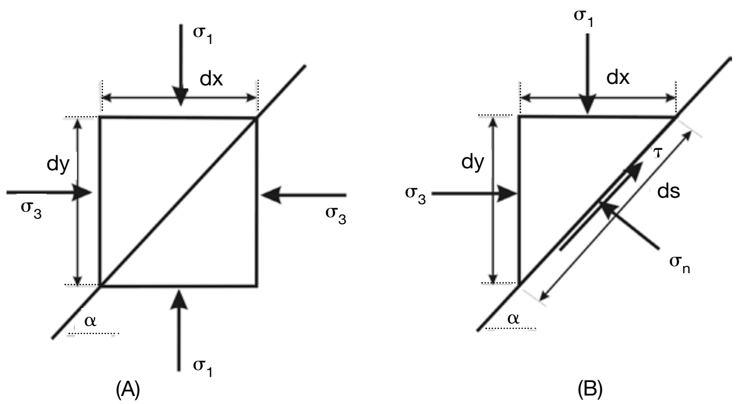

FIGURE 2A.34 Stress on a differential element.

that plane required for stability of the element (Fig 2A.34(A)). σ is the component of the resultant stress on a plane that acts at right angles to the plane, i.e., a normal stress. τ is the component of the resultant stress that is parallel to the plane, i.e., the shearing stress. σ_1 is the maximum normal stress on any plane through the point under consideration. There is no shearing stress on this plane. σ_3 is the minimum normal stress on any plane through the point under consideration. There is no shear stress on this plane. σ_3 acts at right angles to σ_1. σ_2 is the normal stress acting on a plane at right angles to the planes on which σ_1 and σ_3 act. There is no shear stress in this plane. By definition σ_2 may not exceed the magnitude of σ_1 nor may it be less than σ_3. σ_1, σ_2, and σ_3 are called principal stresses. They are orthogonal; that is, they act at right angles to one another.

If we consider Fig. 2A.34, we can determine the relationship between the normal and shear stresses, σ_n and τ, on a plane inclined at an angle α to the major principal plane, and the major and minor normal stresses, σ_1 and σ_3:

$$\tau = \frac{\sigma_1 - \sigma_3}{2}\sin 2\alpha$$

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\alpha \tag{2A.64}$$

These two equations allow the calculation of the stresses σ and τ on any plane inclined at an angle α to the plane of the major principal plane when σ_1 and σ_3 are known.

This can also be accomplished graphically using Mohr's circle. The ordinates represent shear stress and the abscissa, normal stresses (see Fig. 2A.35).

2A.8.3.2 Triaxial Tests

The direct shear test does not lend itself to the measurement of soil pore pressures, and the test has several other disadvantages. The triaxial test has become a popular method to determine the shear properties of a soil. The triaxial apparatus is shown in Fig. 2A.36.

Triaxial tests are performed in two stages. The first stage subjects the sample to a system of normal stresses (Fig. 2A.37). Usually, this is done by application of an all-around cell pressure

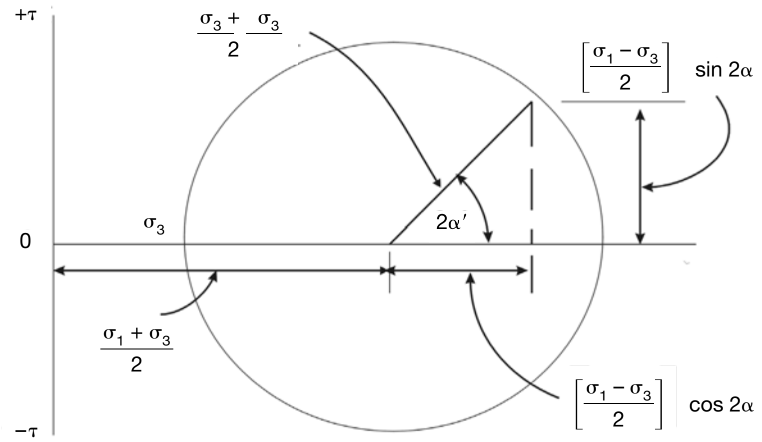

FIGURE 2A.35 Mohr's circle representation of stress on a differential element.

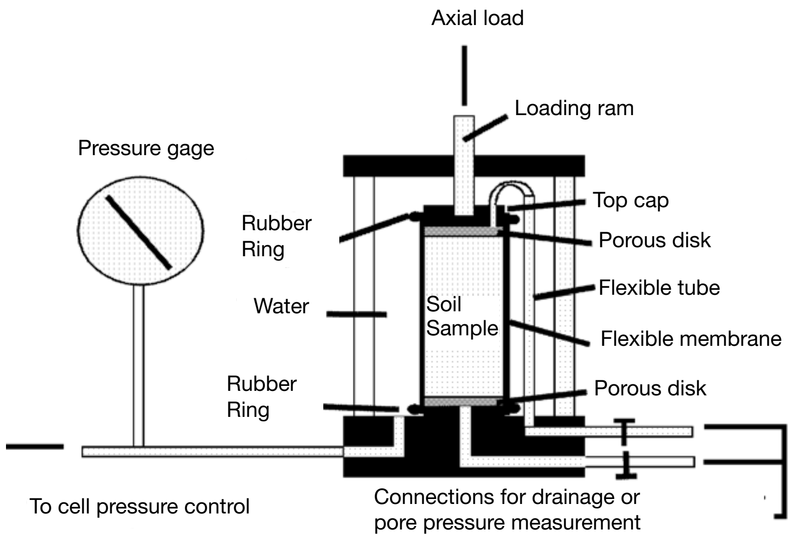

FIGURE 2A.36 Schematic of triaxial test equipment.

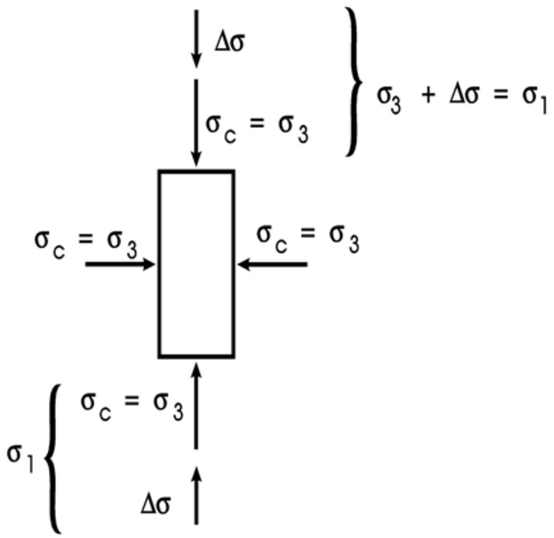

FIGURE 2A.37 Triaxial stresses on specimen.

($\sigma_c = \sigma_3$), in which case the stress acts in all directions. If the drainage passage from the porous stone is opened, the sample may be allowed to consolidate under the stresses applied during the first stage. Alternatively, it may be desired to prevent drainage during the first stage. Stage one can then be either *consolidated* or *unconsolidated*.

In the second stage, an axial stress ($\Delta\sigma$) may be applied to the sample through the loading piston. Again, drainage may or may not be permitted. Stage two may then be either *drained* or *undrained*.

Assuming isotropic stresses are applied in stage one, three types of tests are commonly performed on a soil sample.

2A.8.3.2.1 Consolidated Drained Test. In the first stage of this test, the soil is permitted to consolidate completely under the influence of the cell pressure. If the sample is saturated, the drainage connection from the porous stone may be connected directly to a burette. The progress of consolidation may be followed by measurement of the water outflow from (or inflow to) the sample. When consolidation is complete, with no further drainage from or into the sample, the second stage may proceed. An axial strain that causes a stress is applied so slowly that the pore pressures generated by the shear are permitted to dissipate. In the literature this test is commonly called a consolidated drained test (CD Test), a drained test, a slow test, or a S test.

2A.8.3.2.2 Consolidated Undrained Test. In this test, stage one is performed identically to stage one of the preceding test. In the second stage, drainage connections are closed as the sample is sheared to failure under undrained conditions. This test is called a consolidated undrained test (CU Test), a consolidated quick test (CQ or QC Test), or a R test.

2A.8.3.2.3 Unconsolidated-Undrained Test. In this test, the soil is not permitted to consolidate under the cell pressure, nor is drainage permitted during the shearing stage. This test is known as an unconsolidated undrained test (UU Test), a quick test, or a Q-Test.

2A.8.3.3 The Consolidated Drained Strength of Saturated Normally Consolidated Clay

Since, in this test, no pore pressures are allowed to build up, the stresses in the soil specimen are effective stresses. These stresses can easily be measured or calculated from the measured forces.

In the first stage, the soil is permitted to consolidate completely under the influence of the cell pressure. Mohr's circle for both total and effective stress is a single point along the σ axis with a value equal to the cell pressure, or σ_3.

During the second stage, shear stress is slowly applied by the piston. The applied stress ($\Delta\sigma$) is equal to the diameter of Mohr's circle. The cell pressure, equal to the minor principal stress (σ_3), is not varied during the course of the shear test. The progress of shear can be shown as a series of Mohr's circles increasing in diameter, anchored at their left side on the value of the cell pressure (Figure 2A.38).

For a single test, a plot of stress difference versus axial strain is shown in Fig. 2A.39. Points 1 through 5 in Fig. 2A.39 show values of the stress difference used to obtain the five Mohr's circles shown in Fig. 2A.38. It is obvious that the stress difference causes a shear stress so that the similarity of results to those obtained by direct shear is not surprising.

If a series of drained triaxial shear tests are performed on a normally consolidated soil with the consolidation (cell) pressure varied from test to test, a plot of the Mohr's circles for the maximum stress differences might appear as in Fig. 2A.40.

An envelope line, enclosing all possible Mohr's circles for this normally consolidated, saturated clay soil, will be a straight line and will pass through the origin when extended backward. The angle of inclination of this line is $\overline{\phi}$, approximately equal to the friction angle as measured in the drained direct shear test. The physical interpretation of this is that the shear strength is directly proportional to the consolidation pressure, i.e.,

$$s = t_f = \overline{\sigma} \tan\overline{\phi} \tag{2A.65}$$

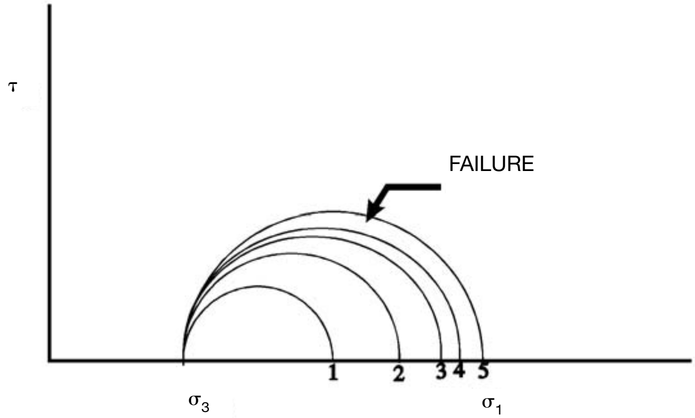

FIGURE 2A.38 Mohr's circles for increasing axial stress.

There is a failure on one plane in the triaxial sample, and this failure is on a plane theoretically represented by the point of tangency of Mohr's circle and the envelope.

2A.8.3.4 Consolidated Undrained Strength of Saturated Normally Consolidated Clay

In the consolidated undrained test, the first stage proceeds in the same manner as the consolidation phase of the consolidated drained triaxial test. The soil is consolidated to an isotropic (all-around) stress equal to the cell pressure. For the sample to remain normally consolidated, the cell pressure

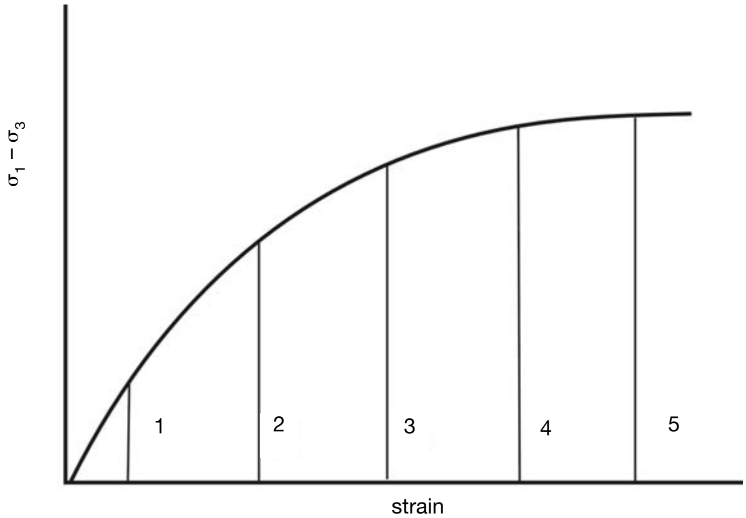

FIGURE 2A.39 Stress difference versus strain for triaxial S test on normally consolidated clay.

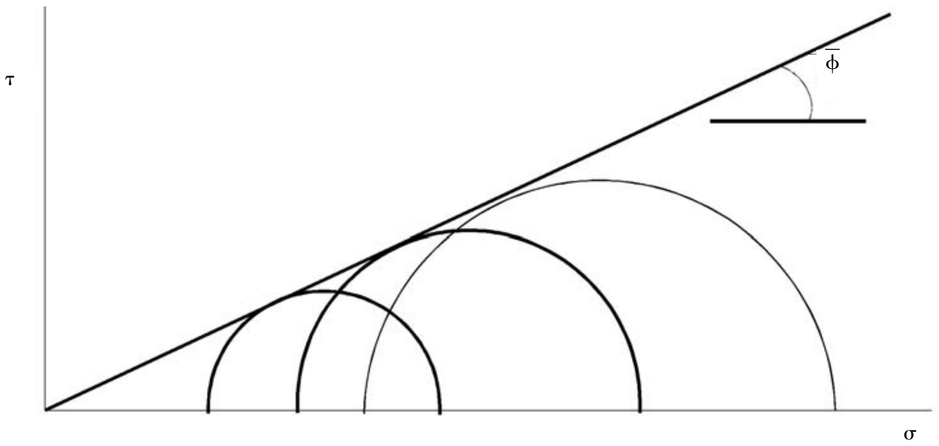

FIGURE 2A.40 S-test envelope for normally consolidated clay.

must exceed the consolidation pressure of the soil in the ground. If plotted on a Mohr's diagram, at the end of the consolidation phase, the stresses would plot as a point circle along with the σ axis, since, as with the drained test, the progress of consolidation may be monitored. Upon completion of consolidation, the shear phase begins. To maintain an undrained condition, drainage from this sample is prevented. Since no water leaves the system, the shearing occurs at constant volume.

A volume change versus strain curve is not appropriate because there is zero volume change at all strains. On the Mohr's diagram, Fig. 2A.41, one may see that the diameter of the failure circle for the undrained test is about half the diameter of the failure circle for the drained case at the same consolidation stress.

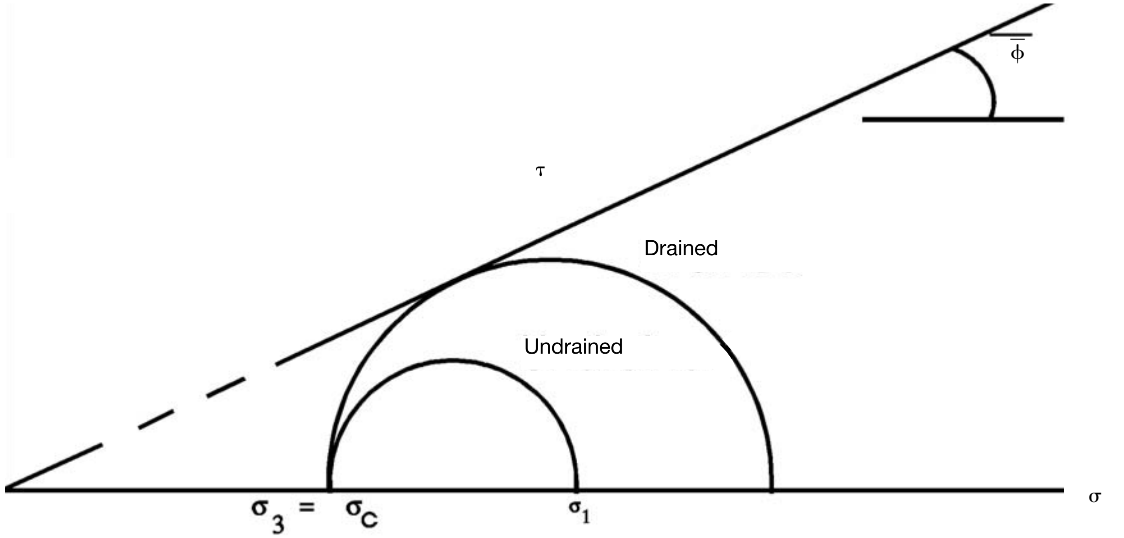

FIGURE 2A.41 Mohr's circle for undrained test compared to drained test.

A second sample is consolidated to a higher stress and sheared under constant volume. The corresponding Mohr's circle (Fig. 2A.42) shows a proportionately larger failure circle, but again, only about one half the diameter of the drained failure circle at the higher consolidation stress. Since this proportionality exists, one may draw a failure envelope.

This envelope is also a straight line, which can be traced backward through the origin. It is known as the R or CU envelope. The dotted line portion shows extrapolation back to the origin, at stresses below the field consolidation pressure of the soil. If, in the laboratory, a sample had been consolidated to a stress less than the field consolidation stress, it would no longer be normally consolidated since stresses were at one time greater than now. Its strength would also be slightly greater than shown by the dotted line, as will be explained in the section on overconsolidated soils.

With only a simple adjustment to the R test, one can obtain a great deal more data. Instead of closing the drainage connection to the base of the sample, if, at the end of consolidation, a pore pressure transducer is placed in the line, one can still run a test essentially undrained during the shearing phase. But the pore pressures caused by the shearing can be constantly monitored. This test is known as the R (R-bar) test.

After consolidation, as the soil is sheared under the stress difference, there is a tendency for the soil to decrease in volume as with the drained test. But drainage is prevented; water cannot leave the soil. The pore water pressure gradient that existed to move the water out of the soil is now permitted to build up. A positive pore water pressure is generated and is measured by the transducer. Typical stress and porewater pressure versus strain curves are shown in Fig. 2A.43.

During drained shear it was not necessary to differentiate between total stresses and effective stresses. The pore pressures were zero; total stresses were equal to the effective stresses and a single Mohr's circle results.

For the R test, the consolidation phase (which implies full drainage) yields a single point Mohr's circle equal to the cell pressure. However, the moment the undrained shear begins, pore pressures are generated and

$$\begin{aligned}\bar{\sigma}_1 &= \sigma_1 - u \\ \bar{\sigma}_3 &= \sigma_3 - u\end{aligned} \tag{2A.66}$$

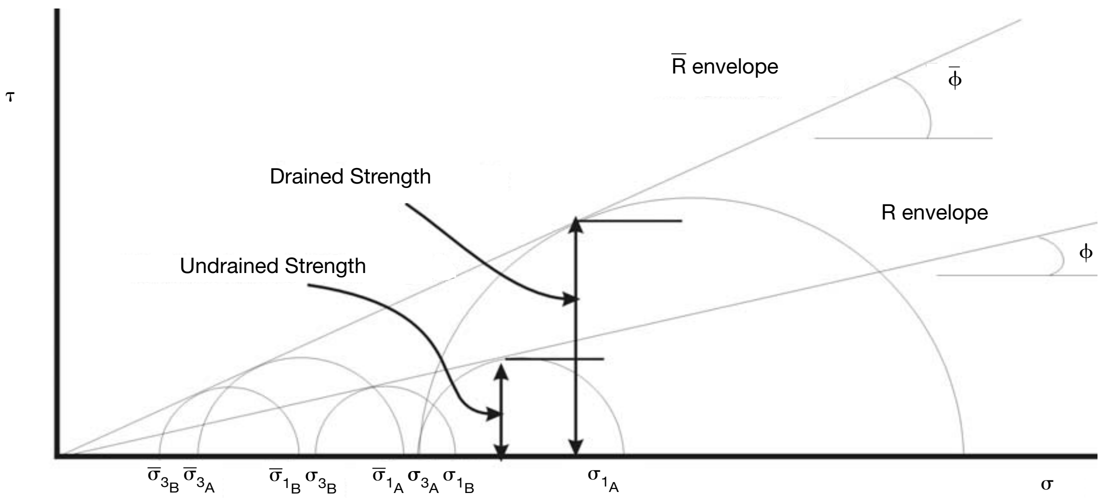

FIGURE 2A.42 Mohr's circles for increasing consolidation pressures in the undrained test.

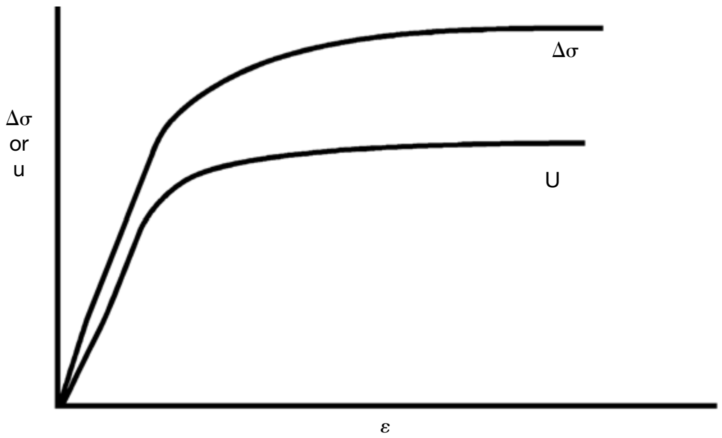

FIGURE 2A.43 Typical stress and pore pressure versus strain in undrained shear.

Since the cell pressure is constant and equal to σ_3 in the triaxial compression test, the total stress Mohr's circles are anchored on the left at $\sigma_3 = \sigma_c$, just as in the drained test. However, the effective stress σ_3 becomes less than the cell pressure by the magnitude of pore pressure generated. Since pore pressure constantly increases with shear, the effective stress Mohr's circles are constantly shifting to the left, i.e., σ_3 is constantly decreasing. For any given time or strain, there are two Mohr's circles, a total stress circle with $\sigma_3 = \sigma_{\text{cell}}$ and an effective stress circle with σ_3 and σ_1, displaced leftward (reduced) by the amount of the generated pore pressure at that time or at that given strain. Fig. 2A.44 illustrates this.

The total stress Mohr's circle will have the same diameter as the effective stress circle:

$$\begin{aligned}\Delta\sigma = \bar{\sigma}_1 - \bar{\sigma}_3 &= \text{diameter of Mohrs circle}\\ &= (\sigma_1 - u) - (\sigma_3 - u)\\ &= \sigma_1 - u - \sigma_3 + u\\ &= \sigma_1 - \sigma_3\end{aligned} \tag{2A.67}$$

The effective stress and total stress Mohr's circles will be identical in size and all points on the effective stress circle are simply displaced by the magnitude of the pore pressure.

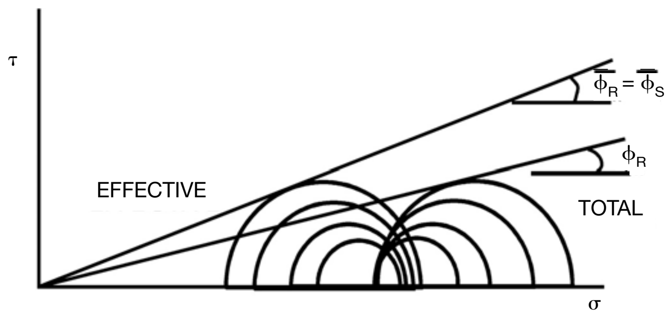

FIGURE 2A.44 Total and effective stress circles for undrained test.

In Fig. 2A.44 note that shear continues until the effective stress Mohr's circle touches the effective stress envelope. This envelope is essentially the same as the drained envelope.

In the drained direct shear test, the normal effective stress remains constant as shear stress is increased to failure. In the drained triaxial test, normal effective stress on the potential plane of failure increases slightly as shear stresses increase rapidly. In the consolidated undrained triaxial test, normal effective stresses generally decrease somewhat due to generation of pore pressures while shear stresses increase to failure.

It should be quite clear now that effective stresses at failure govern the strength of a soil. For a given normally consolidated clay, the effective normal stress on a potential failure plane governs the shear strength of the soil, both drained and undrained. Even though this effective stress concept is relatively simple to explain, in practice we more often wish to relate the strength to stresses *before* shearing. Relating undrained strength to consolidation pressure before shearing is more useful for engineering predictions.

Previously, the results of performing R tests (without pore pressure measurement) on two samples consolidated to different pressures were explained. We now perform two R tests (with pore pressure measurement) on samples consolidated to different pressures. Results are shown in Fig. 2A.45 and Fig. 2A.46. Note that the total stress circles, with σ_3 equal to the cell pressures, form the R envelope. The effective stress circles each touch the R or effective stress envelope which is essentially the same envelope as would be obtained from drained tests. Note also that for the larger consolidation pressure, larger pore pressures are generated. With increasing consolidation pressure, Mohr's circle at failure (the shear strength) increases proportionately, as does the failure pore pressure, which is shown by the offset of the effective stress circle from its total stress circle. In each case, the offset is approximately equal to the diameter of Mohr's circle. Only the $\bar{R}$ envelope represents a failure line, where stresses become critical on a plane on which a given effective stress acts. The R envelope is simply an envelope enclosing the maximum size Mohr's circle.

Sometimes the R envelope is called a total stress envelope. This leads to confusion. Total stress conditions are meaningless unless they are related to effective stresses. In the case of the R envelope, it is an envelope of total stresses at undrained failure only if and when the σ_3 of the failure cir-

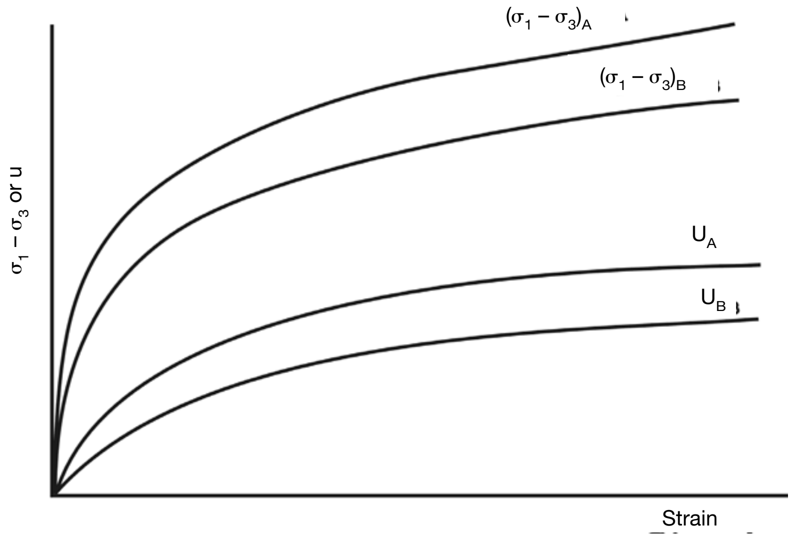

FIGURE 2A.45 Stress and pore pressure versus strain for two tests at two consolidation pressures.

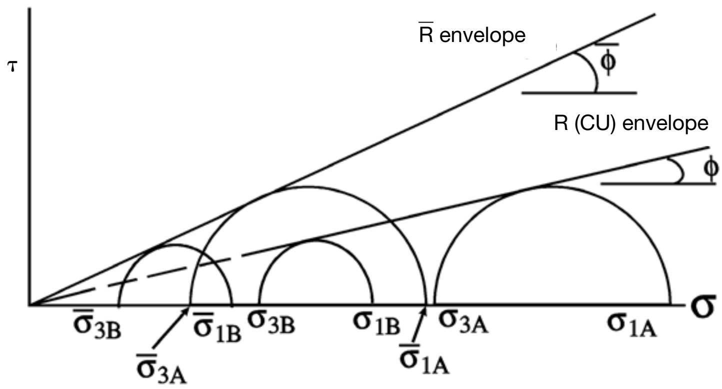

FIGURE 2A.46 Mohr's circles for two tests.

cle is the consolidation pressure of the sample. In this way it bears a fixed relationship to effective stresses.

Figure 2A.46 also shows, for a given consolidation pressure, the drained strength and failure circle, and both the undrained failure circles and the undrained strength. Now a main advantage of the $\overline{R}$ test becomes evident. From the $\overline{R}$ envelope, we can predict the drained strength of a soil at any given consolidation pressure. The $\overline{R}$ test gives predictions of both the drained and undrained strengths. Moreover, the drained test requires a very slow shear rate so that pore water pressures generated in the failure zone can migrate from the sample. The R test still requires time for pore pressures in the failure zone to equilibrate throughout the sample, but the time involved is substantially less than the time for full drainage.

From the foregoing we can conclude that for a normally consolidated saturated clay (constant stress history) for the drained condition, shear strength is directly proportional to the consolidation pressure. For the undrained case, the same is true, but the constant of proportionality is reduced to tan ϕ.

2A.8.3.5 Unconsolidated Undrained Test and Unconfined Compression Test of Normally Consolidated Clay

The undrained strength of the soil in this test is also directly proportional to the consolidation pressure, which is the consolidation pressure of the soil in the ground.

In this test there is no drainage allowed and hence no volume change in the specimen can occur. Consequently, when the sample is sheared, identical effective stress circles will develop no matter what the value of the cell pressure. At failure, the strength will also be the same for all tests. The failure total stress circle will be the same size as the effective stress circle, but displaced by the greater pore pressure (Fig. 2A.47). A Mohr's plot for total stresses for the UU tests have resulted in a series of circles of the same diameter. The UU envelope, with respect to total stresses, has a ϕ of zero and cohesion intercept, c. There may be a number of total stress circles, but only one effective stress circle and this one is governed by the common consolidation pressure.

Both the cost of triaxial apparatus and the requirement for highly skilled labor used in triaxial testing are large. Since the strength of the UU sample was dependent only on the consolidation stress in the ground and independent of the cell pressure, the use of a zero confining stress would be obvious. It dispenses with the need for much of the triaxial apparatus and should result in a Mohr's circle of equal size by the $\phi = 0$ principle. If we perform such a test, called the unconfined compression test, ideally the resulting Mohr's circle is of size equal to those failure circles produced by the unconsolidated undrained test (Fig. 2A.48). In such a test, we cannot measure pore pressures, so the

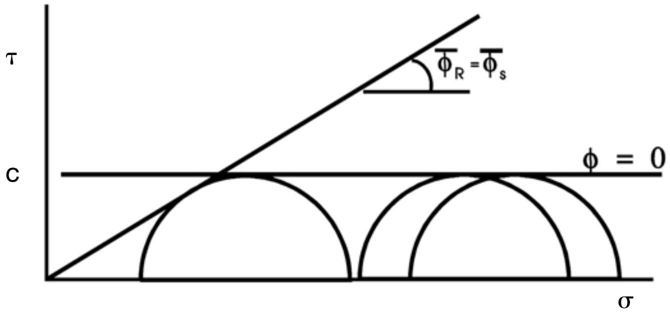

FIGURE 2A.47 Total and effective stress circles for the UU test.

position of the failure effective stress circle cannot be established. Although we have deduced the radius of the effective stress Mohr's circle (it is the same as the total stress circle radius), we cannot know what is the effective stress friction angle of the soil. The radius is also half the diameter of the circle, which is the unconfined compressive strength, q_u, which in turn is σ_1 at failure when σ_3 is zero. In general then, for undrained shear strength of cohesive soils:

$$s = \tau_f = c = \frac{q_u}{2} = \frac{\Delta\sigma}{2} \tag{2A.68}$$

2A.8.3.6 Shear Strength of Saturated Overconsolidated Soil

A reconsolidated or overconsolidated soil is one which, at some time in its geologic history, has had acting on it a consolidation pressure greater than that presently acting on it. Consolidation, of course, implies complete drainage and transfer of total stresses to the effective stresses of the soil skeleton. The causes of the preconsolidation and subsequent removal of effective stress might have been by deposition and subsequent erosion, a glacial load and melting, desiccation with subsequent more humid conditions, or any of the other causes discussed when the subject of consolidation was treated.

Fig. 2A.49 shows, on a natural scale of void ratio versus effective stress, a soil normally consolidated to point C and unloaded to point E. This plot is similar to the usual consolidation curve except that is shown as the effective stress on a natural scale and not log p, effective stress plotted on a log-

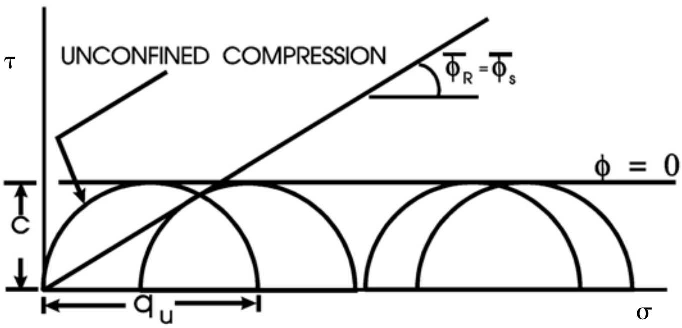

FIGURE 2A.48 UU envelope and unconfined compression, Mohr's circle.

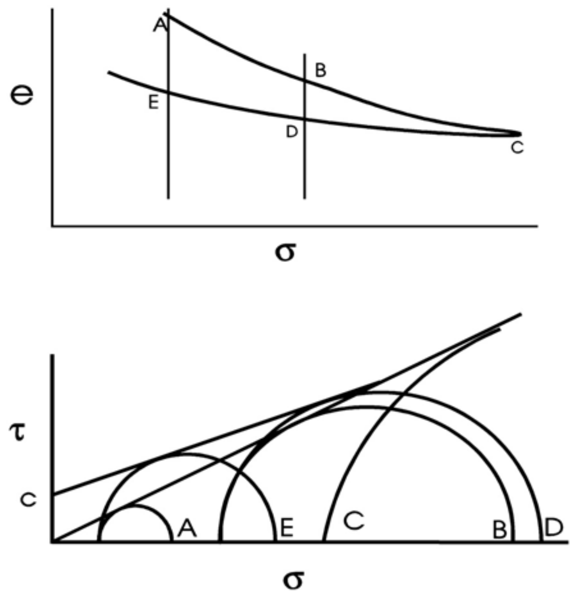

FIGURE 2A.49 Volume change and Mohr's circle plots for overconsolidated soils.

arithmic scale. What would be a relatively straight line on a logarithmic plot will be shown concave upwards on an arithmetic plot. This curve is a consolidation curve, even though the data were obtained from triaxial consolidation tests where $\sigma_1 = \sigma_2 = \sigma_3$.

In Fig. 2A.49, circles A, B, and C are for soils that are normally consolidated. Points D and E correspond to the consolidation pressures of B and A, respectively, but each of these soils has been reconsolidated to the stress level of C. Fig. 2A.49 shows Mohr's failure circles for soils A–E for the consolidated drained condition. The diameter of the failure circles and the shear strength of the soil are directly proportional to the consolidation pressure, which is the σ_3 of the drained failure circles.

It should be expected that density expressed as void ratio and effective consolidation stress will control the strength of these soils. Soil D has been consolidated to the same effective stress as soil B. Soil D, however, is shown in Fig. 2A.49 to be denser, to have a lower void ratio than B. It would therefore be expected that D will be stronger than B. Fig. 2A.49 shows that this is so.

Soil E is denser than A, and both have been consolidated to the same effective stress level. Soil E was at one time reconsolidated (with drainage, of course) to the stress level of C. In rebound, it had only a slightly greater void ratio than C. The ratio of the σ_3 of E and the σ_3 of C is the overconsolidation ratio, which is shown to be on the order of eight or so. Again, soil E is stronger than A, as denoted by the larger Mohr's failure circle.

In comparing the failure circles, consolidation stresses, and void ratios it is apparent that the strength of these soils is controlled more by effective consolidation stresses than by void ratios.

The failure envelope through the origin and tangent to Mohr's circles A, B, and C was previously introduced as the drained strength envelope for a normally consolidated clay, or the S envelope. It indicates that the shear strength is directly proportional to the effective consolidation pressure and in particular to the consolidation pressure on the plane of failure, i.e., where the Mohr's circle touches the envelope.

The failure envelope for soils C–E, the soils that have been reconsolidated to the stress level of C, is not strictly a straight line; it is curved downward slightly and does not pass through the origin.

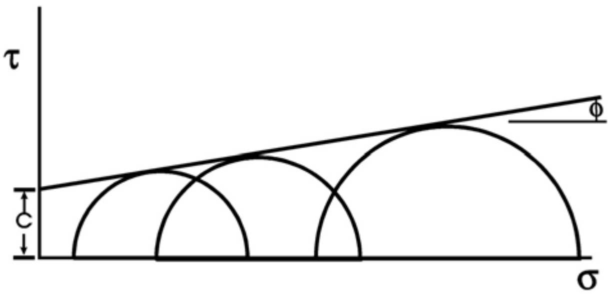

FIGURE 2A.50 Overconsolidated soil Mohr's envelope.

It joins the ϕ_ss envelope for normally consolidated soil at the point where the failure circle for C is tangent to the normally consolidated failure circle. When tests are performed on a reconsolidated soil, the final consolidation pressures generally are in a relatively narrow range and are generally substantially less than the preconsolidation pressure. In this range, it is customary to fit a straight line to the circles obtained from the tests. Such a line will have a slope expressed as a ϕ angle and will have a cohesion intercept c. Fig. 2A.50 shows such an effective stress envelope.

It can be concluded that the greater the preconsolidation pressure, the higher is the envelope, and therefore the greater is c in the expression $\tau = c + \sigma \tan\phi$. It can be seen also that c and ϕ are not material properties of a soil.

Typical stress–strain and volume change–strain properties of heavily reconsolidated soils are shown in Fig. 2A.51 and Fig. 2A.52, respectively. The soil behaves very much the same as does a dense sand, having a peak shearing resistance at low strain, and dilating or expanding in volume at strains greater than those mobilized at the peak strength.

2A.9 CORRELATIONS BETWEEN SOIL INDEX PROPERTIES AND FOUNDATION DESIGN PARAMETERS

Often, high-quality laboratory shear strength data is unavailable, at least for preliminary studies. Consequently, it is necessary to estimate shear strength parameters from other, less rigorous evalua-

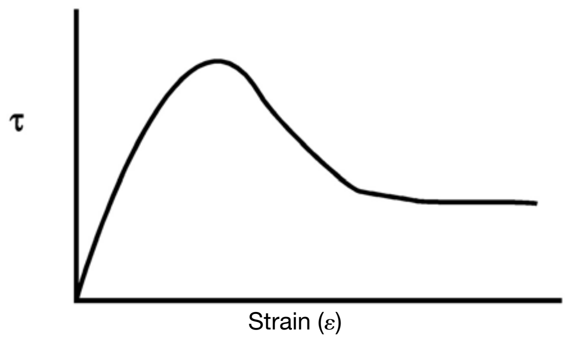

FIGURE 2A.51 Stress–strain for overconsolidated clay.

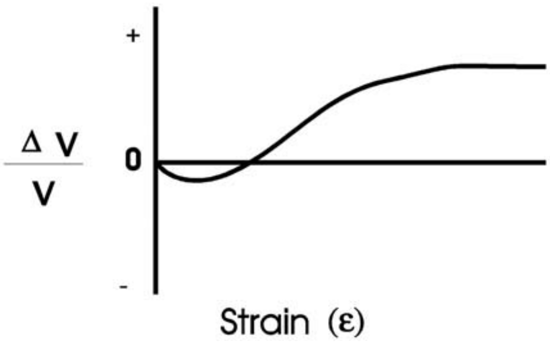

FIGURE 2A.52 Strain–volume change for overconsolidated clay.

tion. Methods for evaluating effective stress friction angles from soil index properties are presented below.

2A.9.1 Effective Stress Friction Angle of Cohesionless Soils

Many researchers have presented correlations for estimating the effective stress friction angles for cohesionless soils. These correlations are usually with soil type, relative density, and unit weight or void ratio. Fig. 2A.53 shows a general relationship between effective stress friction angle and dry density.

2A.9.2 Effective Stress Friction Angle of Cohesive Soils

Correlations for effective stress friction angles for cohesive soils are generally limited to normally consolidated soils and remolded clays and residual (large strain) friction angles. Mitchell[14] published the following correlation for normally consolidated soils:

$$\sin \varphi \approx 0.8 - 0.094 \ln(PI) \qquad (2A.69)$$

where φ = effective stress friction angle for normally consolidated soil
PI = Plasticity Index

Fig. 2A.54 illustrates the typical changes in the residual friction angle for soils at the Amuay landslide site with changes in effective stress and plasticity. P_a is the atmospheric stress in appropriate units.

2A.9.3 Undrained Shear Strength of Cohesive Soils

The undrained shear strength, s_u, is probably the most widely used parameter for describing cohesive soils. It is affected by the type of test used, boundary conditions, rate of loading, and confining stress level. Skempton suggested the following correlation for normally consolidated soil, s_u, from the field vane shear test:

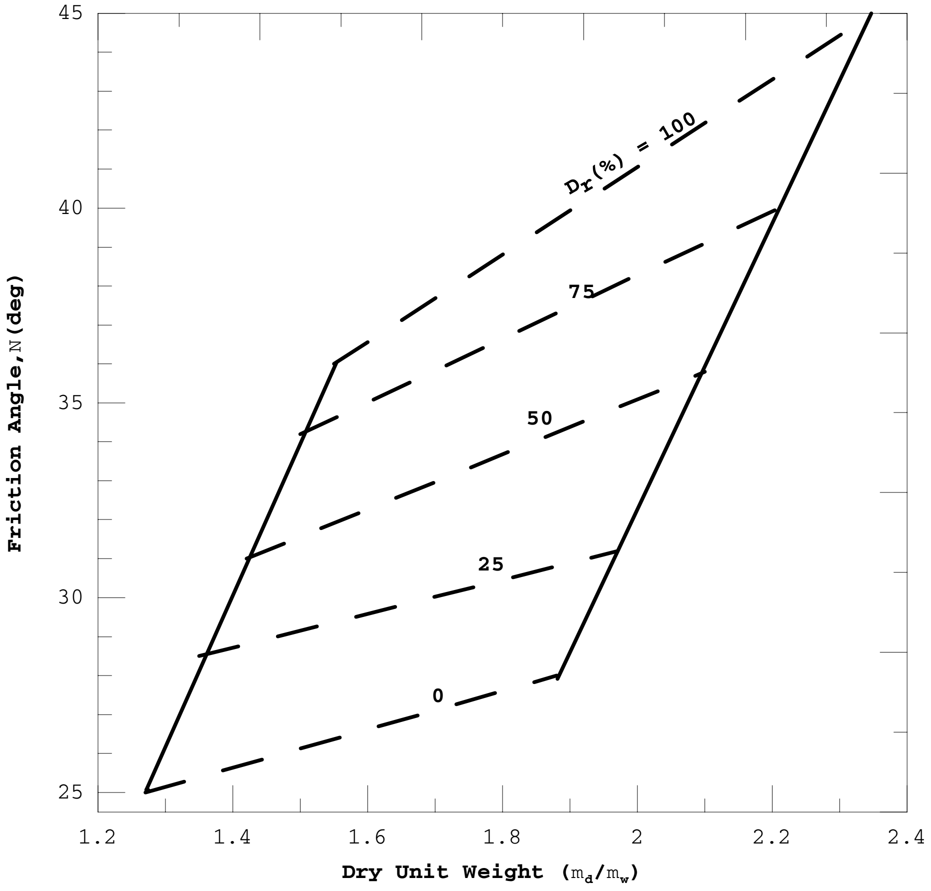

FIGURE 2A.53 Friction andgle versus relative density and unit weight.

$$\sigma_u/\bar{\sigma}_{vo} = 0.11 + 0.037PI \tag{2A.70}$$

Chandler[15] modified this equation to take into account the preconsolidation stress (σ_p):

$$\frac{s_u}{\bar{\sigma}_p} = 0.11 + 0.0037PI \tag{2A.71}$$

The accuracy is said to be ±25%.

Similarly, Jamiolkowski et al.[16] Provided the following equation for low to moderate PI soils:

$$\frac{s_u}{\bar{\sigma}_{vo}} \approx (0.23 \pm 0.04)OCR^{0.8} \tag{2A.72}$$

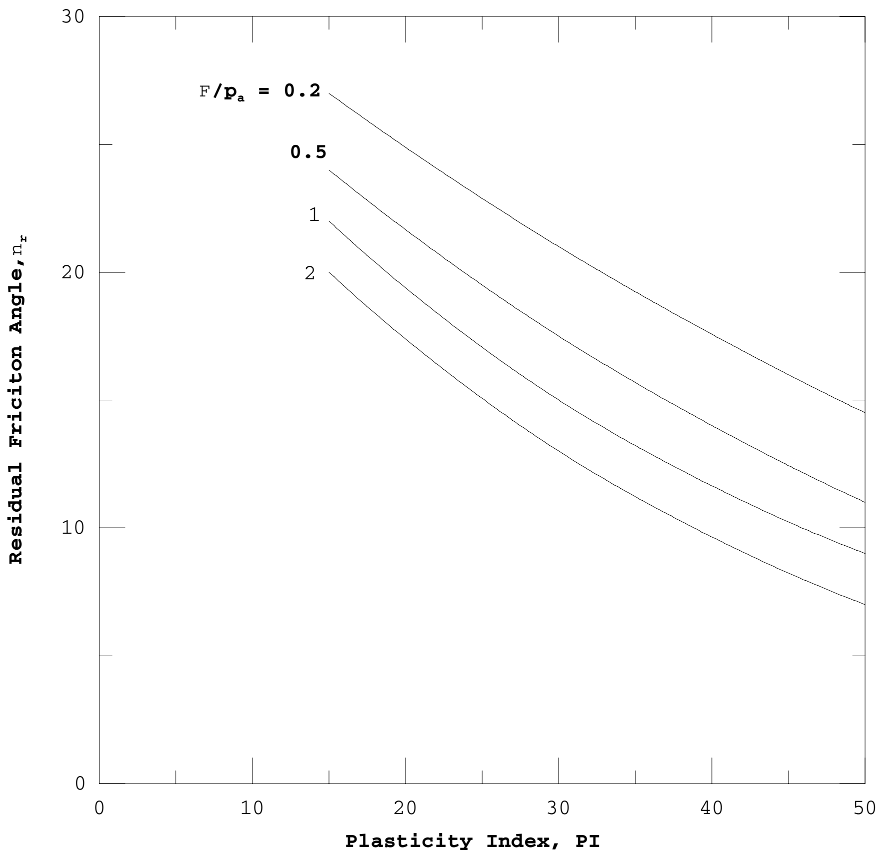

FIGURE 2A.54 Residual friction angles for Amuay landslide soils versus PI and effective normal stress.

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SECTION 2B
SHALLOW FOUNDATIONS

RICHARD W. STEPHENSEN

2B.1 INTRODUCTION

A foundation is a structure built to transfer the weight of a building to the material below. This transfer must occur such that the soil below does not rupture or compress to such magnitude that the integrity of the superstructure is threatened.

Foundations are generally classified as either deep or shallow. The depth of the bearing area of shallow foundations is generally no deeper than about the width of the bearing surface.

Deep foundations provide support for a structure by transferring the loads to competent soil and/or rock at some depth below the structure.

2B.2 SHALLOW FOUNDATIONS

Shallow foundations can be either footings or mats. They consist of reinforced concrete slabs formed directly on a prepared soil base. Footings may be either spread, combined, or continuous.

2B.2.1 Footings

Spread footings are footings that support one column or load. These footings are also called isolated or column footings. They typically are 3 to 8 to 10 ft (0.9 to 2.4 to 3 m) square. Their bearing surface depth is typically less than 2.5 times their length ($D_f < 2.5B$) (Figure 2B.1).

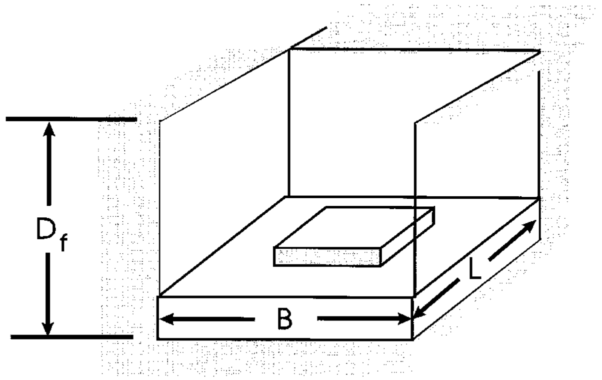

FIGURE 2B.1 Isolated shallow footing.

Combined footings are similar to spread footings but support two or more columns. The shape is more likely to be a rectangle or occasionally trapezoidal (Figure 2B.2). These footings are used where column spacing is nonuniform and for the support of exterior columns near property lines where there isn't enough room for a spread footing.

A continuous or strip footing is an elongated shallow foundation that typically supports a single row of columns or a wall or other type of strip loading. Continuous footings tie together columns in one direction at their base, and reduce construction costs through use of appropriate equipment for trenching (Figure 2B.3).

The advantages of shallow foundations lie primarily in their low cost and speed of construction.

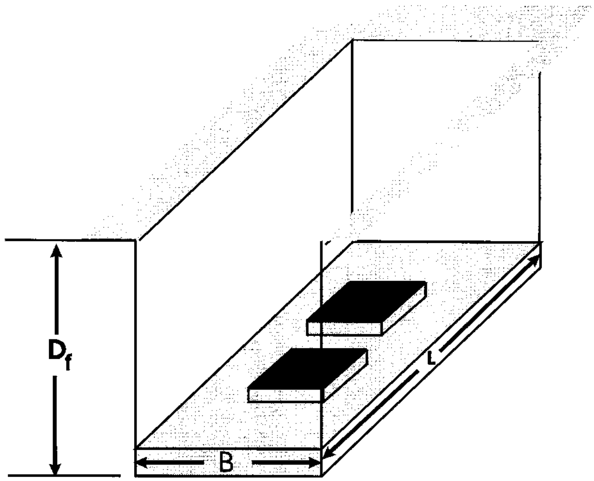

FIGURE 2B.2 Combined footing.

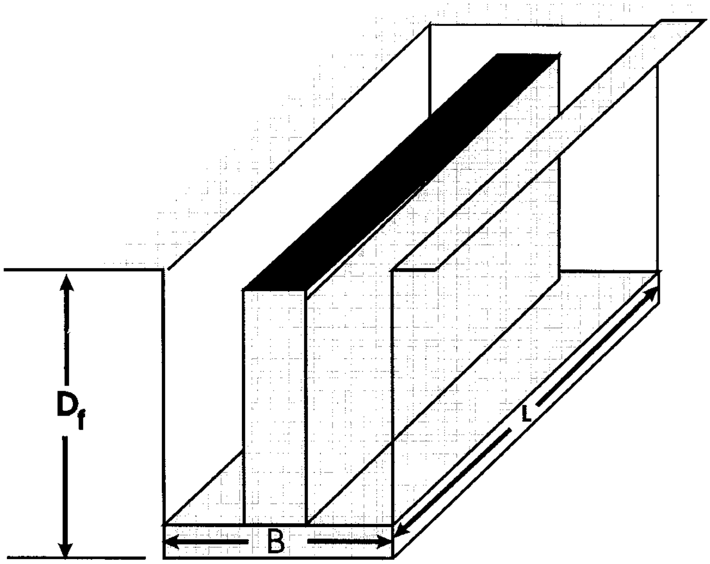

FIGURE 2B.3 Continuous footing.

2B.2.2 Mats

A mat (raft) foundation is a structural reinforced concrete slab that supports a number of columns distributed in both horizontal directions or supports uniform pressure, as from a tank. Rafts are used to bridge over soft spots if the spots are very localized, and to reduce the average pressure applied to the soil (Figure 2B.4).

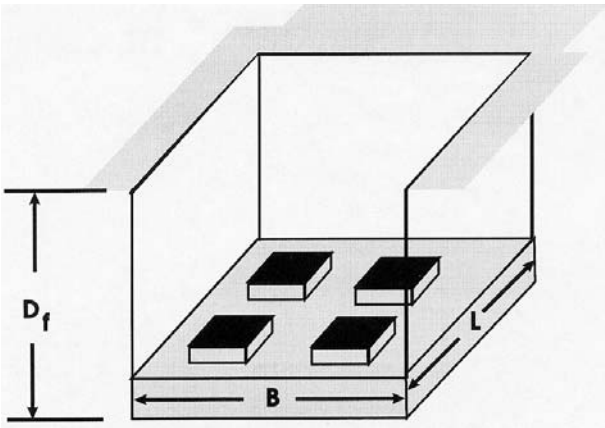

FIGURE 2B.4 Mat foundation.

2B.3 DESIGN PARAMETERS

Design parameters for shallow foundations fall into two classes: structural design parameters and geotechnical design parameters.

2B.3.1 Structural Design Parameters

Structural design parameters that influence the design of the shallow foundation include the building type and use, loading (live, dead, and uplift), column spacing, presence or absence of a basement, allowable settlement, and applicable building codes.

2B.3.2 Geotechnical Design Parameters

Geotechnical factors that influence the design include the thickness and lateral extent of bearing strata, the depth of frost penetration, the depth of seasonal volume change, and the cut/fill requirements. The strength, compressibility, and shrink-swell potential of the bearing strata are the properties of concern.

In addition, the presence or absence of groundwater and its minimum and maximum elevations have an important impact on the design process.

2B.4 BEARING CAPACITY OF SHALLOW FOUNDATIONS

The design of a shallow foundation requires that the applied load does not exceed the load that would cause the soil strata beneath the foundation to rupture. The maximum load that can be applied to the foundation soil without rupture is called the bearing capacity.

2B.4.1 Development of General Bearing Capacity Equation

Simplifying the model of a shallow, continuous footing at impending failure, as in Figure 2B.5, allows the problem to be treated as an earth pressure problem. When the footing is loaded, the wedge of soil beneath the footing generates lateral pressures at the wedge boundaries. These pressures cause the adjacent wedges to displace, as shown in Figure 2B.5. The slip surfaces for the wedge beneath the footing develop slip lines at $\alpha = 45 + \phi/2$ with the horizontal. The adjacent wedge has slip line angles of $\rho = 45 - \phi/2$ with the horizontal.

If the effect of the soil above the base level of the footing is replaced with a surcharge γD_f then

$$\bar{q} = \gamma D_f \tag{2B.1}$$

A look at the stress block on the right allows computations of the total resisting earth pressure as force P_p from equation (2B.2).

$$P_p = \int_0^H \sigma_1 (dz) = \int_0^H \left[(\gamma z + \bar{q}) \tan^2\left(45 + \frac{\phi}{2}\right) + (2c)\tan\left(45 + \frac{\phi}{2}\right) \right] dz \tag{2B.2}$$

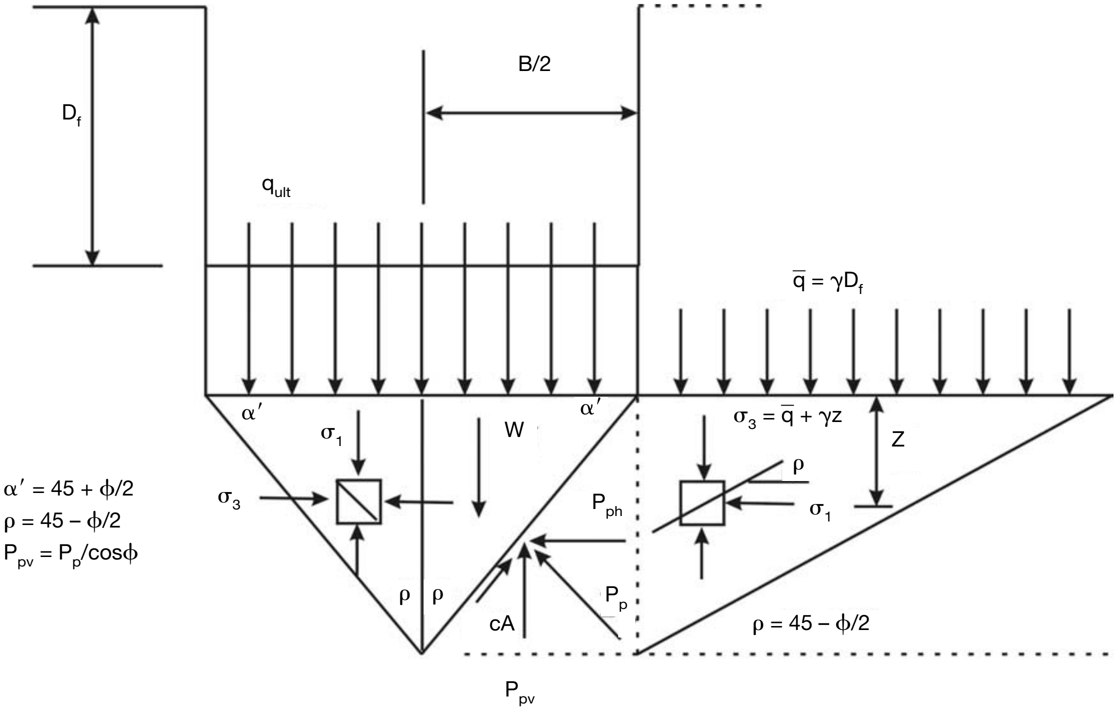

FIGURE 2B.5 Simplified bearing capacity diagram.

where σ_1 is

$$\sigma_1 = \sigma_3 N_\phi + 2cN_\phi \tag{2B.3}$$

where c is the cohesion and ϕ is the angle of internal friction.

Defining K_p as in equation (2B.4), the result of the integration is given in equation (2B.5).

$$K_p = N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right) \tag{2B.4}$$

$$P_p = \frac{\gamma H^2}{2} K_p + \bar{q} H K_p + 2cH\sqrt{K_p} \tag{2B.5}$$

To find q_{ult}, the vertical forces are added to obtain

$$q_{ult}\left(\frac{B}{2}\right) + \gamma\left(\frac{B}{2}\right)\left(\frac{H}{2}\right) - (cA)\cos\rho - \frac{P_p}{(\sin\rho)(\cos\phi)} = 0 \tag{2B.6}$$

since

$$H = \left(\frac{B}{2}\right)\tan\alpha; \quad W = \frac{1}{2}\gamma\left(\frac{B}{2}\right)\left(\frac{B}{2}\right)\tan\alpha$$

$$A = \frac{B}{2\cos\alpha}; K_p = \tan^2\left(45 + \frac{\phi}{2}\right) \tag{2B.7}$$

$$P_{p,v} = \frac{P_p}{\cos\phi}; K_a = \tan^2\left(45 + \frac{\phi}{2}\right)$$

then

$$q_{\text{ult}} = c\left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right] + \bar{q}\frac{\sqrt{K_p}K_p}{\cos\phi} + \frac{\gamma B}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right] \tag{2B.8}$$

This equation can be written as

$$q_{\text{ult}} = cN_c + \bar{q}N_q + \gamma BN_\gamma \tag{2B.9}$$

where

$$N_c = \left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right]$$

$$N_q = \frac{\sqrt{K_p}K_p}{\cos\phi} \tag{2B.10}$$

$$N_\gamma = \frac{1}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right]$$

This is a form of the *general bearing capacity equation*. The N factors are the *bearing capacity* factors.

N_c = nondimensional bearing capacity factor relating the influence of soil cohesion on bearing capacity (a function of ϕ of the soil).

N_q = nondimensional bearing capacity factor relating the influence of soil overburden on bearing capacity (a function of ϕ of the soil).

N_γ = nondimensional bearing capacity factor relating the influence of soil unit weight on bearing capacity (a function of ϕ of the soil).

Equation (2B.9) generally underestimates the capacities of footings.

Various investigators have studied the bearing capacity problem. Each has made assumptions as to the character of the failure surface, the effect of the footing depth and shape and other factors. Although almost all the investigators developed equations similar to equation (2B.9), they computed different values of the bearing capacity factors. Some researchers included modifications to account for the footing depth, shape and inclination of loading. Bearing capacity equations reported by several authors are given in Bowles[1].

2B.4.2 Net Ultimate Bearing Capacity

The net ultimate bearing capacity is defined as the ultimate bearing capacity of the foundation in excess of the surcharge from the surrounding soil In most cases, the difference in the unit weight of soil and the unit weight of concrete is ignored. Consequently

$$q_{\text{net}} = q_{\text{ult}} - \gamma \cdot D_f - \overline{q} \tag{2B.11}$$

$$q_{\text{ult}} = cN_c + \overline{q}(N_q - 1) + \gamma B N_\gamma \tag{2B.12}$$

2B.4.3 Choice of Unit Weight, *c*

The choice of which soil parameters to use for a given design is primarily related to the soil's hydraulic conductivity in relation to the rate of foundation loading.

2B.4.3.1 Cohesive (Clay) Soils

For cohesive soils, the assumption is usually made that the loads are applied much more rapidly than the soil can drain. Consequently: $\phi = 0$ and $N_c = 5.14$; $N_q = 1.00$ and $N_\gamma = 0$.

$$q_{\text{ult}} = cN_c \tag{2B.13}$$

2B.4.3.2 Cohesionless (Drained)

For soils that will drain rapidly, the use of effective stress parameters is suggested, i.e., use ϕ and $c = 0$.

$$q_{\text{ult}} = \overline{q}(N_q - 1) + \gamma B N_\gamma \tag{2B.14}$$

2B.4.4 Influence of Water

The presence of a water table affects equation (2B.9), depending upon where the water table lies with respect to the bearing surface of the footing (Fig. 2B.6). Consider the following conditions.

2B.4.4.1 Case I: $D_w = D_f$

In this case, the maximum level of the water table is at the base of the footing, i.e., $D_w = D_f$. Therefore, the soil above the footing base is at its natural moisture content, whereas the soil below the footing base is submerged. Therefore, the unit weight in the N_γ term should be the submerged unit weight and the unit weight in the N_q term should be the total unit weight.

$$q_{\text{ult}} = cN_c + (\gamma_{\text{sat}} D_f)N_q + \gamma' B N_\gamma \tag{2B.15}$$

where γ'= submerged weight.

2B.4.4.2 Case II: $D_w = 0$

In this case, the water table is at the ground surface. Therefore, the unit weight in both terms is the submerged unit weight.

$$q_{\text{ult}} = cN_c + (\gamma' D_f)N_q + \gamma' B N_\gamma \tag{2B.16}$$

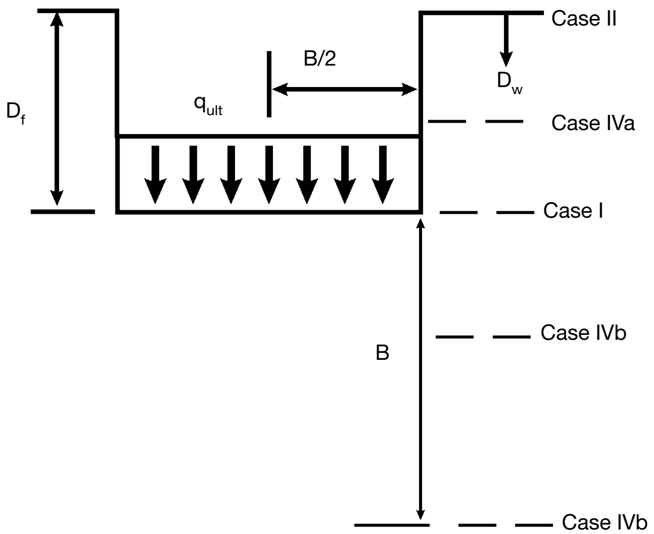

FIGURE 2B.6 Influence of water table on bearing capacity.

2B.4.4.3 Case III: $D_w = D_f + B$

In this case, the maximum height of the water table is a distance B below the base of the footing, D_f. In this case, the unit weight in both terms is the total unit weight.

$$q_{\text{ult}} = cN_c + (\gamma)N_q + \gamma BN_\gamma \tag{2B.17}$$

2B.4.4.4 Case IV: Intermediate Cases

For cases where $0 < D_w < D_f$, the unit weight in N_γ term is the submerged unit weight. A weighted average is used for the unit weight in the N_q term:

$$\gamma = \gamma' + \left[\frac{D_w}{D_f}\right][\gamma - \gamma'] \tag{2B.18}$$

For cases where $D_f < D_w < [D_f + B]$, the unit weight in the N_γ term is

$$\gamma' + \left[\frac{D_w - D_f}{B}\right][\gamma - \gamma'] \tag{2B.19}$$

The unit weight in the N_q term is the total unit weight.

2B.4.4.5 Approximate Procedure

Peck, Hanson, and Thornburn[2] developed a procedure to adjust the bearing capacity to take into account the presence of the water table. In their procedure, first solve for q_{ult} for no water table within $D_w = D_f + B$. Then set $q = C_w q_{\text{ult}}$, where

$$C_w = 0.5 + 0.5\frac{D_w}{D_f + B} \tag{2B.20}$$

2B.4.5 General Bearing Capacity Equation

Equation (2B.9) can be generalized to take various geometric and loading factors into account:

$$q_u = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \tfrac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma \tag{2B.21}$$

where

$$s_c, s_q, s_\gamma = \text{shape factors}$$
$$d_c, d_q, d_\gamma = \text{depth factors}$$
$$i_c, i_q, i_\gamma = \text{load inclination factors}$$

These factors are discussed below.

2B.4.5.1 Bearing Capacity Factors

Various investigators have published solutions to the bearing capacity equation. Each has provided equations for the bearing capacity factors. A selection of the most useful factors is presented below.

From Meyerhof[3]:

$$N_c = \tan^2\left(45 + \frac{\phi}{2}\right)e^{\pi \tan\phi} \tag{2B.22}$$

$$N_c = (N_q - 1)\cot\phi \tag{2B.23}$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi) \tag{2B.24}$$

From Hansen[4]:

$$N_\gamma = 1.5(N_q - 1)\tan\phi \tag{2B.25}$$

From Vesic[5]:

$$N_\gamma = 2(N_q + 1)\tan\phi \tag{2B.26}$$

2B.4.5.2 Shape Factors

Although the derivation of the bearing capacity equation was in two dimensions, it is obvious that the problem is three dimensional. Therefore, the relationship between the width, B, and the length, L, on the bearing capacity is of great importance. In general, this relationship is given by application of shape factors applied to the appropriate component of the general bearing capacity equation. Listed below are shape factors reported by various authors.

$$\begin{aligned} s_c &= 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)(\phi > 0) \\ s_{c'} &= 0.2\frac{B}{L}(\phi = 0) \\ s_c &= 1 + 0.2K_p\frac{B}{L} \end{aligned} \tag{2B.27}$$

$$s_c = 1.0 \text{ strip} = 1.3 \text{ round} = 1.3 \text{ square}$$

$$s_q = 1 + \left(\frac{B}{L}\right) \tan \phi$$

$$s_q = 1 + 0.1K_p \frac{B}{L} (\phi > 10°) \tag{2B.28}$$

$$s_q = 1 (\phi = 0)$$

$$s_\gamma = 1 - 0.4\left(\frac{B}{L}\right)$$

$$s_\gamma = 1 + 0.1K_p \frac{B}{L} (\phi > 10°)$$

$$s_\gamma = 1 (\phi = 0) \tag{2B.29}$$

$$s_\gamma = 1 \text{ (strip)} = 0.6 \text{ (round)} = 0.8 \text{ (square)}$$

2B.4.5.3 Depth Factors

The depth of placement of the shallow footing below the surrounding ground surface also has a significant impact on the footing's bearing capacity. In particular, this depth influences the length of the failure surface available to resist movement. These factors are usually presented in a nondimensional format, similar to that of shape factors. A listing of the most useful depth factors is given below.

$$d_c = 1 + 0.4\left(\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} \leq 1\right)$$

$$d_c = 1 + 0.4\left(\tan^{-1}\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} > 1\right)$$

$$d_c = 0.4\frac{D}{B} \text{ or } 0.4\left(\tan^{-1}\frac{D}{B}\right) \text{ for } \phi = 0 \tag{2B.30}$$

$$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$$

$$d_q = 1 + 2 \tan \phi(1 - \sin \phi)^2\left(\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} \leq 1\right)$$

$$d_q = 1 + 2 \tan \phi(1 - \sin \phi)^2\left(\tan^{-1}\frac{D_f}{B}\right) \text{ for } \left(\frac{D_f}{B} > 1\right) \tag{2B.31}$$

$$d_q = 1 + 0.1\sqrt{K_p}\frac{D_f}{B} \text{ for } (\phi > 10°)$$

$$d_q = 1 \text{ for } (\phi = 0)$$

$$d_\gamma = 1 \text{ Hansen or Vesic}$$

$$d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D_f}{B} \text{ for } (\phi > 10°) \text{ Meyerhof} \qquad (2B.32)$$

$$d_\gamma = 1 \text{ for } (\phi = 0) \text{ Meyerof}$$

Note that the factor $\tan^{-1}[D_f/B]$ is in radians.

2B.4.5.4 Inclination Factors

Inclined loading significantly reduces the capacity of a shallow foundation. This reduction is determined by the use of inclination factors.

$$i_\gamma = \left(1 - \frac{\beta}{\phi}\right)^2$$

$$i_\gamma = \left(1 - \frac{0.7H}{V + A_f c_a \cot\phi}\right)^5 \text{ for } (\eta = 0)$$

$$i_\gamma = \left(1 - \frac{(0.7 - \eta°/450)H}{V + A_f c_a \cot\phi}\right)^5 \text{ for } (\eta > 0) \qquad (2B.33)$$

$$i_\gamma = \left(1 - \frac{H}{V + A_f c_a \cot\phi}\right)^{m+1}$$

$$\eta = \text{tilt angle from horizontal with (+) upward}$$

$$i_c = i_q = \left(1 - \frac{\beta}{90°}\right)^2$$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1}$$

$$i_c = 0.5 - 0.5\sqrt{1 - \frac{H}{A_f c_a}} \text{ for } (\phi = 0)$$

$$i_c = 1 - \sqrt{\frac{H}{A_f c_a}} \text{ for } (\phi = 0)$$

$$A_f = B' x L' \qquad (2B.34)$$

$$c_a = \text{adhesion}$$

$$H = \text{horizontal component of footing load}$$

$$V = \text{total load on footing}$$

$$m = m_B = \frac{2 + B/L}{1 + B/L} \text{ for } (H \text{ parallel to } B)$$

$$m = m_L = \frac{2 + L/B}{1 + L/B} \text{ for } (H \text{ parallel to } L)$$

where β = inclination of the load on the foundation with respect to the vertical (Figure 2B.7).

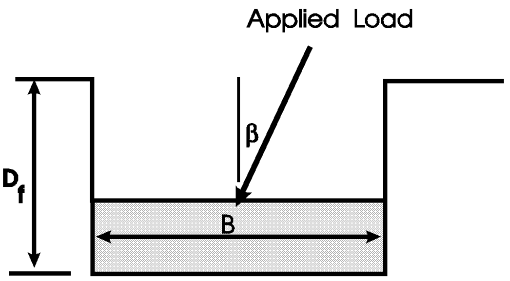

FIGURE 2B.7 Inclined loading.

2B.4.6 Eccentrically Loaded Foundations

Foundations are often subjected to moments in addition to the vertical load (Fig. 2B.8).

$$q_{\max} = \frac{Q}{BL} + \frac{6M}{B^2L}$$
$$q_{\min} = \frac{Q}{BL} - \frac{6M}{B^2L} \qquad (2B.35)$$

where Q = total vertical load
M = moment on the foundation

$$e_x = \frac{M_x}{Q};\ e_y = \frac{M_y}{Q}$$

$$q_{\max} = \left(\frac{Q}{BL}\right)\left(1 + \frac{6e}{B}\right) \qquad (2A.36)$$

$$q_{\min} = \left(\frac{Q}{BL}\right)\left(1 - \frac{6e}{B}\right)$$

2B.4.6.1 Meyerhof's Effective Area Method

When $e = B/6$, $q_{\min} = 0$. For $e > B/6$m $q_{\min}$ is negative, i.e., tension will develop. Since soil cannot take tension, there will be a separation between the foundation and the soil underlying it. Therefore

$$q_{\max} = \frac{4Q}{3L(B - 2e)} \qquad (2B.37)$$

Determine the effective dimensions of the foundation as:

$$B' = \text{effective width} = B - 2e$$
$$L' = \text{effective length} = L.$$

If the eccentricity is in the direction of the length of the foundation, $L' = L - 2e$.

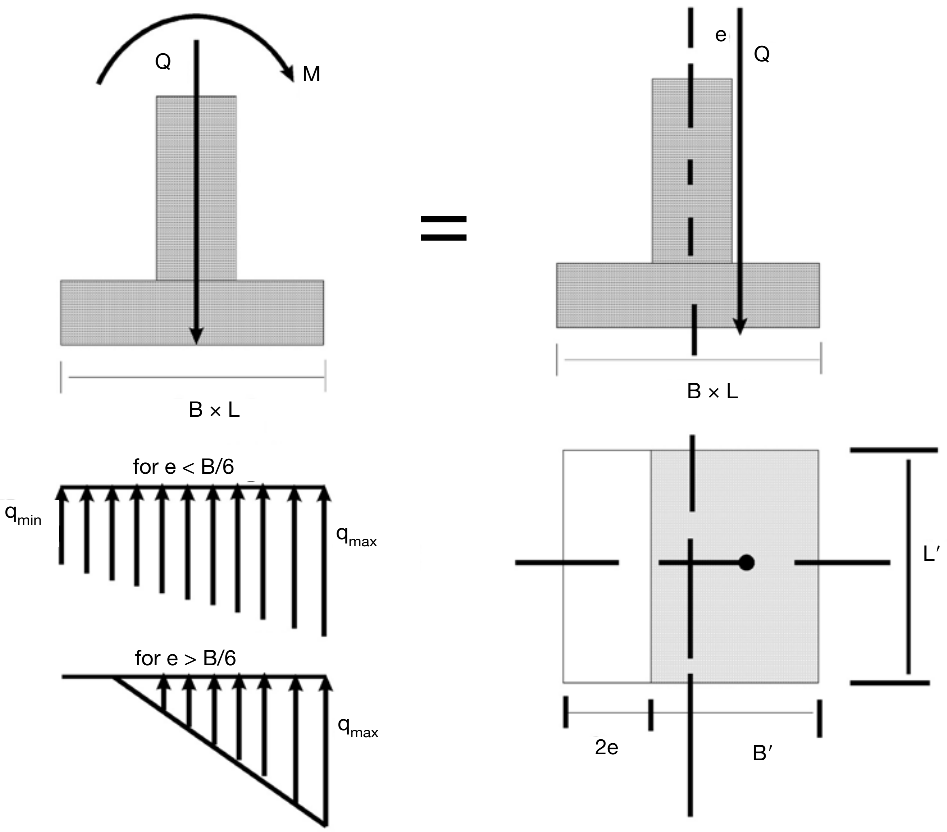

FIGURE 2B.8 Eccentrically loaded footing.

Use the general equation for bearing capacity except substitute B' for B. *Do not replace B with* B' for calculation of the depth factors. The total ultimate load that the foundation can sustain is:

$$Q_{ult} = q_u(B')(L') \tag{2A.38}$$

The factor of safety against bearing capacity failure is:

$$FS = \frac{Q_{ult}}{Q}$$

Bowles reduced the Meyerhof method such that

$$q_{ult} = q_{ult}R_e$$

$$R_e = 1 - 2\frac{e}{B} \text{ (cohesive soil)} \tag{2B.39}$$

$$R_e = 1 - \left(\frac{e}{B}\right)^{1/2} \left(\text{cohesionless soil and } 0 < \frac{e}{B} < 0.3\right)$$

where q_{ult} = ultimate bearing capacity for concentric loading.

As you can see, eccentricity decreases the bearing capacity of a footing. It is advantageous to place the foundation columns off center, as shown below. This, in effect, produces a centrally loaded foundation with uniformly distributed pressure.

2B.5 FACTOR OF SAFETY

Because of the inherent uncertainty of the bearing capacity analysis, the usual practice is to reduce the applied stress from the foundation load by some arbitrary factor. This reduction is usually presented as a factor of safety. The reduced bearing capacity is known as the allowable bearing capacity, or allowable foundation stress q_{allow}:

$$q_{\text{allow}} = \frac{q_{\text{ult}}}{\text{F.S.}} \tag{2B.40}$$

where F.S. is the factor of safety. In general, the factor of safety runs from 2 to 3.

2B.6 BEARING CAPACITY OF MAT

The ultimate bearing capacity of mat foundations is calculated using equation (2B.41).

$$q_u = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \frac{1}{2}\gamma B\, N_\gamma s_\gamma d_\gamma i_\gamma \tag{2B.41}$$

However, the great advantage of mat foundations is that by excavating below the ground surface for the placement of the mat, the net allowable applied load from the structure is increased. If Q is the total of the dead and live loads applied to the base of the mat, then

$$q_{(\text{net applied})} = \frac{Q}{A} - \gamma D_f \tag{2B.42}$$

where A is the area of the mat.

It is possible to place the footing at such a depth D_{crit} such that the net applied load is zero, i.e., let

$$\begin{aligned} q_{(\text{net applied})} &= 0 = \frac{Q}{A} - \gamma D_{(\text{critical})} \\ D_{(\text{critical})} &= \frac{Q}{A\gamma} \end{aligned} \tag{2B.43}$$

For this condition, the factor of safety is:

$$F.S. = \frac{q_{\text{ult}}}{q_{(\text{net allowable})}} = \frac{q_{\text{ult}}}{0} = \infty \qquad (2B.44)$$

2B.7 SETTLEMENT OF SHALLOW FOUNDATIONS

Although the analysis and design of foundations usually begins with the study of the bearing capacity of the foundation–soil system, in general, the settlement of the foundation controls the design.

2B.7.1 Allowable Settlements

The amount of settlement that a foundation can tolerate is called the allowable settlement. the magnitude of this settlement depends upon it mode.

2B.7.1.1 Uniform Settlement

A structure that has undergone uniform settlement is one where all points within the structure have moved vertically the same amount (Figure 2B.9a). This type of settlement does not result in struc-

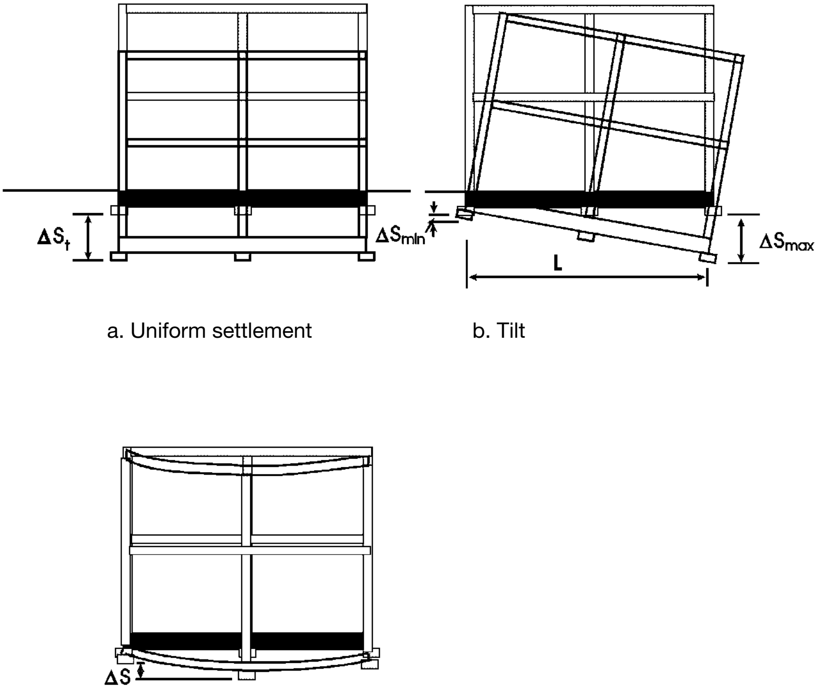

FIGURE 2B.9 Types of foundation settlement.

tural damage if it is constant across whole structure. However, there will be problems with appurtenances such as with pipes, entrance-ways, etc.

2B.7.1.2 Tilt

Tilt is usually measured by its angular distortion (Figure 2B.9):

$$\text{angular distortion} = \frac{s_{max} - s_{min}}{L} \tag{2B.45}$$

The amount of tilt that a structure can tolerate is a function of many factors, including the size and type of construction. The Leaning Tower of Pisa is currently at about 10% tilt and is still standing. However, the Campenella in the Plaza San Marcos in Venice collapsed when it reach a 0.8% tilt. Tilt is visible at about 1/250 or 0.4%

2B.7.1.3 Differential (Distortion) Settlement

If s_{max} is the maximum total settlement anywhere in the structure then Δs_{max} is the maximum difference in total deformations between adjacent foundations. This is called differential settlement. Distortion is then defined as: $\Delta s_{max}/L$.

Field evidence indicates that architectural damage occurs when $\Delta s_{max}/L = 1/300$ and structural damage occurs when $\Delta s_{max}/L = 1/150$.

2B.7.1.4 Maximum Allowable Settlement

In general, foundations are limited to a specified amount of settlement. This settlement is called the design or maximum allowable settlement. For isolated foundations that support individual columns or small groups of columns on clay:

$$\frac{\Delta s_{max}}{L} = \frac{1}{1200} s_{max} \tag{2B.46}$$

Since $\Delta s_{max}/L = 1/300$, $s_{max} = 4$ in (10 cm).

For isolated foundations that support individual columns or small groups of columns on sand:

$$\frac{\Delta s_{max}}{L} = \frac{1}{600} s_{max} \tag{2B.47}$$

Since $\Delta s_{max}/L = 1/300$, $s_{max} = 2$ inches (5 cm). Therefore we must design for total settlements of isolated foundations less than 2–4 in (5–10 cm).

2B.7.1.5 Allowable Settlement of Mat (Raft) Foundations

There are few data available that document allowable settlement for raft foundations. Therefore only minor problems probably occur.

2B.7.2 Elastic (Immediate) Settlement

Immediate settlements occur as the load is applied to the soil. The soil particle matrix distorts and the soil voids are compressed. If the soil voids contain air, or if the permeability of the soil matrix is high, then the volume of the voids decreases, thereby contributing to the settlement. Since sands and gravels are highly conductive, almost all of the settlement of foundations on sands and gravels can be classified as immediate. Clays, on the other hand, have very low hydraulic conductivity; hence, if they are saturated, the immediate deformation is usually quite small and is limited to structural distortion of the soil fabric.

2B.7.2.1 Key Variables in Elastic Settlements

The magnitude of settlement is inversely proportional to the strength of the soil. Factors affecting the soil strength are relative density, embedment of the foundation, and the effect of groundwater. Relative density is measured in the field using the standard penetration test, the cone penetrometer, or other devices.

The relationship between settlement and footing width was described by Terzaghi and Peck[6]. For the same load on the same soil, the settlement s is related to the square of the footing width B through the settlement of a 1 ft (0.3 m) square plate s_1 by:

$$s = s_1\left[\frac{2B}{B+1}\right]^2 \tag{2B.48}$$

The magnitude of settlement is also directly proportional to the magnitude of the applied load up to the allowable bearing pressure, with all else constant.

2B.7.2.2 Settlement Models

Many techniques are presented in the literature for predicting the settlement of shallow foundations on sand. Depending upon which method is used, this calculation can be a very simple one or it can be moderately complex, and the resulting predictions can differ greatly. A recent publication by the Corps of Engineers, Waterways Experiment Station[7] reported on fifteen methods. Most of the methods can be placed within one of two categories: some are modeled after the Terzaghi and Peck[6] (1948) bearing capacity and settlement–footing width relationship, and others are modeled after elasticity methods. A few methods combine some aspects of both. The backgrounds for both the Terzaghi–Peck-based settlement methods and elastic-based settlement methods are given below.

2B.7.2.2.1 Terzaghi–Peck-based Settlement. Terzaghi and Peck[6] developed the well-known design chart, Figure 2B.10, for estimating allowable bearing pressures for shallow foundations on sand using standard penetration blow count and footing width. These design curves correspond to a maximum footing settlement of 1 in (2.5 cm) and total differential settlement of ¾ in

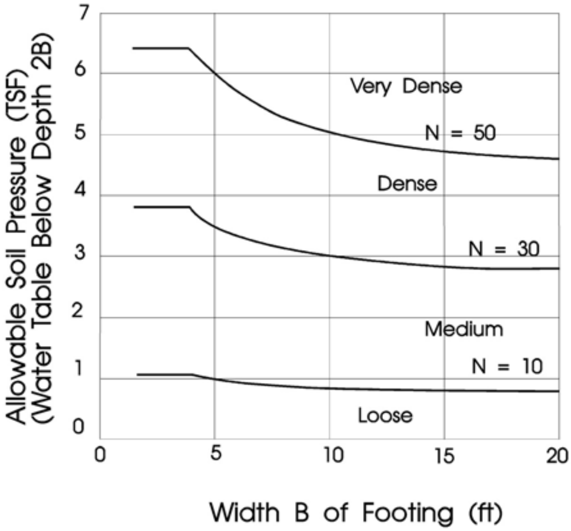

FIGURE 2B.10 Terzaghi and Peck design chart for q_{allow}.

(1.9 cm). Data were interpreted conservatively in the development of this chart. History has proven that these values are very conservative. Modification to these values for less conservatism have been made by many. A general expression for this relations is

$$s = C\left[\frac{q}{N}\right]\left[\frac{B}{B+1}\right]^2 \tag{2B.49}$$

where s = settlement
q = net applied load
B = footing width
N = blowcount
C = empirical constant determined by observation and or experimentation

The Terzaghi–Peck chart gives $C = 8$ for footings less than 4 ft (1.2 m), and $C = 12$ for footings greater than 4 ft (1.2 m) in width.

2B.7.2.2.2 Elastic Soil Settlement. Soil is often treated as an elastic medium, linear or nonlinear, to which the elastic theory assumptions and principles of stress and strain are applied. Settlement computations of this form use the elastic properties of Poisson's ratio and Young's modulus to represent the soil. A general expression for the elastic settlement relation is:

$$s = \frac{qBI\mu}{E} \tag{2B.50}$$

where μ = Poissons ratio
E = elastic modulus
I = influence factor based on footing shape depth and extent of elastic region

One main difference between the Terzaghi–Peck model and the elastic model is the relationship between footing width and settlement. The elastic theory models a linear relation between settlement and footing width, while Terzaghi and Peck's work shows this to be a nonlinear relation. Elastic theory settlement methods can account for this nonlinear relationship through an appropriate use of the elastic or compressibility modulus.

2B.7.2.2.3 Standard Penetration Test. The relative density of the soil is the major factor that controls the settlement of foundations on cohesionless soil. The standard penetration test (SPT) is the most widely used test in the United States for indirectly determining the relative density of a sand. A description of the test can be found in Bowles[1]. With all other factors being the same, the larger the blow count (N), the less the settlement. However, a number of factors significantly effect the blow count. The overburden pressure dramatically impacts the SPT value. In a homogeneous deposit where relative density and friction angle are constant with depth, blow counts increase with depth due to the increasing confining pressures with depth. Therefore, each measured SPT value should be corrected for the influence of its corresponding overburden pressure. There are many techniques available to correct the SPT value for overburden pressure. In general, all of the techniques have the form of

$$N_c = C_N N \tag{2B.51}$$

where N_c = corrected SPT value
C_N = correction factor based on overburden pressure

Blow count correction factors for overburden developed by various authors are given in Table 2B.1. Each of the equations normalizes N to a standard reference overburden pressure. Typically, this is 1 tsf (96 kN/m2). However, Peck and Bazara[12] normalize to 0.75 tsf (72 kN/m^2) and Teng[10]

TABLE 2B.1 Overburden Correction Factors

Reference	Equation for C_N	Units for p_o'
Skempton[8]	fine to medium sand: $2/(1 + p_o')$ coarse, dense sand: $3/(2 + p_o')$ overconsolidated: $1.7/(0.7 + p_o')$	p_o' = effective overburden pressure
		tsf
Peck, Hanson & Thornburn[2]	fine sand: $0.77 \log(20/p_o')$	tsf
Bazaraa[9]	$p_o' < 1.5$ ksf: $4/(1 + 2p_o')$ $p_o' > 1.5$ ksf: $4/(3.25 + 0.5p_o')$	ksf
Teng[10]	$50/(p_o' + 10)$	psi
Liao and Whitman[11]	$(1/p_o')^{0.5}$	tsf

uses 40 psi (276 kN/m^2). Some procedures for computing settlement do no advocate correcting the blow count for overburden but use the blow count values as obtained in the field. Most experiments and theories show that this correction is necessary.

Variations in the borehole diameter, rod length, and hammer type can affect the measured blow counts for identical sands at the same overburden and relative density values. The blow count is directly related to the driving energy of test equipment:

$$E_\epsilon = \frac{1}{2}mv^2 = \frac{1}{2}\frac{W}{g}v^2$$

$$v = (2gh)^{1/2} \tag{2B.52}$$

$$E_\epsilon = \frac{1}{2}\frac{W}{g}(2gh) = Wh$$

where W = weight or mass of hammer
h = height of fall
v = velocity of hammer

The energy ratio E_r is defined as:

$$E_r = \frac{\text{actual hammer energy to sampler, } E_a}{\text{input energy, } E_{in}} \times 100 \tag{2B.53}$$

Bowles suggests that the energy should be adjusted to a standard energy ratio of 70 (E_{70}) and that the equation for the standard corrected blow count be given as

$$N_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \tag{2B.54}$$

where the η factors can be found in Table 2B.2. Each of the factors correct the field blow counts for differences in hammers, rod length, sampler differences, and borehole diameter differences.

A footing placed below the ground surface will settle less than a footing at the surface. The

TABLE 2B.2 Blow Count Adjustment Factors, η_i

Hammer adjustment factor, η_1			
Average energy ratio E_r			
Donut		Safety	
R–P	Trip	R-P	Trip/Auto
5	—	70–80	80–100
R–P = Rope-pulley or cathead: $\eta_1 = E_r/E_{rb}$.			
For U.S. trip/auto $w/E_r = 80$: $\eta_1 = 80/70 = 1.14$.			
Rod Length correction factor, η_2			
Length > 10 m			$\eta_2 = 1.00$
6–10 m			$= 0.80$
4–6 m			$= 0.85$
0–4 m			$= 0.75$
Sampler correction factor, η_3			
Without liner			$\eta_3 = 1.00$
With liner:			
Dense sand, clay			$= 0.80$
Loose sand			$= 0.90$
Borehole diameter correction factor, η_4			
Hole diameter:			
60–120 mm			$\eta_4 = 1.00$
150 mm			$= 1.05$
200 mm			$= 1.15$

depth correction factor reduces the calculated settlement to account for the increase in bearing capacity achieved by embedment. The embedment correction equations by the various authors are given in Table 2B.3.

2B.7.2.3 Settlement Computing Methods

Several of the more prominent methods of computing elastic settlements are presented in the following paragraphs. Other methods can be extracted from published literature.

TABLE 2B.3 Embedment Correction Factors

Reference	Equation for Embedment Correction Factor, C_D
Terzaghi and Peck[13] (1967)	$C_D = 1 - 0.25(D_f/B)$
Schultz and Sherif[14] (1973)	$C_D = 1/[1 + 0.4(D_f/B)]$
D'Appoplonia, et al.[14]	$C_D = 0.729 - 0.484 \log (D_f/B) - 0.224[\log(D_f/B)]^2$
Bowles[1] (1977)	$C_D = 1/[1 + 0.33(D_f/B)]$
Teng[10] (1962)	$C_D = 1/[1 + (D_f/B)]$
Bazaraa[9] (1969)	$C_D = 1 - 0.4[\gamma D_f/q]^{0.5}$
Schmertmann[16]	$C_D = 1 - 0.5[\gamma D_f/(q - D_f]$

Terms: D_f = foundation depth, B = foundation width, q = loading pressure.

2B.7.2.3.1 Terzaghi and Peck[6,13]. This method is based on the bearing capacity charts given in Figure 2B.10. The equations shown below are given by Meyerhof (1956)[3]. The chart is used to determine the allowable bearing capacity for a range of footing widths and SPT blow count values with maximum settlement not to exceed 1 in (2.5 cm) and differential settlement not to exceed 3/4 in (1.9 cm). Their settlement expression is:

$$s = \frac{8q}{N}(C_W C_D) \qquad \text{for } B \leq 4 \text{ ft (1.2m)}$$

$$s = \frac{12q}{N}\left[\frac{B}{B+1}\right]^2 (C_W C_D) \qquad \text{for } B \geq 4 \text{ ft (1.2m)} \tag{2B.55}$$

$$s = \frac{12q}{N}(C_W C_D) \qquad \text{for rafts}$$

Their correction factors for water are:

$$C_W = 2 - \left[\frac{D_W}{2B}\right] \leq 2.0 \text{ (for surface footings)}$$

$$C_W = 2 - 0.5\left[\frac{D_W}{2B}\right] \leq 2.0 \text{ (for sumerged, embedded footing; } D_W \leq D_f) \tag{2B.56}$$

And for depth:

$$C_D = 1 - 0.25\left[\frac{D_f}{B}\right]$$

For blow count use the measured SPT blow count value. If the sand is saturated, dense, and very fine or silty, correct the blow count by:

$$N_c = 15 + 0.5(N - 15) \qquad \text{for } N > 15 \tag{2B.57}$$

2B.7.2.3.2 Teng[10]. Teng's method for computing settlement is an interpretation of the Terzaghi and Peck bearing capacity chart. Teng includes corrections for depth of embedment, the presence of water, and the blow count. The settlement expression is:

$$\Delta s = \frac{q_o}{720(N_c - 3)}\left[\frac{mH}{B+1}\right]^2 \frac{1}{(C_W)(C_D)} \tag{2B.58}$$

where: q_o = net pressure in psf

The correction factor for water is

$$C_W = 0.5 + 0.5\left[\frac{D_W - D_f}{B}\right] \geq 0.5 \qquad \text{for water at and below } D_f \tag{2B.59}$$

For depth:

$$C_D = 1 + \left[\frac{D_f}{B}\right] \leq 2.0$$

For blow count:

$$N_c = N\left[\frac{50}{p_o{}' + 10}\right]$$

where $p_o{}'$ = effective overburden at median blowcount depth abut $D_f + B/2$, in psi ($\leq$ 40 psi, 276kPa)

2B.7.2.3.3 Peck, Hanson, and Thornburn[2]. This method is based on Terzaghi and Peck settlement method.

$$\Delta s = \frac{q}{0.11 N_c C_w} \qquad \text{for intermediate width footings (> 2 ft, 0.6m)}$$

$$\Delta s = \frac{q}{0.22 N_c C_w} \qquad \text{for rafts} \tag{2B.60}$$

where q is in tsf.

The correction factor for water is

$$C_W = 0.5 + 0.5\left[\frac{D_W}{D_f + B}\right] \qquad \text{for water from 0 to } D_f + B \tag{2B.61}$$

For blow count:

$$N_c = NC_n$$

$$C_n = 0.77\log\left[\frac{20}{p'}\right] \tag{2B.62}$$

where p' = effective overburden pressure for the measured blow count at $D_f + (B/2)$ in tsf (0.25 tsf = 24 kPa).

2B.7.2.3.4 Bowles[17,18]. Bowles' settlement method is based on the Terzaghi and Peck method, but is modified to produce results that are not as conservative. His equations are:

$$\Delta s = \frac{2.5 q_o}{N}\left[\frac{C_W}{C_D}\right] \qquad \text{for } B \leq 4 \text{ ft}$$

$$\Delta s = \frac{4 q_o}{N}\left[\frac{B}{B+1}\right]^2\left[\frac{C_W}{C_D}\right] \qquad \text{for } B \geq 4 \text{ ft} \tag{2B.63}$$

$$\Delta s = \frac{4 q_o}{N}\left[\frac{C_W}{C_D}\right] \qquad \text{for mats}$$

where q is in kips/sf, N is measured in the field, and the settlement is in inches.

The correction factor for water is:

$$C_W = 2 - \left[\frac{D_W}{D_f + B}\right] \leq 2.0 \text{ and } \geq 1.0 \tag{2B.64}$$

The correction factor for depth is:

$$C_D = 1 + 0.33\left[\frac{D_f}{B}\right] \leq 1.33 \tag{2B.65}$$

Therefore, the settlement can be computed from:

$$\Delta s = \frac{2.5q_o}{N}\left[\frac{C_W}{C_D}\right] \qquad \text{for } B \leq 4 \text{ ft}$$

$$\Delta s = \frac{4q_o}{N}\left[\frac{B}{B+1}\right]^2\left[\frac{C_W}{C_D}\right] \qquad \text{for } B \geq 4 \text{ ft} \tag{2B.66}$$

$$\Delta s = \frac{4q_o}{N}\left[\frac{C_W}{C_D}\right] \qquad \text{for mats}$$

2B.7.2.3.5 Elastic Theory. Settlement computed by elastic theory uses elastic parameters to model a homogeneous, linearly elastic medium. The elastic modulus of a soil depends upon confinement and is assumed in elastic theory to be constant with depth. For uniform saturated cohesive soils, this assumption is usually valid. For cohesionless soils, elastic methods can be inappropriate because the modulus often increases with depth. However, the immediate settlement of sand is often considered to be elastic within a small strain range.

The equations are based on the theory of elasticity and are for settlement at the surface of a semi-infinite, homogeneous half-space. The equations are

$$\Delta s = \frac{q_o B'}{E_s}(1-\mu^2)\left[I_1 + \frac{(1+2\mu)}{(1-\mu)}I_2\right]I_F \tag{2B.67}$$

$$I_1 = \frac{1}{\pi}\left[M \ln \frac{(1+\sqrt{M^2+1})\sqrt{M^2+N^2}}{M(1+\sqrt{M^2+N^2+1})} + \ln \frac{(M+\sqrt{M^2+1})\,\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right] \tag{2B.68}$$

$$I_2 = \frac{N}{2\pi}\tan^{-1}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right)(\tan^{-1} \text{ in rad}) \tag{2B.69}$$

where $M = (L'/B')$; $H = (H/B')$

For the center influence factor,

$$B' = \frac{B}{2}; \quad L' = \frac{L}{2}$$

For the corner influence factor, $B' = B5$; $L' = L$.

The correction factors are from Das[19] (Figure 2B.11). The influence factor I_s is defined as

$$I_s = I_1 + \frac{1-2\mu}{1-\mu}I_2 \tag{2B.70}$$

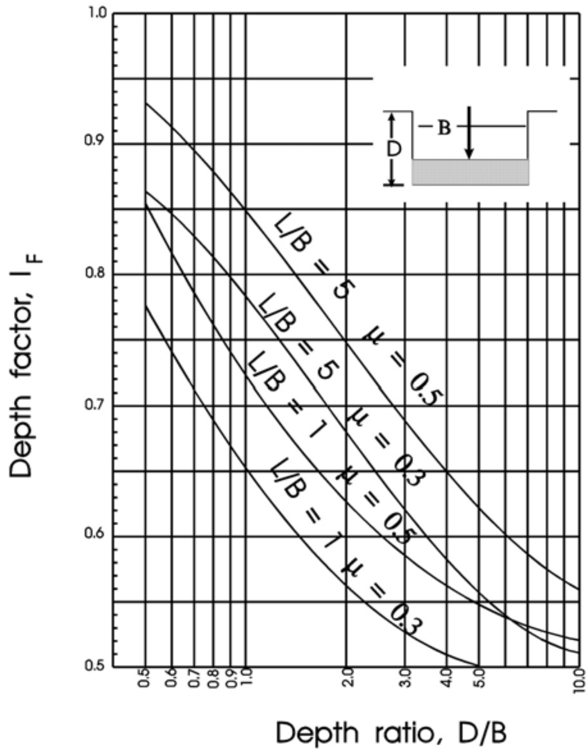

FIGURE 2B.11 Influence factors for footing at depth *D*. Use actual footing width and depth dimensions (Das[19]).

Therefore, Equation (2B.67) can be written as:

$$\Delta s = q_o B' \frac{1 - \mu^2}{E_s} I_s I_F \tag{2B.71}$$

For rigid footings, the value of I_s should be reduced by 7%, i.e., $I_{sr} = 0.93\ I_s$. Poisson's ratio, μ, can be determined from Table 2B.4.

Bowles suggests the following procedure:

1. Make the best estimate of q_o.
2. Convert round footings to an equivalent square.
3. Determine the point where the settlement is to be computed and divide the base so the point is at the corner or common corner of the contributing rectangles.
4. Note that the stratum depth actually causing settlement is not at H/B but is either:
   a. Depth $z = 5B$ (B = least total lateral dimension of base), or
   b. Depth to where a hard stratum is encountered. Take Hard as that where E_s in the hard layer is about $10E_s$ of adjacent layer. Table 2B.5 can be used for approximate values. Table 2B.6 gives equations for E_s as functions of cone or standard penetration test values.

TABLE 2B.4 Range of values for Poisson's Ratio*

Soil	Poisson's Ratio
Loose Sand	0.2–0.4
Medium Sand	0.25–0.4
Dense Sand	0.3–0.45
Silty Sand	0.2–0.4
Soft Clay	0.15–0.25
Medium Clay	0.2–0.5

*After Das.[19]

TABLE 2B.5 Range of Elastic Modulus, E_s*

Soil	E_s psi(kPa)
Soft Clay	250–500(1725–3450)
Hard Clay	850–2,000(5860–13,800)
Loose Sand	1,500–4,000(10,350–27,600)
Dense Sand	5,000–10,000(34,400–69,000)

*After Das.[19]

TABLE 2B.6 Equations for E_s from SPT and CPT

SOIL	SPT (kPa)	CPT (units of q_c)
Sand	$E_s = 500(N + 15)$	$E_s = (2 \text{ to } 4)q_c$
	$E_s = 18{,}000 + 750N$	$E_s = 2(1 + D_r^2)q_c$
	$E_s = (15{,}000 \text{ to } 22{,}000)\ln N$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
Gravelly sand	$E_s = 1{,}200(N + 6)$	
Soft clay		$E_s = (6 \text{ to } 8)q_c$

5. Compute H/B' ratio. For a depth $H = z = 5B$ and for the center of the base we have $H/B' = 5B/0.5B = 10$. For a corner $5B/B = 5$.
6. Obtain I_1 and I_2 with the best estimate for μ and compute I_s.
7. Determine I_F from Figure 2B.11.
8. Obtain the weighted average E_s in the depth $a = H$ using

$$E_{s(av)} = \frac{H_1E_{s1} + H_2E_{s2} + \cdots + H_nE_{sn}}{H} \tag{2B.72}$$

2B.7.2.3.6 Das[19] Elastic Settlement of Foundations on Saturated Clay (μ = 0.5). Das computes the settlement of a foundation on saturated clay using:

$$S_e = A_1A_2\frac{q_oB}{E_s} \tag{2B-73}$$

The coefficients A_1 and A_2 are found in Figure 2B.12 and Figure 2B.13, respectively.

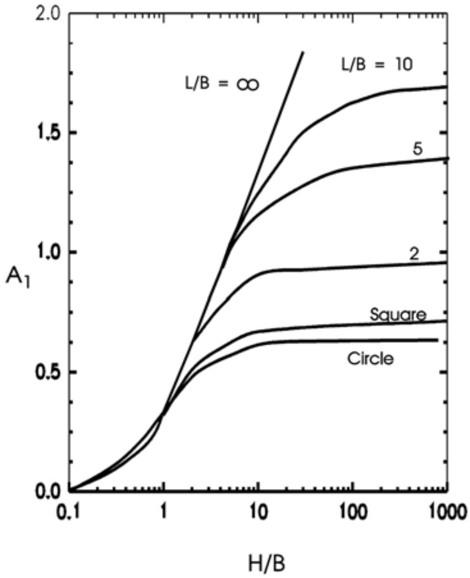

FIGURE 2B.12 Values of A_1.

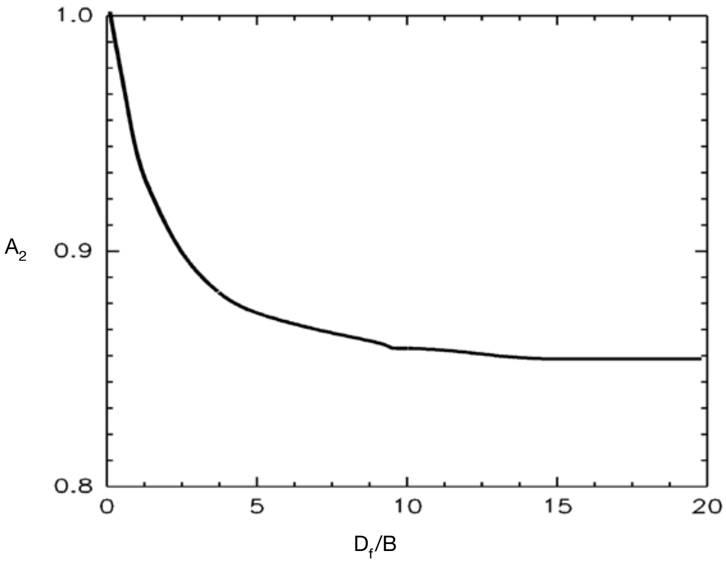

FIGURE 2B.13 Values of A_2.

2B.7.2.3.7 Schmertmann[16]. Schmertmann proposes calculating total settlement by subdividing the compressible stratum and summing the settlements of each sublayer. The sublayer boundaries are defined by changes in the SPT or cone penetrometer (CPT) profile. His equation is

$$\Delta s - C_1 C_2 \Delta q \Sigma \frac{I_z \Delta z}{E_s} \tag{2B.74}$$

The correction factor for embedment is

$$C_1 = 1 - 0.5 \frac{\overline{q}}{q_o - \overline{q}} \tag{2B.75}$$

where $\overline{q}$ = surchage = γD_f.

$$C_2 = 1 + 0.2 \log \frac{t}{0.1} \tag{2B.76}$$

where t = time (> 0.1 years).

The variation of the strain influence factor is given in Figure 2B.14. Note that for square or circular foundations,

$$\begin{gathered} I_z = 0.1 \text{ at } z = 0 \\ I_z = 0.5 \text{ at } z = z_1 = 0.5B \\ I_z = 0 \text{ at } z = z_2 = 2B \end{gathered} \tag{2B.77}$$

Similarly, for foundations with $L/B > 10$,

$$\begin{gathered} I_z = 0.2 \text{ at } z = 0 \\ I_z = 0.5 \text{ at } z = z_1 = B \\ I_z = 0 \text{ at } z = z_2 = 4B \end{gathered} \tag{2B.78}$$

For values of L/B between 1 and 10, necessary interpolations can be made.

This profile is used to determine the elastic modulus as it changes with depth.

If E_s is constant over $2B$ below the footing base, the simplified expression is

$$\Delta s = C_1 C_2 q_o \frac{0.6B}{E_s} \tag{2B.79}$$

2B.7.2.4 Proportioning Footings for Equal Settlement

For clay soils, the usual method is to use the following equation:

$$\frac{\Delta s_1}{\Delta s_2} = \frac{B_{2'}}{B_{1'}} \tag{2B.80}$$

for constant contact pressure. This has proven to work reasonably well.

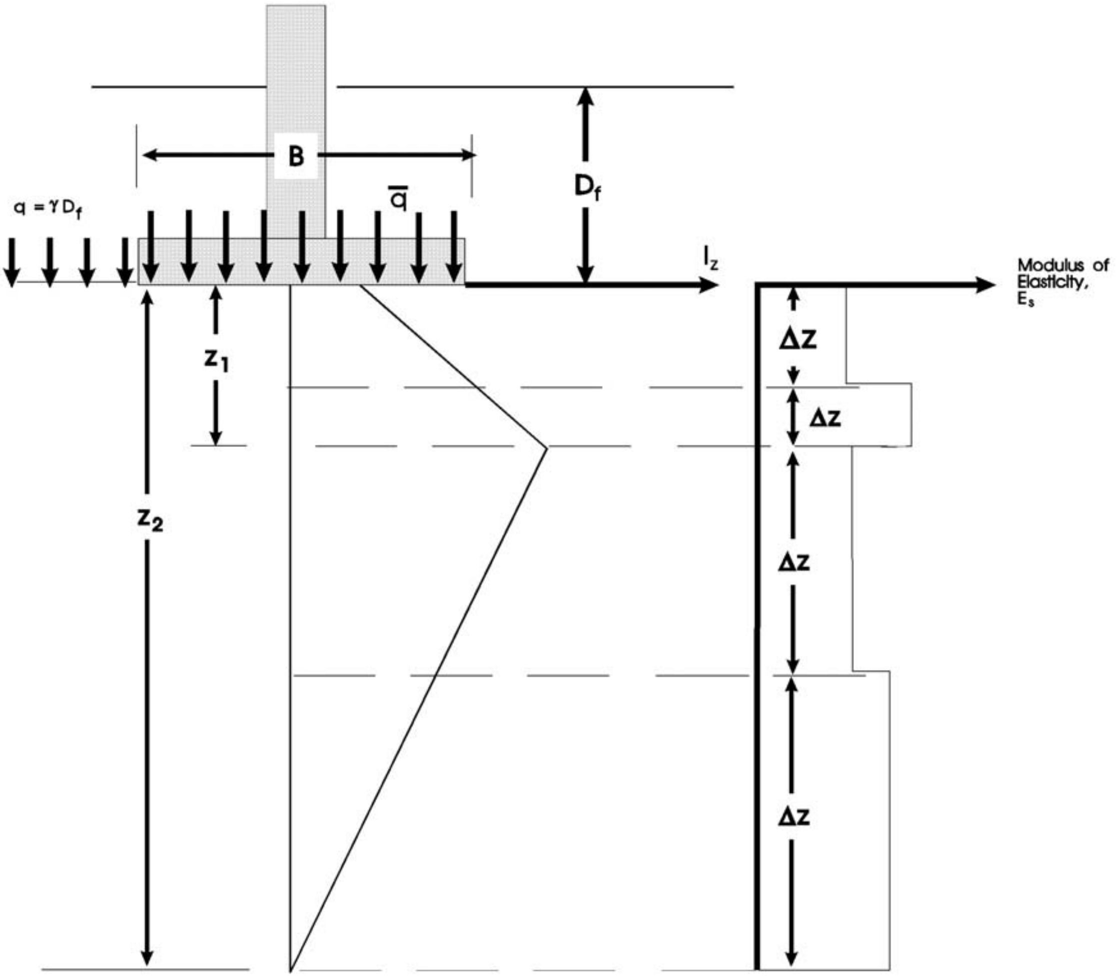

FIGURE 2B.14 Elastic settlement by the strain influence factor.

For sand soils, Bowles recommends

$$\frac{\Delta s_1}{\Delta s_2} = \frac{B_{2'}}{B_{1'}} \frac{I_{s2}}{I_{s1}} \frac{I_{f2}}{I_{f1}} \frac{E_{s'2}}{E_{s'1}} \tag{2B.81}$$

for constant contact pressure.

2B.7.3 Consolidation Settlement of Foundations on Clays

The settlement of a shallow foundation located on cohesive soils is governed by consolidation theory (Section 2A.8).

In order to compute the consolidation settlement of a foundation on clay, the following steps must be done.

1. Determine if the soil is normally consolidated or overconsolidated.
2. Determine the thickness, H, and existing void ratio, e_0, of the consolidating soil layer.
3. Compute the average existing effective stress acting on the consolidating soil layer, p_0.
4. Compute the average increase in stress Δp on the consolidating layer due to the addition of the foundation load.
5. Compute the consolidation settlement using:

For normally consolidated soils:

$$\Delta s = \frac{H}{1 + e_0} C_c \log\left[\frac{\overline{p_0} + \Delta p}{\overline{p_0}}\right] \tag{2B.82}$$

For overconsolidated soils:

$$\Delta s = \frac{H}{1 + e_0} C_r \log\left[\frac{\overline{p_0} + \Delta p}{\overline{p_0}}\right] \tag{2B.83}$$

2B.8 ELASTIC SETTLEMENT OF ECCENTRICALLY LOADED FOUNDATIONS

Whitman and Richart[20] developed a procedure to estimate the settlement of a shallow foundation under eccentric loading. If Q (applied total load) and the eccentricity e are known, then determine the ultimate load Q_{ulte} that the foundation can sustain using the methods for eccentrically loaded foundations previously presented. Determine the factor of safety for the eccentrically loaded foundation as

$$FS = Q_{ulte}/Q = F_1 \tag{2B.84}$$

Next, determine the ultimate load $Q_{ulte=0}$ for the same foundation with $e = 0$:

$$Q_{ulte=0}/F_1 = Q_{e=0} \tag{2B.85}$$

$Q_{e=0}$ is the allowable load for the foundation with a factor of safety $FS = F_1$ for a central loading condition.

For the load $Q_{e=0}$ on the foundation, estimate the settlement by using the techniques presented previously. This settlement is equal to $S_{e=0}$. Calculate S_{e1}, S_{e2}, and t using:

$$S_{e1} = S_{e=0}\left[1 - 2\left(\frac{e}{B}\right)\right]^2 \tag{2B.86}$$

Next, solve for t as

$$t = \tan^{-1}\left[CS_e\left(\frac{e/B}{\sqrt{BL}}\right)\right] \tag{2B.87}$$

where $C = \beta_1\beta_2$

$\beta_1\beta_2$ = factors that depend on the L/B ratio (Fig.2B.15)

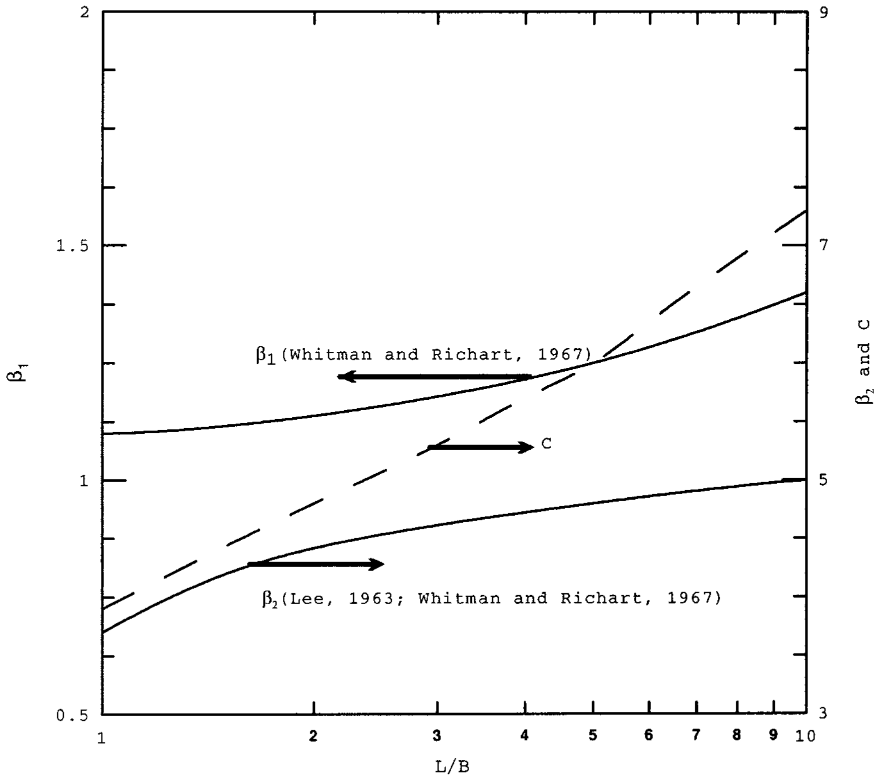

FIGURE 2B.15 Settlement factors for eccentrically loaded foundations.

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P • A • R • T • 3

FUNDAMENTALS OF FOUNDATION CONSTRUCTION AND DESIGN

SECTION 3
CONCRETE

P. BALAGURU

Concrete is one of the basic construction materials; it finds a place in almost all structures. Even in such structures as steel bridges, the deck is quite often made of concrete. Concrete is the preferred and most widely used material for foundation construction. Even if the superstructure is made of steel or wood, the foundation is usually made of concrete. In the case of slab on grade floors, whether industrial, commercial, or residential, concrete is the preferred material.

This section deals with some of the fundamental aspects of concrete. Only the basic information considered necessary for the design and construction engineer is presented. The reader can refer to the literature for more details and in-depth information on a particular aspect. This section deals only with plain concrete. Reinforced concrete is discussed in Part 4.

3.1 CONSTITUENT MATERIALS

Concrete is a composite material made of portland cement (often simply called cement), aggregates, and water. In most cases, additional constituents, called admixtures, are used to improve the properties of fresh and hardened concrete. For example, water-reducing admixtures are often used to improve the workability of fresh concrete without increasing its water content, thus maintaining the strength and durability characteristics of the hardened concrete. The admixtures can be classified broadly as chemical and mineral admixtures.

This section presents basic information with regard to the various constituent materials used in concrete. They are grouped as (1) cement, (2) aggregates, (3) water and water-reducing admixtures, (4) chemical admixtures, and (5) mineral admixtures. Even though water-reducing admixtures are chemical admixtures, they are discussed together with water because of their direct impact on the quantity of water used in the mix and their widespread use in practice.

3.1.1 Cement

Cement, which is the binding ingredient of concrete, is produced by combining lime, silica, and alumina. A small amount of gypsum is added to control the setting time of the cement. Portland cement was first patented by Joseph Aspdin of England in 1824. David Saylor of Coplay, Pennsylvania, was the first to produce portland cement in the United States in 1871. He used vertical kilns that were similar to the ones used for burning lime. The rotary kiln was introduced in 1899. In the 1990s cement production in the United States was in the range of 800 million tons (725 million tonnes); worldwide it reached 5 billion tons (4.5×10^9 tonnes).

Manufacture of Cement

The raw materials for portland cement consist primarily of limestone or some other lime-containing material such as marl, chalk, or shells, and of clay or shale or some other clayey material such as ash or slag. Sometimes other ingredients, such as high-calcium limestone, sandstone, and iron ore, are added to control the chemical composition of the final product. The manufacturing process can be briefly described as follows.

The raw materials are ground into impalpable powder and thoroughly mixed. In the dry process, blending and grinding operations are done in the dry form, and the mixing is primarily accomplished during the grinding phase. In the wet process, water is used to form a slurry. The slurry is often mixed in large vats to obtain a thorough mixing, even though the ingredients have already been mixed during the grinding process. The wet process, which requires about 15% more energy than the dry process, is often chosen for environmental and technological reasons. Continuous quality control measures are used to ascertain the proper chemical composition of the raw material so that the chemical ingredients of the final product will be within the limits specified.

In most cases the slurry is fed into the upper end of a slightly inclined rotary kiln. In some instances part of the water is removed from the slurry before feeding it into the kiln. The length and the diameter of the kilns vary between 60 and 500 ft (18 and 150 m) and between 6 and 15 ft (1.8 and 4.5 in), respectively. The kilns, set at an inclination of about 0.5 in/ft (40 mm/m), rotate between 30 and 90 revolutions per hour, moving the material toward the lower (discharge) end. Heating is usually done by using powdered coal and air. In some instances oil or gas is used instead of coal. The temperature varies along the kiln, reaching a maximum in the range of 2300 to 3450°F (1250 to 1900°C).

As the mix passes through the kiln, various reactions take place, including (1) evaporation of free water, (2) dehydroxylation of clay minerals, (3) crystallization of the products of clay mineral dehydroxylation, (4) decomposition of $CaCO_3$, (5) reaction between $CaCO_3$ (CaO) and aluminosilicates, and (6) liquefaction and formation of cement compounds. The temperature variations are controlled in such a way as to keep the compounds in the molten stage to a minimum. The molten liquid agglomerates into nodules. The nodules, ranging in size from 0.125 to 2 in (3 to 50 mm), are called cement clinkers. These clinkers are dropped off from the kiln.

The clinkers are cooled and ground to a fine powder. About 3 to 5% of gypsum ($CaSO_4{\cdot}2H_2O$) is added during the grinding process to control the setting time of the cement. Addition of gypsum retards the hydration of cement, or increases its setting time. After grinding, the cement is stored in silos.

In the United States, cement can be bought in bulk or in bags containing 94 lb (42.5 kg). It is common to designate concrete mixes as 5, 6, or 7 bag mix, and hence it is useful to remember the weight of the cement in a bag.

Composition

Compounds of four oxides containing lime, silica, alumina, and iron constitute about 9S% of the portland cement clinkers. The other 5% could include magnesia, sodium and potassium oxide, titania, sulfur, phosphorous, and manganese oxide. The major components, namely, tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrate (C_4AF), play important roles in the rate of strength development, the heat of hydration, and the ultimate cementing value. For example, the early strength of hydrated portland cement is higher if the percentage of tricalcium silicate is higher, whereas long-term strengths will be higher with higher percentages of dicalcium silicate.

Types

Various types of cement are produced to suit the various applications. The American Society for Testing and Materials (ASTM) recognizes the following five main types:

Type I — For general use in construction

Type II — For use that requires moderate heat of hydration and exposure to moderate sulfate action

Type III — For use where high early strength is needed

Type IV — For use that requires low heat of hydration

Type V — For use that requires high sulfate resistance

Types 1,11, and III can be obtained with air-entraining agents. These are then designated types IA, IIA, and llIA. Some standard blended portland cements that are available are called portland blast-furnace slag cement and portland pozzolan cement.

Typical composition values for the various compounds of the five cement types are shown in Table 3.1. These numbers are mean values, and there is a specified minimum and maximum for each compound.

Fineness

The term fineness refers to the average size of the cement particles. The fineness of the cement determines the rate of reaction because finer particles have more surface area and, hence, generate more reactivity when water is added. Type III high-early-strength cement has more fine particles than type I cement. Finer cement bleeds less than coarser cement. In addition, finer cement contributes to better workability and produces less autoclave expansion. But the finer cement is more

TABLE 3.1 Percentage Composition of Portland Cements

Type of cement		Component, %							General characteristics
		C_3S	C_2S	C_3A	C_4AF	CaSO4	CaO	MgO	
I	Normal	49	25	12	8	2.9	0.8	2.4	All-purpose cement
II	Modified	45	29	6	12	2.8	0.6	3.0	Comparative low heat liberation; used in large structures
III	High early strength	56	15	12	8	3.9	1.4	2.6	High strength in 3 days
IV	Low heat	30	46	5	13	2.9	0.3	2.7	Used in mass concrete dams
V	Sulfate resisting	43	36	4	12	2.7	0.4	1.6	Used in sewers and structures exposed to sulfates

expensive to produce, and if the particles are overly fine, they could lead to increased shrinkage, higher water demand, strong reaction with alkali-reactive aggregates, and poor stability.

The particle size usually varies from 1 to 200 ~m. Fineness of cement can be expressed using the Blame specific surface area. The cement is considered overly fine if the Blame specific surface area is greater than 2440 ft^2/lb (5000 cm^2/g). The specific surface areas, measured using the air permeability method, vary from 1220 to 1760 ft^2/lb (2500 to 3600 cm^2/g) and from 1760 to 2200 ft^2/lb (3600 to 4500 cm^2/g) for type I and III cements, respectively.

Cements with small particle sizes, known as microcements, are available for special purposes such as grouting. The fine particles facilitate the grouting of soils containing small pore sizes.

Testing Methods

Typically cement is tested at periodic intervals for chemical composition and certain physical properties to satisfy quality control requirements. Special tests may be needed for particular cases such as the determination of compatibility with certain admixtures. ASTM specifications exist for most of the tests. The following list contains commonly used tests and the corresponding ASTM specifications.

- *Chemical compositions:* Chemical analysis of portland cement (ASTM C14).
- *Fineness:* Sieve analysis (ASTM C 184), Wagner turbidimeter method (ASTM C115), Blame air-permeability method (ASTM C204).
- *Normal consistency* (ASTM C187): This test is used to determine the amount of water to be used in making samples to be tested for soundness, time of setting, and strength. The amount of water needed for normal consistency varies between 22 and 28% for portland cement.
- *Time of setting:* Time of setting includes both initial and final setting times. The time needed for the paste to start stiffening is called the initial setting time, whereas the final setting time represents the end of plasticity. The ASTM standards are C191 for Vicat apparatus and C266 for Gillmore needles. There is also a term called false set. This represents a stage in which the mix that is stiff can be remixed without adding water to restore plasticity. ASTM specifications require that initial setting time be at least 45 min by Vicat apparatus and 60 min by Gillmore needles. The corresponding final setting times are 8 and 10 h, respectively.
- *Soundness:* The test for soundness involves the measurement of the expansion of hardened cement paste. One of the popular tests is the autoclave test (ASTM C151).
- *Strength:* Strength is probably the most important property sought after. Both the magnitude of strength and the rate of strength development are important. The basic strength tests are compression (ASTM, C 109), tension (ASTM C190), and flexure (ASTM C348 or C349).
- *Heat of hydration (ASTM C186):* This property, which provides an indication of the amount of heat generated during hydration, is extremely important for most concrete construction such as dams, thick slabs, and pile caps.
- *Other tests:* Other tests that are used infrequently include shrinkage or expansion tests and tests for measuring specific gravity, alkali reactivity, sulfate resistance, air entrainment, bleeding, and efflorescence.

3.1.2 Aggregates

Aggregates are much less expensive than the cementing material and could constitute up to 90% of the volume of the concrete. They are typically considered as inert filler material, even though some aggregates do react minimally with the cement paste. Aggregates can be classified as coarse or fine based on their size; normal weight, lightweight, or heavyweight based on their bulk densities; and natural mineral or synthetic based on their type of production. Aggregate characteristics that affect the final product (namely, the concrete) include porosity, grading and size distribution, shape, surface texture, crushing strength, elastic modulus, moisture absorption, and type and amount of deleterious substances present. The primary concerns are the quality of the aggregate and its grading.

Coarse Aggregates
If the particle size is greater than 0.25 in (6 mm), the aggregates are classified as coarse aggregates. A list of the common coarse aggregate types follows.

- *Natural gravel:* About 50% of the coarse aggregates used in the United States consists of gravel. Natural cobbles and gravel are produced by weathering action. They are usually round in shape with a smooth surface. Hence these aggregates provide better workability. When they are made of siliceous rocks and uncontaminated with clay or silt, they make strong and durable aggregates. Some of the very high strength concretes with a compressive strength in the range of 20,000 psi (140 MPa) are made using gravel aggregates.
- *Natural crushed stone:* Crushed stone aggregate is produced by crushing the rocks and grading them. About two-thirds of the crushed aggregate in the United States is made of carbonate rocks (limestone, dolomite). The remainder is made of sandstone, granite, diorite, gabbro, and basalt. Carbonate rocks are softer than siliceous sedimentary rocks. The characteristics such as strength, porosity, and durability could vary considerably. Hence care should be taken to avoid the rocks that are not suitable for aggregates. Crushed stone aggregates are typically angular in shape and thus less workable than gravel under similar conditions.
- *Lightweight aggregates:* Aggregates with a bulk density of less than 70 lb/ft^3 (1120 kg/m^3) are normally considered lightweight aggregates. However, there is a whole spectrum of lightweight aggregates weighing from 5 to 55 lb/ft^3 (80 to 900 kg/m^3), as shown in Fig. 3.1.[1] Natural lightweight aggregates are made by processing naturally occurring lightweight rock formations such as pumice, scoria, and tuff. But most of the lightweight aggregates used for structural concrete are made by expanding or thermally treating a variety of materials such as clay, shale, slate, diatomite, perlite, or vermiculite. Industrial by-products such as blast-furnace slag and fly ash are also used to manufacture lightweight aggregates. The lightweight aggregates can be grouped into

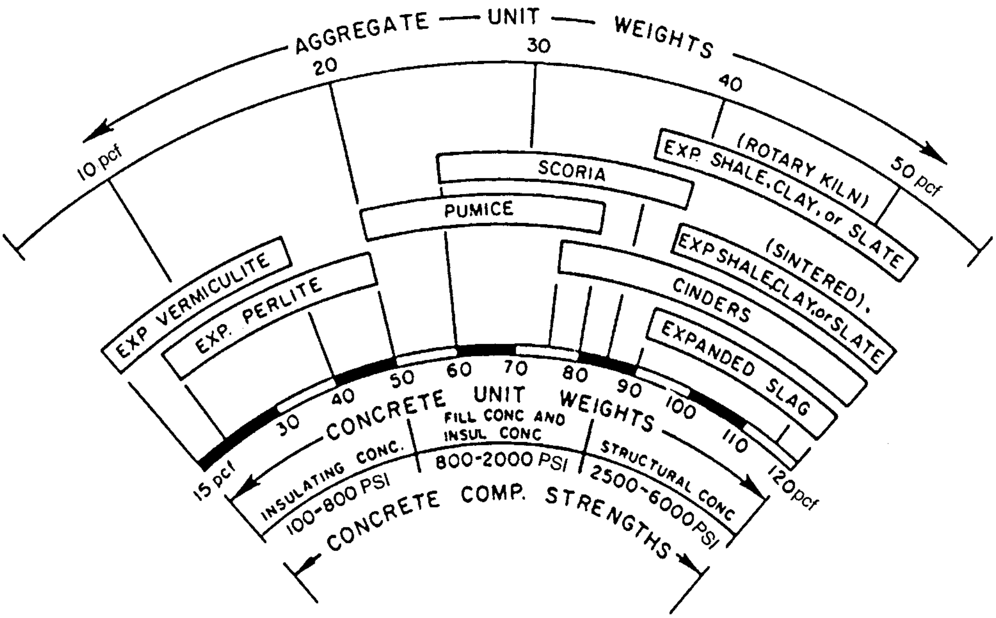


FIGURE 3.1 Lightweight aggregate spectrum. (*From Litvin and Fiorato.*[1])

three categories based on their end use: structural concrete, production of masonry units, and insulating concrete.

- *Heavyweight aggregates:* The bulk density of heavyweight aggregates ranges from 145 to 280 lb/ft^3 (2320 to 4480 kg/m^3). The primary use of these aggregates is for nuclear radiation shields. Natural rocks suitable for heavyweight aggregates may contain barium minerals, iron ores, or titanium ores. The aggregate types are witherite ($BaCO_3$), barite, ($BaSO_4$), magnetite (Fe_3O_4), hematite (Fe_2O_3), hydrous iron ores, ilinenite ($FeTiO_3$), ferrophosphorus (Fe_3P, Fe_2P, FeP), and steel aggregate (Fe). Ferrophosphorus aggregates when used with portland cement might generate flammable (and possibly) toxic gases and, hence, should be used with caution. Boron and hydrogen are very effective for neutron attenuation, and hence the aggregates containing these compounds are more useful for shielding. Heavyweight aggregates tend to produce more segregation. Sometimes preplaced-aggregate techniques are employed to make heavyweight concrete in order to avoid segregation.
- *Aggregates made from recycled concrete and other waste products:* Rubble from demolished buildings could be used as aggregates. The best source is the recycled concrete that contains less hydrolized cement paste and gypsum. Concrete made with these aggregates has about two-thirds of the compressive strength of concrete made with stone aggregates. One of the major projects built using recycled aggregate was a repavement project in Michigan. The work was carried out by the Michigan Department of Transportation and involved about 125,000 yd^3 (96,000 m^3) of concrete made using recycled concrete pavements.

Investigations have also been conducted for utilizing aggregates made with municipal wastes and incinerator ashes. The results are not very positive. The presence of glass particles tends to reduce workability and long-term durability. Metals such as aluminum react with alkaline materials in concrete and expand, causing deterioration. Organic wastes and paper interfere with the setting of concrete.

Fine Aggregates

Fine aggregates are usually made of natural or crushed sand. Their size ranges from 0.25 to 0.01 in (6 to 0.25 mm). The fine aggregates should be free of organic materials, clay, or other deleterious materials. Minimum particle sizes should be not less than 0.01 in (0.25 mm) because these fine particles tend to increase the water demand and reduce strength. For all-lightweight concrete, light-

TABLE 3.2 Grading Requirements for Coarse Aggregates

Size number	Nominal size (sieves with square openings)		Amounts finer than laboratory sieve (square openings), wt %				
			100 mm (4 in)	90 mm (3½ in)	75 mm (3 in)	63 mm (2½ in)	50 mm (2 in)
1	90–37.5 mm	(3½–1½ in)	100	90–100	—	25–60	—
2	63–37.5 mm	(2½–1½ in)	—	—	100	90–100	35–70
3	50–25.0mm	(2–1 in)	—	—	—	100	90–100
357	50–4.75 mm	(2 in–No. 4)	—	—	—	100	95–100
4	37.5–19.0mm	(1½–¾ in)	—	—	—	—	100
467	37.5–4.75 mm	(1½in–No. 4)	—	—	—	—	100
5	25.0–12.5 mm	(1–½ in)	—	—	—	—	—
56	25.0–9.5 mm	(l–⅜ in)	—	—	—	—	—
57	25.0–4.75 mm	(1 in–No. 4)	—	—	—	—	—
6	19.0–9.5 mm	(¾–⅜ in)	—	—	—	—	—
67	19.0–4.75 mm	(¾ in–No. 4)	—	—	—	—	—
7	12.5–4.75 mm	(½ in–No. 4)	—	—	—	—	—
8	9.5–2.36 mm	(⅜ in–No. 4)	—	—	—	—	—

Source: From *Annual Book of ASTM Standards*.[2]

weight sand made of expanded shale or clay materials is used. Similarly, for heavyweight concrete, regular sand is replaced with fine steel shot, iron ore, or other high-density material.

Production of Aggregates

The natural sand and gravel aggregates are the most economical aggregates to produce. They only need cleaning and grading. Cleaning can be accomplished by either screening or washing. Washing is more effective for removing clay and silt particles, but the process is more expensive.

When aggregates are produced by crushing rock, the type of crushing equipment depends on the type of rock to be crushed. The major sequential operations are crushing, cleaning, separation of various sizes, and blending of various sizes to obtain the required gradation.

In the case of synthetic lightweight aggregates, an extra step of expanding is needed. Generally, crushed or ground and pelletized raw material (clay, slate, or shale) is heated to about 2012°F (1100°C). At this temperature, the material expands because of the entrapped gases. The expanded material is processed to obtain the required grading. Lightweight aggregates are typically more expensive because of the extra effort and energy needed to produce them.

Size and Grading

The maximum size of the aggregates depends on the type of structure for which the concrete is to be used. For example, in dams and large foundations, maximum sizes greater than 2 in (50 mm) are very common, whereas for beams and columns containing extensive reinforcement, the maximum size is often restricted to 0.75 in (19 mm). The maximum size of the aggregate should be smaller than one-fifth of the narrowest dimension of the form and three-fourths of the maximum clear distance between reinforcements. For high-strength concrete the maximum size is generally limited to 0.75 in (19 mm).

As mentioned earlier, aggregates constitute a combination of different-size particles. The distribution of these particle sizes is called grading. The aim is to attain a grading (size distribution) that will produce a concrete with the required strength using minimum cement. At the same time, the mix should be workable in the plastic stage. Theoretically the best combination of particle sizes is the one that produces minimum voids and minimum total surface area. ASTM has established grading requirements for both coarse and fine aggregates so as to obtain workable concrete mixtures. Tables 3.2 and 3.3 present these requirements for normal-weight concrete. Similar requirements for

TABLE 3.2 (*Cont.*)

Amounts finer than laboratory sieve (square openings), wt %							
37.5 mm (1½ in)	25.0mm (1 in)	19.0mm (¾ in)	12.5 mm (½ in)	9.5 mm (⅜ in)	4.75 mm (No. 4)	2.36mm (No. 8)	1.18mm (No. 16)
0–15	—	0–5	—	—	—	—	—
0–15	—	0–5	—	—	—		
35–70	0–15	—	0–5	—	—	—	—
—	35–70	—	10–30	—	0–5	—	—
90–100	20–55	0–15	—	0–5	—	—	—
95–100	—	35–70	—	10–30	0–5	—	—
100	90–100	20–55	0–10	0–5	—	—	—
100	90–100	40–85	10–40	0–15	0–5	—	—
100	95–100	—	25–60	—	0–10	0–5	—
—	100	90–100	20–55	0–15	0–5	—	—
—	100	90–100	—	20–55	0–10	0–5	
—	—	100	90–100	40–70	0–15	0–5	—
—	—	—	100	85–100	10–30	0–10	0–5

TABLE 3.3 Grading Requirements for Fine Aggregates

Sieve (Specification E11)	Percent passing
9.5mm (⅜ in)	100
4.75 mm (No. 4)	95–100
2.36mm (No. 8)	80–100
1.18 mm (No.16)	50–85
600 µm (No. 30)	25–60
300 µm (No. 50)	10–30
150 µm (No. 100)	2–10

Source: From *Annual Book of ASTM Standards.*[2]

lightweight (C320) and heavyweight (C637) concrete can be found in the *Annual Book of ASTM Standards*, vol. 04.02.

The grading requirement for fine aggregate is straightforward (Table 3.3). The distribution of particles has to follow a certain pattern, with particles of different sizes limited to a certain range. For the coarse aggregate, first a maximum size should be chosen. The size distribution of the other particles depends on this maximum size. For example, if the maximum size is 1.0 in (25 mm), size numbers 5, 56, or 57 can be chosen (Table 3.2). If size 5 is chosen, 100% of the aggregates should pass through a 1.5-in (37.5-mm) sieve. Reading horizontally, 90–100, 20–55, 0–10, and 0–5% of material should pass through 1-, 0.75-, 0.5-, and 0.375-in (25-, 19-, 12.5-, and 9.5- mm) sieves, respectively. The sieve size represents the dimensions of the square openings. For example, a 1-in (25-mm) sieve has a mesh with a 1-in (25-mm) square opening. The reader should refer to ASTM C33 for other restrictions, such as fineness modulus.

Most of the time the aggregates have a uniform gradation. However, gap-graded aggregates are shown to provide better strength characteristics. In gap grading, the sizes of aggregates do not decrease uniformly. Certain size segments are left out in order to obtain a better packing and, hence, a more efficient utilization of cement.

Other Aggregate Characteristics

Grading is the most influential parameter that affects the behavior of concrete. Other factors influencing the quality of concrete are density and apparent specific gravity, absorption and surface moisture, crushing strength, elastic modulus, abrasion resistance, soundness, shape and surface texture, and the presence of deleterious substances. The significance of some of these parameters is listed in Table 3.4. The table also presents the ASTM test methods that can be used for the evaluation of these characteristics.

TABLE 3.4 Aggregate Characteristics and Their Significance

Characteristic	Effect on concrete	ASTM test method
Gradation	Economy, workability, long-term performance	Cl 17, C136
Bulk unit weight	Mix proportioning calculations	C29
Absorption and surface moisture	Quality control of concrete	C70, C 127, C 128, C566
Abrasion resistance	Wear resistance of floors and pavements	C131, C295, C535
Resistance to freezing and thawing	Surface scaling, durability	C295, C666, C682
Particle shape and surface texture	Workability of fresh concrete	C295, C398
Elastic modulus	Elastic modulus of concrete	C469

Selection of Aggregates

The ideal aggregate consists of particles that are strong, durable, clean, do not flake when wetted and dried, have somewhat rough surface texture, and contain no constituents that interfere with cement hydration. The grading of these particles should be done so that the concrete has good workability and the cement is utilized to its maximum efficiency. It is seldom possible to obtain ideal aggregates because of economical constraints.

In practical situations, aggregate selection should be made based on field conditions and end use. For example, a foundation built on sulfate-containing soil should not have aggregates vulnerable to sulfate attack. If the structure is going to be exposed to freezing and thawing cycles, durability of the aggregate plays a major role. The gradation requirements should be chosen not only for the maximum utilization of cement, but also to produce a workable concrete mixture. The availability of aggregates in close proximity plays an important economical role. If past records regarding the performance of the potential aggregate source based on existing structures are not available, suitable tests should be run to evaluate their properties. It should be noted that some of the disadvantages of the aggregate could be overcome by making minor modifications using admixtures. For example, if workability is the problem because of angular surfaces, water-reducing admixtures could be used to overcome this problem. The only constraint is the economy. The additional cost of the admixture should be justifiable.

3.1.3 Water and Water-Reducing Admixtures

Water is one of the primary ingredients of concrete. Water used during the mixing, called mixing water, performs the basic functions of hydration and lubrication. It also provides space for expanding hydration products. The lubrication action influences the workability of fresh concrete for placing, compaction, and finishing operations. The hydration, or the chemical reaction between water and cement, results in the hardening of concrete. Water is also needed for curing and sometimes for washing the aggregates. The amount of water needed for adequate workability is always greater than that needed for hydration. In addition, complete hydration of cement does not produce the highest strength. Therefore a number of admixtures were developed to improve the workability of concrete containing a limited amount of water. The most notable admixture is called high-range water-reducing admixture because of its very high efficiency. This section describes the requirements of water quality and the properties of water-reducing admixtures.

Water

Typically the water that is good for drinking is good for making concrete. Certain mineral waters that are potable may not be suitable for concrete. The water should be free of a particular taste, color, and odor, and should not foam or fizz when shaken. If in doubt, the water should be tested for suitability by evaluating the setting time of the cement paste, compressive strength, and durability. In most cases the setting time test alone may be sufficient.

The harmful contaminates not permitted in the water used for concrete are sugar, tannic acid, vegetable matter, oil, humic acid, alkali salts, free carbonic acid, sulfates, and water containing effluents from paint and fertilizer factories and sewage treatment plants. The following can be considered the general maximum limits for impurities:

Acidity	0.1*N* NaOH; 2 mL maximum to neutralize 200-mL sample
Alkalinity	0.1*N* HCl; 10 mL maximum to neutralize 200-mL sample; pH in the range of 6 to 9
Organic solids	$\ngtr$0.02%
Inorganic solids	$\ngtr$0.30%
Sulfuric anhydride	$\ngtr$0.04%
Sodium chloride	$\ngtr$0.10%
Turbidity	$\ngtr$2000 ppm

Water containing more impurities than the limits mentioned, such as seawater, has been used successfully. However, special tests should be conducted for the particular application prior to approval. In most cases a strength reduction of 15% can be expected if the mixing water contains salts. Decreased durability and corrosion of reinforcement can also be anticipated.

The requirements for curing water are less stringent because it comes in contact with the concrete on the surfaces only and that for too short a duration. Nevertheless, water with excessive impurities should not be used because it could cause surface discoloration. Under the worst conditions curing water could cause surface deterioration.

With regard to the water used for washing the aggregates and for concrete mixing and placing equipment the primary concern is the deposit of minerals on aggregate particles and equipment. The water should be clean enough not to leave any deposit. Washing the aggregate is not usually recommended because the disposal of contaminated water presents a problem. It should be noted that the disposal of contaminated water into natural streams and drains is not permitted in the United States.

Water-Reducing Admixtures

The water–cement ratio is the most influential parameter controlling the strength. Typically, a lower water content results in higher strength. However, a certain amount of water is needed to obtain workability so that the fresh concrete can be placed in position and compacted. Water-reducing admixtures improve the workability, and thus workable concrete can be obtained without increasing the water content. These admixtures can be used either to improve workability for the same water content, as the reduction of water without losing workability results in higher compressive strength, or to reduce the cement content, maintaining workability and strength.

For example, consider a reference concrete with 500 lb/yd^3 (300 kg/m^3) cement and a water–cement ratio of 0.62. This concrete had a slump of 2 in (50 mm) and a 28-day compressive strength of 5.3 ksi (37 MPa). Addition of the admixture resulted in an increase of slump (workability) to 4 in (100 mm). The compressive strength was 5.4 ksi (38 MPa).

When the admixture was used to reduce the water–cement ratio to 0.56, maintaining a slump of 2 in (50 mm), the compressive strength increased to 6.8 ksi (46 MPa). If the slump and strength levels were kept at the reference levels of 2 in (50 mm) and 5.3 ksi (37 MPa), the cement content could be reduced to 450 lb/yd^3 (270 kg/m^3). The 10% reduction in cement provides not only economy but also other benefits such as low heat of hydration and reduced shrinkage.

The principal active ingredients in water-reducing admixtures are salts, modifications and derivatives of lignosulfonic acid, hydroxylated carboxylic acids, and polyhydroxy compounds. Typically these admixtures provide better dispersion of cement particles and, hence, could provide an increase in the early-age strengths. Larger amounts of admixtures could retard the setting time by preventing the flocculation of hydrated particles. Some commercial formulations may contain accelerating admixtures to overcome this effect. Admixtures derived from lignin products also tend to entrain considerable air. This effect is nullified by adding air-detraining agents. The period of effectiveness varies with the formulations. However, in all cases the improved workability will be at least partially lost as the cement starts to hydrate.

A special type of water-reducing admixture called high-range water-reducing admixture or superplasticizer was developed in the 1970s and is commonly used now. As the name implies, this admixture provides substantial improvement in the workability. A water reduction of 20 to 25% is possible, as compared to S to 10% for normal water-reducing admixtures. Introduction of the superplasticizer is also responsible for the use of high-volume fly ash and silica fume in concrete because these mineral admixtures (at high volume fractions) have a high water demand and could not be used without the aid of a superplasticizer.

Superplasticizers consist of long-chain, high-molecular-weight anionic surfactants with a large number of polar groups in the hydrocarbon chain. These compounds are adsorbed on the cement particles during the mixing and impart a strong negative charge, helping to lower the surface tension of the surrounding water. This results in a uniform distribution of the cement particles and increased fluidity. Because of the better dispersion of cement particles, concrete made with superplasticizers tends to have higher 1-, 3-, and 7-day strengths as compared to reference concrete having the same water–cement ratio. The negative effect of better cement distribution is a rapid loss of workability because of accelerated setting. Hence set-retarding admixtures are typically added to control the setting time.

The four basic types of superplasticizers available in the market are as follows:

1. Sulfonated melamine formaldehyde condensates
2. Sulfonated naphthalene formaldehyde condensates
3. Modified lignosulfonates
4. Sulfonic acid esters or other carbohydrate esters

Typical dosage of superplasticizers is in the range of 1 to 2.S%, even though dosages as high as 4% have been used successfully. One of the major problems encountered in using superplasticizers is the loss of workability with time. Some of the original versions lost their effectiveness in less than 1 h. The currently available formulations are effective for longer durations. The individual commercial product should be checked for its effective duration. Multiple dosages or retempering with superplasticizer was also found to be effective. It was found that the mix can be retempered three times without affecting the mechanical properties adversely.

The water-reducing admixtures are covered in ASTM C494. High-range water-reducing admixlures are called type F, and the regular admixtures are called type A water-reducing admixtures.

3.1.4 Chemical Admixtures

As mentioned earlier, the primary ingredients of concrete are cement, aggregates, and water. Any other ingredient added to concrete can be classified as an admixture. The functions performed by admixtures include improved workability, acceleration or retardation of setting time, control of strength development, improved freeze-thaw durability, and enhanced resistance to water permeation, frost action, thermal cracking, aggressive chemicals, and alkali-aggregate expansion. The admixtures are also used to improve economy and save energy. In some countries up to 80% of all concrete produced contains some kind of admixtures.

The admixtures can be broadly classified as chemical and mineral admixtures. The mineral admixtures are discussed in Sec. 3.l.5. The most frequently used chemical admixtures are (1) accelerators or retarders, (2) water reducers, and (3) air-entraining admixtures. The water-reducing admixtures were discussed in Sec. 3.l.3. This section deals with the other admixtures.

Accelerating Admixtures

Accelerating admixtures, or accelerators, classified as type C admixtures in ASTM C494, are used in concrete to reduce the time of setting or to enhance early strength development, or both. It should be noted that increased early-strength development could lead to a reduction in strength at later ages. In any case, improvement in long-term strength should not be expected. The chemical components used in accelerating admixtures include soluble chlorides, carbonates, silicates, fluorosilicates, hydroxides, bromides, and organic compounds.

The higher early strength can be used to achieve the following benefits in construction:

- Early finishing of surfaces
- Early removal of forms
- Early opening of construction for service
- More efficient plugging of leaks against hydraulic pressure
- Partial or complete compensation for effects of low temperature
- Reduction of the time required for curing and protection against cold weather

The most common accelerator used for concrete is calcium chloride ($CaCl_2$). Calcium chloride can be safely used up to 2% by weight of cement. The influence of this chemical on setting time and strength development is presented in Fig. 3.2. From this figure it can be seen that (1) the initial and final setting times can be reduced by as much as 50 and 70%, respectively, (2) 1- and 3-day strengths can be increased significantly, and (3) as the curing temperature decreases, the effectiveness of the admixture increases.

The influence of calcium chloride on various properties of concrete is shown in Table 3.5. Almost all of the mechanical properties at early age are improved. The most detrimental effect is on the corrosion of metals. Corrosion becomes a major problem only in locations where there is a steep gradient in the chloride ion concentration. However, even a uniform concentration beyond 2% is viewed with suspicion. The addition of calcium chloride also increases creep and shrinkage and aggravates alkali-aggregate reaction.

A number of nonchloride accelerating admixtures have been developed during the late 1980s. Some of these admixtures are almost as effective as calcium chloride.

Air-Entraining Admixtures

Air-entraining admixtures are used to entrain small spherical air bubbles about 10 to 1000 μm in diameter. Entrained air significantly increases the frost resistance and durability under freezing and thawing conditions. Most specifications mandate air entrainment for exposed structures. About 9% by volume of mortar is recommended for proper freeze-thaw durability. The air voids should be distributed uniformly with low spacing factors (distance between bubbles). Smaller bubbles are more

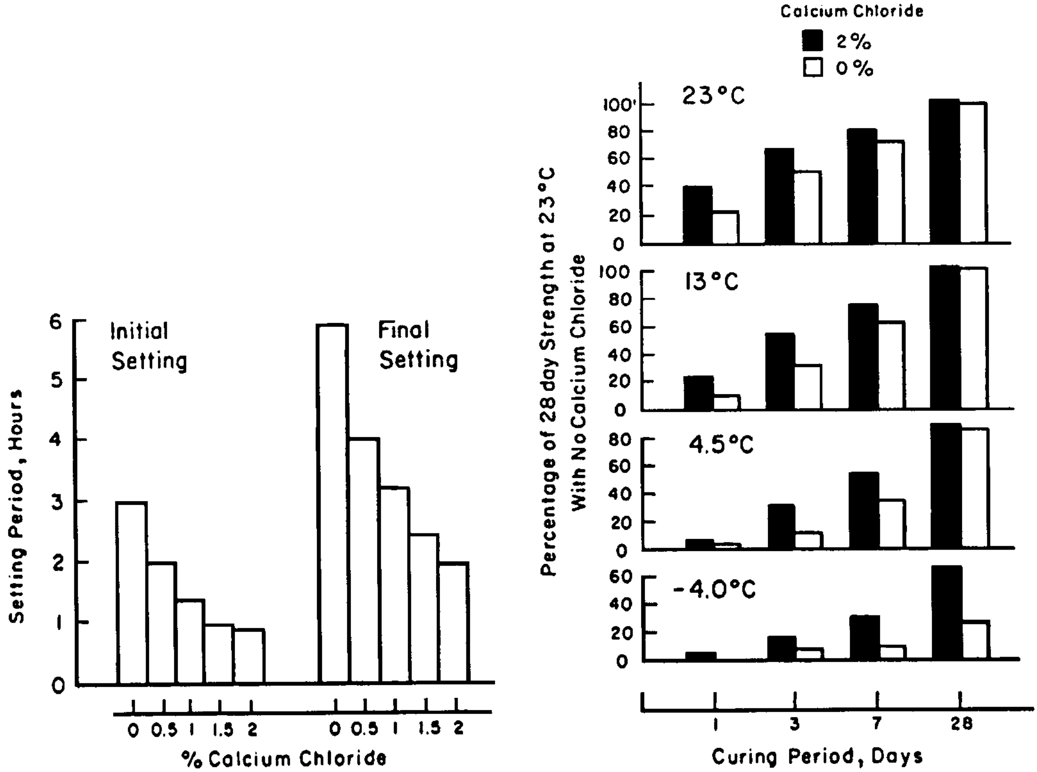


FIGURE 3.2 (a) Effect of calcium chloride addition on setting time of portland cement. (b) Effect of calcium chloride addition on strength at various curing temperatures. (*From Ramachandran.*[3])

TABLE 3.5 Some Properties Influenced by Calcium Chloride Admixture in Concrete

Property	Effect	Remarks
Setting	Reduces both initial and final setting times	ASTM standard requires that initial and final setting times occur at least 1 h earlier with respect to reference concrete.
Compressive strength	Significant increase at 3 days (gain may be about 30–100%)	ASTM requires an increase of at least 125% over control concrete at 3 days.
Tensile strength	Slight decrease at 28 days	
Flexural strength	Decrease of about 10% at 7 days	This figure may vary depending on starting materials and method of curing. The decrease may be more at 28 days.
Heat of hydration	Increase of about 30% in 24 h	Total amount of heat at longer times is almost the same as that evolved by reference concrete.
Resistance to sulfate attack	Reduced	Can be overcome by using type V cement with adequate air entrainment.
Alkali-aggregate reaction	Aggravated	Can be controlled by using low-alkali cement or pozzolana.
Corrosion	Causes no problems in normal reinforced concrete if adequate precautions taken (dosage should not exceed 1.5% $CaCl_2$ and adequate cover given)	Calcium chloride admixture should not be used in prestressed concrete or in a concrete containing a combination of dissimilar metals. Some specifications do not allow use of $CaCl_2$ in reinforced concrete.
Shrinkage and creep	Increased	
Volume change	Increase of 0-15% reported	
Resistance to damage by freezing and thawing	Early resistance improved	At later ages may be less resistant to frost attack.
Watertightness	Improved at early ages	
Modulus of elasticity	Increased at early ages	At longer periods almost same with respect to reference concrete.
Bleeding	Reduced	

Source: From Ramachandran.[3]

effective than larger ones. Entrapped air, which is the result of incomplete compaction, is not effective in improving durability.

The usual dosage of air-entraining mixtures is in the range of 0.02 to 0.06% by weight of cement. Higher dosages may be required for mixes containing type III cement, pozzolan cements, fly ash, or other finely divided powders. The volumetric air content in typical air-entrained concrete varies from 4 to 10%. The presence of air improves the workability of fresh concrete because the bubbles increase the spacing of the solids, resulting in decreased dilatancy. The air also reduces segregation and bleeding. Entrained air is particularly helpful in lightweight concrete because of the unfavorable shape and surface texture of the fine fraction of most light-weight aggregates. The air reduces the harshness of the mix and the bleeding rate in addition to providing improved workability. Entrained air typically reduces the compressive strength. Concrete containing 8 vol % air can be expected to register about 15% reduction in compressive strength as compared to control concrete with no entrained air and comparable workability. Note that to obtain comparable workability, the water content of the control concrete has to be increased.

Most of the commercially available air-entraining admixtures are in liquid form; a few are available in powder, flake, or semisolid form. The ingredients used for manufacturing admixtures include salts of wood resins, synthetic detergents, salts of sulfonated lignin, salts of petroleum acids, salts of proteinaceous materials, fatty and resinous acids and their salts, and organic salts of sulfonated hydrocarbons. The air-entraining admixture is typically added to the mixing water. Air-entrained concrete can also be made using air-entraining portland cement. The former method is preferable because the amount of air can be controlled easily.

The amount of air, or air content, is the primary factor that influences freeze-thaw durability and frost resistance. Other factors that are important include size and distribution of air voids, specific surface, and spacing of bubbles. These factors are influenced not only by the amount of air-entraining agent but also by the nature and proportions of the other ingredients of the concrete, including other admixtures, water–cement ratio and consistency of the mix, type and duration of mixing, temperature, and the type and degree of compaction employed. Based on the constituent materials and the type of mixing, placing, and compaction, the amount of air-entraining agent should be adjusted to obtain the desired air content in the final product.

Set-Controlling Admixtures

Set-controlling admixtures are typically used in conjunction with water-reducing admixtures. As mentioned in Sec. 3. 1.3, retarders have to be used in conjunction with some water-reducing admixtures in order to extend the life, or effectiveness, of the water-reducing admixtures. Materials, such as lignosulfonic acids and their salts and hydroxylated carboxylic acids and their salts, can act as water-reducing, set-retarding admixtures.

Set-retarding admixtures are used by themselves to offset the accelerating effects of high temperatures and to extend the workable period for proper placing. Set retarders are also used to keep the concrete workable for a longer duration. The longer duration may be needed to avoid cold joints in large constructions or to prevent cracking in beams, bridge decks, and composite construction because of the deflections caused when the adjacent spans are loaded.

The amount of retardation obtained depends on the chemical composition of the admixture, its dosage, type and amount of cement, temperature, mixing sequence, and other job conditions. The quantity of admixture required should be determined carefully. Excessive retardation can damage the setting and hardening process, resulting in long-term detrimental effects.

Other Chemical Admixtures

Chemical admixtures other than the ones discussed so far include admixtures used for (1) air detraining, (2) gas forming, (3) producing expansion, (4) damp proofing, (5) bonding, (6) reducing alkali-aggregate expansion, (7) inhibiting corrosion, (8) flocculating, (9) coloring, and (10) fungicidal, germicidal, and insecticidal purposes. In addition, admixtures are available for water thickening and for reducing friction in pumping concrete.

Polymer and latex-modified concretes are also being used, especially for repair and restoration. Polymers are used as bonding agents and surface coatings to reduce permeability of the surface layer. Impervious surface layers typically improve the long-term durability.

3.1.5 Mineral Admixtures

Mineral admixtures in concrete provide improved resistance to thermal cracking because of reduced heat of hydration, reduce the permeability by reducing pore sizes, increase strength, and improve resistance against chemicals such as sulfate water and alkali-aggregate expansion. Most of the mineral admixtures have some pozzolanic property. A pozzolan is defined as a siliceous or siliceous and aluminous material that will chemically react with calcium hydroxide at normal temperatures to form compounds possessing cementitious properties. The material has to be in a finely divided form, and the reaction will take place only in the presence of moisture. Pozzolans possess little or no cementitious value by themselves. Typically, mineral admixtures are used in large volume fractions, generally in the range of 20 to 100% by weight of cement.

Even though a number of naturally occurring pozzolans exist and can be used as admixtures, most of the admixtures used in concrete, especially by industrialized nations, are industrial by-products. The most commonly used industrial by-product is coal fly ash produced by power plants. Blast-furnace slag and silica fume are the other major industrial by-products used in concrete. Silica fume, which contains much finer particles, is typically used for high-strength and impermeable concrete. These three admixtures are discussed next, followed by other mineral admixtures.

Fly Ash

In modern power plants, coal is fed into furnaces in powder form to improve thermal efficiency. As the coal powder passes through the high-temperature zone in the furnace, the volatile matter and carbon burn off, providing heat generation. The impurities, such as clay, feldspar, and quartz, melt and fuse. When the fused matter moves through zones of lower temperature, it solidifies as glassy spheres. The glass contents in these spheres range from 60 to 85%. Some of these spheres are hollow and very light. The spheres get blown out with the flue gas stream. These particles, collected with special equipment such as electrostatic precipitators, are called fly ash. Special equipment is needed for collecting most of the tiny particles so that the amount of ash discharged into the atmosphere is at an absolute minimum.

Based on the amount of calcium content, fly ash is classified as low-calcium or high-calcium. ASTM classifies high-calcium fly ash (CaO content 15 to 35%) as type C and low-calcium fly ash (CaO less than 10%) as type F fly ash. High-calcium fly ash is more reactive because the calcium occurs in the form of crystalline compounds. The chemical that is considered to be harmful to concrete is carbon. In most commercial fly ashes the carbon content is limited to 2%, and it rarely exceeds 5%. If the carbon content is high, the fly ash should not be used in concrete. A higher carbon content typically increases water demand and interferes with air-entraining admixtures.

The sizes of fly ash particles vary from <1 to 100 μm. The particle size distribution is shown in Fig. 3.3. This figure shows the particle distributions of type C and F fly ash, cement, and silica

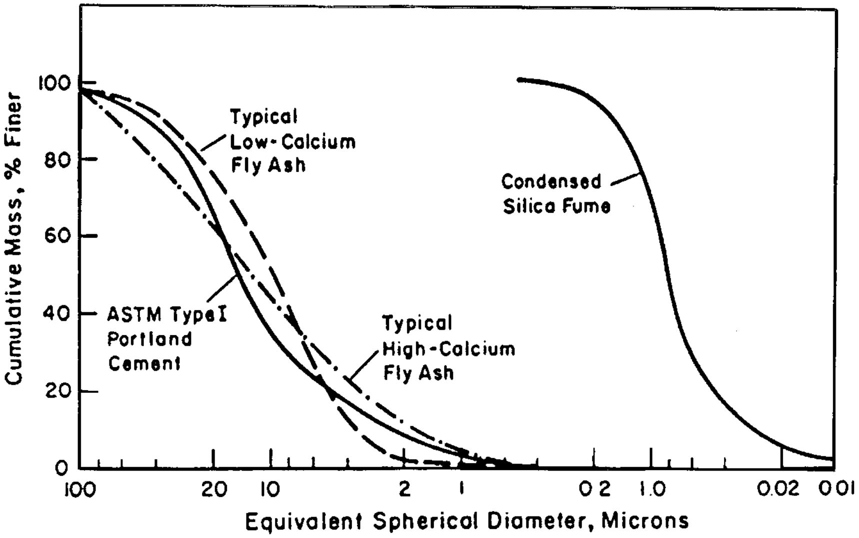


FIGURE 3.3 Particle-size distributions—comparison of cement, fly ash, and silica fume.

fume. It can be seen that the fly ash particle sizes are about the same as the cement particle sizes. About 50% of the particles are smaller than 20 μm. The particle size distribution influences the workability of fresh concrete and the rate of strength development.

It is well established that the addition of fly ash in the range of 10 to 20% by weight of cement improves the workability. Fly ash containing finer particles is more effective in improving workability. As much as 7% water reduction was reported with an addition of 30% fly ash.

One of the primary reasons for using fly ash in concrete is to reduce the heat of hydration during placement and the early curing period. Fly ash could reduce the rise in temperature almost in direct proportion to the amount of cement it replaces. Fly ash has been used in concrete as early as the 1930s. In most of the earlier applications the amount of cement replaced was limited to about 30%. Recently equal amounts of cement and fly ash have been used successfully for mass concrete construction. As mentioned earlier, the advent of superplasticizers was responsible for the use of high volume fractions of fly ash. At lower dosages, fly ash improves the workability. When large volume fractions are used, more water is needed to wet the fly ash particles. The use of superplasticizers allows for the least increase in water content, and hence high strengths can be achieved even with large volume fractions of fly ash.

The pozzolanic activity of fly ash refines the pores, and thus concrete containing fly ash is less permeable. The 90-day permeability of the cement can be reduced by almost an order of magnitude by replacing 1010 30% of cement with fly ash. This reduced permeability enhances the durability against chemical attacks significantly. Alkali-aggregate expansion can also be reduced by adding fly ash.

Replacement of cement with fly ash normally results in a reduction of the 28-day strength. But the long-term (56- or 90-day) strengths are always higher for fly ash concrete. High-strength concretes with compressive strengths greater than 8000 psi (55 MPa) always contain some form of mineral admixture.

Blast-Furnace Slag

Blast-furnace slag is a by-product of cast iron production. The chemical components present in the slag do not react with water at normal temperatures. The slag is ground to fine particles and used as mineral admixtures. In some cases the liquid slag is cooled rapidly to produce sandlike and pelletlike particles, called granulated and pelletized slag, respectively.

The properties of concrete containing slag are almost the same as the properties of concrete containing fly ash, in both the plastic and the hardened forms. The major difference is that slag particles smaller than 10 μm were found to contribute to an increase in the 28-day strength.

Silica Fume

Silica fume, also called condensed silica fume, volatilized silica, or microsilica, is a by-product of silicon metal and ferrosilicon alloy industries. The reduction of quartz to silicon at temperatures of 3632°F (2000°C) produces SiO vapors. These vapors oxidize and condense to form small spheres with diameters in the range of 0.01 to 0.2 μm (Fig. 3.3). Silica fume particles are about two orders of magnitude smaller than cement particles. Because of its extremely small particle size, silica fume is highly pozzolanic. But its higher surface area increases the water demand. Hence the use of silica fume was almost impossible until the advent of superplasticizers. The use of silica fume for high-strength concrete became a common occurrence after the super-plasticizers were introduced.

The silica fume content in concrete ranges from 5 to 30% by weight of the cement. The addition of silica fume typically results in denser concrete with low permeability. Silica fume is more effective in reducing permeability than fly ash. The strength increase provided by silica fume is also more substantial than that obtained by the addition of fly ash. Because of the collecting and processing expense, silica fume is much more expensive than fly ash. It is available in powder and slurry form. Silica fume concrete typically has a higher Young's modulus and is more brittle than concrete of comparable strength that contains no silica fume.

Rice-Husk Ash

Rice-husk ash can be considered a natural product, even though it is produced by controlled combustion of rice husks. Rice husks are the product of a dehusking operation in which outer shells are removed from rice. Husk constitutes about 20% of paddy by weight. The combustion process reduces the weight about fivefold. Ash produced by controlled combustion was found to produce non-reactive silica minerals. These ashes have to be finely ground before using in concrete.

Naturally Occurring Mineral Admixtures

Naturally occurring pozzolans are typically mined and processed. Processing normally involves the steps of crushing, grinding, and size separation. In some cases thermal activation may be needed.

Natural pozzolans are derived from volcanic rocks and minerals and from diatomaceous earth. Diatomaceous earth consists of hydrated silica derived from skeletons of diatoms. Diatoms are tiny water plants whose walls are composed of silica shells. Pozzolans were formed during volcanic eruption because of the quick cooling of magma-containing aluminosilicates.

Based on their major chemical components, natural pozzolans can be classified as (1) volcanic glasses, (2) volcanic tuffs, (3) calcined clays or shales, and (4) diatomaceous earth.

3.2 COMPUTATION OF REQUIRED AVERAGE COMPRESSIVE STRENGTH f'_{cr} AND MIX PROPORTIONING

The structural engineer who designs the components specifies the minimum compressive strength required for the concrete to be used. This strength is called specified compressive strength f'_c In most cases the specified strength is measured at the age of 28 days. In some cases 56- or 90-day strengths are also specified. Compressive strength tests are normally conducted using cylinders, cubes, or prisms. It is the job of the construction professionals to ensure that the concrete placed in position satisfies the specified strength requirement. Since concrete is a composite, cast in the field using a number of constituent materials and various casting and curing procedures, there is always a variation in strength. The major parameters that influence the strength include the following:

1. Amount of water used in the mix, or water–cement ratio
2. Aggregate-cement ratio
3. Quality of cement
4. Strength, shape, texture, cleanliness, and moisture content of aggregates
5. Type and amount of mineral and chemical admixtures
6. Mixing procedure and adequacy of mixing
7. Placing, compacting, and finishing techniques used during construction
8. Curing conditions and type of curing method
9. Test procedures

Because of variations in any of these parameters, or other factors such as temperature and humidity in the field during the construction, there is always a variation in compressive strength. Typically, quality control tests are run in both the field and the laboratory to monitor the variations and take corrective measures if needed. American Concrete Institute (ACI) code 318-92 specifies the following acceptance criteria for concrete[4]:

1. The average of all sets of three consecutive strength tests must equal or exceed f'_c, and
2. No individual strength test (average of two cylinders) must fall below f'_c by more than 500 psi (3.4 MPa)

The code also provides guidelines for achieving the acceptable concrete.

Stated in simple terms, the code requires that the concrete be proportioned to obtain an average compressive strength f'_{cr}, that is higher than the specified strength f'_c. The magnitude of overdesign, that is, the difference between f'_{cr} and f'_c, depends on the rigorousness and success of the quality control measures used on the job site. The concrete should be proportioned to obtain the average compressive strength f'_{cr}, and not the specified strength f'_c.

This section deals with the computation of f'_{cr}, which is also known as required average strength, and the mix proportioning procedures. The computation of f'_{cr} is based on ACI code guidelines. It should be noted that other codes may require different procedures for the computation of the required average strength.

3.2.1 Computation of Required Average Strength, f'_{cr}

If the concrete production facility has test records from previous projects, these records can be used to establish the variability and the mix proportions. In this case the computation of f'_{cr} is based on previous records. If the records are not available, a certain variability is assumed. As the project progresses, the data from the project can be used to establish the variability.

If field data are available, the computation of f'_{cr} requires determination of the standard deviation and the number of acceptable low tests. It is assumed that the strength variation of concrete (proportioned to obtain the same compressive strength) follows a normal distribution. If a large number of samples is collected, they were found to follow the normal distribution curve. The ACI code specifies a minimum of 30 samples. If 30 or more samples are available, their average and their standard deviation are assumed to be the same as the average and the standard deviation of a large population.

Once the strength variation is assumed to follow a normal distribution, the properties of the normal distribution curve can be used to predict the probability that a given sample will have a strength less than f'_{cr}. For example, if the average is x, and the standard deviation is s, it can be said that:

- Half the samples will have strengths less than x, and hence the probability that a certain sample will have a strength less than x is 50%.
- About 68.27% of all samples will have strengths greater than $(x - s)$ and less than $(x + s)$, or 15.87% of samples will have strengths less than $(x - s)$, or the probability that a given sample will have strength less than $(x - s)$ is 15.87%.
- About 95 .45% of all samples will have strengths between $(x - 2s)$ and $(x + 2s)$, or the probability that an individual sample will have strength less than $(x - 2s)$ is 2.25%.
- About 0.13% of all samples, or 1 in 741 samples, will have a strength less than $(x - 3s)$.

The aforementioned postulations were derived using the property of the normal or bell curve. It can be seen that the two factors that control the prediction are the sample average x and the standard deviation s.

If the average of a given set of strength results x, is assumed to be the required average strength f'_{cr}, then the mix proportions used to obtain the strengths are good enough for a specified strength f'_c subjected to the condition

$$f'_{cr} = f'_c + tS \tag{3.1}$$

where t is a statistic which depends on the number of tests permitted to fall below f'_c. If 50% of the tests are permitted to fall below, then $t = 0$, or f'_{cr} and f'_c are the same. Naturally, in the actual construction such a large number of low tests cannot be permitted. For structural concrete, the ACI code specifies the following two equations:

$$f'_{cr} \geq f'_c + 1.34S \tag{3.2}$$

$$f'_{cr} \geq f'_c + 2.33S - 500 \text{ psi} \qquad (1000 \text{ psi} = 6.89 \text{ MPa}) \tag{3.3}$$

Equation (3.2) results in a probability that not more than 1 in 100 averages of three consecutive strength tests (each being the average of two cylinders) will be less than f'_c. The t value for a probability of 1 in 100 is 2.33, and the number 1.34 is obtained by dividing 2.33 by 3.The division by 3 for three consecutive tests is based on theorems in statistics. Equation (3.3) results in the probability that not more than 1 individual strength in 100 (average of two cylinders) falls below f'_c – 500 psi (f'_c – 3.4 MPa). Note that these two equations are consistent with both acceptance criteria presented previously. Overall ACI code recommendations for the computation of f'_{cr} are based on the acceptance of 1 low test in 100. It should be noted that this probability is used only for the computation of f'_{cr}, and if the low strengths do occur, correction measures should be taken to increase the average strengths.

For the construction of facilities such as footpaths, a higher number of low tests could be acceptable. In this case the t value is reduced. For example, if 1 low test in 10 is acceptable, then the t value is 1.28. More information on the t values for various probabilities and the recommended low tests for various facilities can be found in the report of ACI Committee 214.[5]

Once Eqs. (3.2) and (3.3) are accepted as the basis for the computation of f'_{cr}, the process becomes a set of mathematical steps. The ACI code also establishes procedures for accepting data sets with less than 30 tests. In addition there are certain restrictions to be satisfied for using the data from previous projects. The following is the gist of the various provisions of the ACI code. The restrictions and their interdependence are presented as a flowchart in Fig. 3.4.

If a concrete production facility has test records, establish the standard deviation using its records, provided that the material quality control procedures and conditions are similar to the proposed project and the f'_c for the proposed work is within 1000 psi (6.89 MPa) of the f'_c for which the records exist.

If the records contain 30 or more consecutive tests, compute the standard deviations using the equation

$$S = \left[\frac{(x_i - \bar{x})^2}{n - 1} \right]^{1/2} \tag{3.4}$$

where n = number of consecutive strength tests
x_i = individual strength tests (average of two cylinders at 28 days or at designated test age for the determination of f'_c).

$$\bar{x} = \frac{x_i}{n} \tag{3.5}$$

If the records contain two groups of consecutive tests totaling at least 30 tests, compute the standard deviations S_1 and S_2 for the two sets using Eqs. (3.4) and (3.5). Compute the statistical average of S_1 and S_2 using

$$S = \left[\frac{(n_1 - 1)S_1^2 + (n_2 - 1)S_2^2}{n_1 + n_2 - 2} \right]^{1/2} \tag{3.6}$$

where S = statistical average standard deviation where two test records are used to estimate standard deviation
S_1, S_2 = standard deviations of sets 1 and 2
n_1, n_2 = number of tests in respective test record

If the available number of tests is less than 30 but greater than 25, multiply the estimated standard deviation by a factor of 1.03 and use this value in Eqs. (3.2) and (3.3). The corresponding factors for data sets with a minimum of 20 and 15 tests are 1.08 and 1.16, respectively.

Using the estimated value of X, compute the required average strength f'_{cr} using Eqs. (3.2) and (3.3).

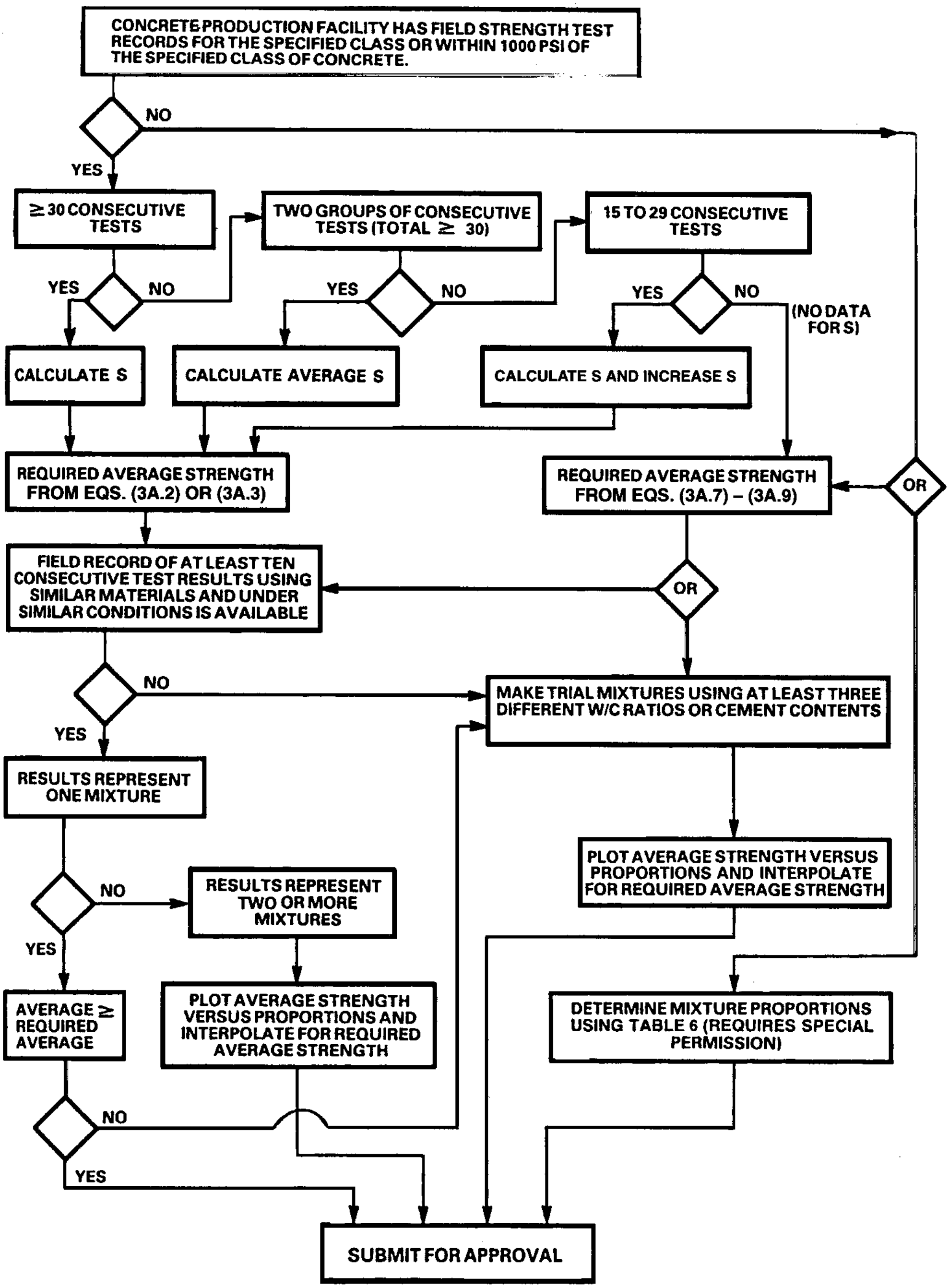


FIGURE 3.4 Flowchart for selection and documentation of concrete proportions. (*From ACI 318-92.*[4])

If data are not available for estimating the standard deviation,

$$f'_{cr} = f'_c + 1000 \text{ psi (6.89 MPa)} \qquad f'_c < 3000 \text{ psi (20.7 MPa)} \tag{3.7}$$

$$= f'_c + 1200 \text{ psi (8.27 MPa)} \qquad 3000 \le f'_c \le 5000 \text{ psi } (20.7 \le f'_c \le 34.5 \text{ MPa}) \tag{3.8}$$

$$= f'_c + 1400 \text{ psi (9.64 MPa)} \qquad f'_c > 5000 \text{ psi (34.5 MPa)} \tag{3.9}$$

Equations (3.7) to (3.9) are based on conservative (or overestimated) estimates of S values.

The following numerical examples further illustrate the procedure used for the computation of f'_{cr} under various conditions.

Example 3.l Compute the required average compressive strength f'_{cr} for the following cases if the specified compressive strength is 4000 psi (27.6 MPa). Assume similar materials and conditions for all cases. The available test results (average of two cylinders) are as follows:

(*a*)	(*b*)	(*c*)	(*d*)	(*e*)
4260	5200	*First set:*	4700	None
4760	5220			
4120	4750			
3650	4440	3100	4050	
4310	4750	4450	3500	
4960	4720	3900	3250	
4350	4650	4900	4100	
3980	5020	3750	4350	
4450	4920	4100	3750	
4430	5780	4400	3600	
4240	5350	4500	3550	
4400	5420	5100	3900	
3420	5070	3500	3850	
4760	5220	3700	4150	
4620	4200	3250	4200	
4260	5070	3750	4600	
3860	5600	4250	4300	
4290	5780	*Second set:*		
5120	4900	4700		
4870	5980	4750		
4150	5200	4850		
4170	4320	3750		
3740	5170	3050		
4180	5030	3000		
4860	5700	3900		
3890	5050	3850		
4720	5200	4100		
4880	4130	4150		
4980	4450	4350		
4240	5300	4400		
	4600	4950		
	4850	4050		
	6140	3450		
	5100	3200		
	5070	3250		
	4850			

1000 psi = 6.g95 MPa.

Solution In case (*a*) the number of tests ≧ 30. Hence the single set can be used:

$$\bar{x} = \frac{x_i}{n}$$

$$= \frac{4260 + 4760 + \cdots + 4980}{30}$$

$$= 4364 \text{ psi } (30 \text{ MPa})$$

The average is within 1000 psi of f'_c. Thus

$$S = \left[\frac{(x_i - x)^2}{n-1}\right]^{1/2}$$

$$= \left[\frac{(4364 - 4260)^2 + \cdots + (4364 - 4240)^2}{29}\right]^{1/2}$$

$$= 426 \text{ psi } (2.94 \text{ MPa})$$

$$f'_{cr} > 4000 + 1.34 \times 426 = 4571 \text{ psi } (31.5 \text{ MPa})$$

$$> 4000 + 2.33 \times 426 - 500 = 4493 \text{ psi } (31.0 \text{ MPa})$$

Hence f'_{cr} = 4571 psi, or 4600 psi (32 MPa).

In case (*b*) the number of tests is greater than 30. Hence the single set can be used:

$$\bar{x} = \frac{5200 + \cdots + 4850}{36}$$

$$= 5061 \text{ psi } (34.9 \text{ MPa})$$

The difference between the average strength and the specified strength of 4000 psi is greater than 1000 psi. Hence the data set cannot be used. The problem has to be treated as though data were not available. Equation (3.8) controls,

$$f'_{cr} = f'_c + 1200 \text{ psi}$$

$$= 4000 + 1200 \text{ psi} = 5200 \text{ psi } (36 \text{ MPa})$$

In case *c*) the average for the first data set $\bar{x}_1$ is 4046 psi, n_1 = 14. The average for the second data set $\bar{x}_2$ = 3985 psi, n_2 = 17. Both averages are within 1000 psi of f'_c and can be used for the computation of f'_c:

$$S_1 = 590 \text{ psi } (4.07 \text{ MPa})$$

$$S_2 = 637 \text{ psi } (4.39 \text{ MPa})$$

$$S = \left[\frac{13 \times 590^2 + 16 \times 637^2}{14 + 17 - 2}\right]^{1/2}$$

$$= 616 \text{ psi } (4.25 \text{ MPa})$$

$$f'_{cr} > 4000 + 1.34 \times 616 = 4825 \text{ psi (33.3 MPa)}$$

$$f'_{cr} > 4000 + 2.33 \times 616 - 500 = 4935 \text{ psi (34.0 MPa)}$$

Therefore,

$$f'_{cr} = 4935 \text{ psi,} \qquad \text{or 4950 psi (34 MPa)}$$

In case (*d*) 15 tests are available:

$$\overline{X} = 3990 \text{ psi}$$

X is within 1000 psi of f'_c,

$$S = 415 \text{ psi (2.86 MPa)}$$

The multiplying factor is 1.16. Therefore S is used for the computation:

$$f'_{cr} = 1.16 \times 415 = 481 \text{ psi (3.32 MPa)}$$

$$f'_{cr} > 4000 + 1.34 \times 481\ 4645 \text{ psi (32.0 MPa)}$$

$$f'_{cr} > 4000 + 2.33 \times 481 - 500 = 4621 \text{ psi (31.9 MPa)}$$

Therefore,

$$f'_{cr} = 4645 \text{ psi,} \qquad \text{or 4650 psi (32 MPa)}$$

For case (*e*) data are not available. Therefore,

$$f'_{cr} = 4000 + 1200 \text{ psi} = 5200 \text{ psi (36 MPa)}$$

which is the same as case (*b*).

3.2.2 Selection of Concrete Proportions

Once the required average compressive strength f'_{cr} is established, the mix proportions that can produce average strengths equal to or greater than f'_{cr} can be chosen based on (1) field records or (2) trial mixes. It should be noted that the required average strength f'_{cr} is only one of the parameters to be considered in mix proportioning. Other primary requirements are workability, consistency, and resistance to special exposure conditions. The final mix proportion should also be the most economical solution for the given set of conditions.

Workability of fresh concrete determines the ease with which the concrete can be transported, placed in position, consolidated, and finished. A mix that is too difficult to handle or consolidate may result in a final product that has honeycombs. Poorly consolidated concrete will not only have poor strength but will deteriorate quickly. Similarly, a mix with excess water may lead to segregation and bleeding, again resulting in a poor final product. The consistency of the mix should be just sufficient for proper placing, consolidation, and finishing operations. Slump is the most widely used indicator of workability. Section 3.3. 1 deals with the slump and its measurement. For general applications, concrete with sufficient strength is automatically assumed to be durable. However, special precautions should be taken when proportioning concrete that is to be exposed to severe environments. The following are the requirements for certain exposure conditions. The list is not comprehensive. Each structure should be treated carefully based on the exposure conditions.

- Normal-weight and lightweight concrete exposed to freezing and thawing or deicer chemicals should be air-entrained. The ACI code specifies 3.5 to 6% and 4.5 to 7.5% air content for moderate and severe exposure conditions, respectively. Higher air content is needed for mixes with smaller-size aggregates. For concretes with strengths higher than 5000 psi (35 MPa) the air-content requirement is slightly relaxed because these concretes made with lower water–cement ratios are generally less permeable. In addition to air entrainment, the water–cement ratio should be limited to 0.45 for concrete subjected to freezing and thawing.
- If the concrete is to be used for reinforced structural components exposed to salts, seawater, or brackish water, the water–cement ratio should be limited to 0.40.
- Water-cement ratio restrictions (preferably not more than 0.45) should also be used for concrete exposed to sulfates.
- For concrete exposed to freezing and thawing in the presence of deicing salts, the ACI code also stipulates a minimum cement content of 520 lb/yd^3 (310 kg/m^3).

The importance of the economics, or the cost-optimum solution, cannot be overemphasized. Typically, cement is the most expensive ingredient in concrete. However, because of the large quantities of materials involved, even a few cents per ton of aggregate may translate into millions of dollars. When selecting ingredients, availability and transportation costs should be considered carefully. In certain instances, locally available materials could be used in conjunction with admixtures rather than transporting materials that do not need admixtures for long distances. Generally the least amount of cement that is needed for obtaining the required strength and durability provides the best economical solution. The least amount of cement also provides some technical advantages, such as lower heat of hydration and less shrinkage. The use of mineral admixtures such as fly ash also provides cost savings. The use of industrial by-products produces savings in disposal costs and better utilization of resources. These aspects are important for both industrialized countries that have limited disposal space and developing countries that have limited resources.

3.2.3 Proportioning on the Basis of Field Strength Records

If field strength records are available for concrete with an average compressive strength in the range of f'_{cr}, the mix proportions used for this concrete can be used for the new project provided they satisfy the workability and durability requirements outlined in the previous section. The field test records should satisfy the following conditions:

- The available test records shall represent materials and conditions similar to those expected. Quality control, in terms of uniformity of materials, conditions, and proportions, shall be the same or better for the proposed work as compared to the project from which the records are taken. If field records were used for the computation of f'_{cr}, the same records can be used for the selection of mix proportions.
- It is preferable to have 30 or more consecutive test records. However, 10 consecutive test results may be used if the records encompass a period of time not less than 45 days.
- Mix proportions can also be chosen by interpolation using records that resulted in compressive strengths higher and lower than f'_{cr}.

3.2.4 Proportioning on the Basis of Trial Mixtures

When acceptable field records are not available, mix proportions can be established using trial mixtures. Trial mixes should be made using at least three different water–cement ratios or cement contents to establish a relation between compressive strength and water–cement ratio or between strength and cement content. The relationship of strength versus water–cement ratio can be used to establish the maximum water–cement ratio that can produce the required f'_{cr}. On the other hand, if the strength versus cement content relationship is obtained, it can be used to establish the minimum cement content. In either case the chosen mix proportion should satisfy the requirements for workability and durability. The trial mixes should also meet the following restrictions:

- The combination of the materials used should be the same as that of the materials to be used for the proposed work.
- Extrapolations should not be used. The trial mixes should have strengths both smaller and higher than f'_{cr}.
- The slump of the trial mix should be within 0.75 in (19 mm) of the permitted slump of the proposed work. Similarly, the air content should be within 0.5% for air-entrained concrete.
- For each test variable, at least three cylinders should be tested at 28 days or the age designated for the determination of f'_c.

The actual proportioning of the constituent materials is explained in Sec. 3.2.6.

3.2.5 Proportioning on the Basis of Maximum Water-Cement Ratio

The ACI code also allows proportioning using a maximum permissible water–cement ratio for the chosen f'_{cr}. The code recommends the maximum water–cement ratio that can be used for either air-entrained or non-air-entrained concrete (Table 3.6). These water–cement ratios cannot be used for lightweight concrete or concrete containing admixtures. The chosen water–cement ratio should also satisfy the durability requirements.

3.2.6 Computation of Mix Proportions

This section deals with the actual computation of the amounts of the various constituent materials that will result in a concrete with the required strength and durability. A number of procedures are available for proportioning normal-weight, lightweight, and heavyweight concretes. The method proposed by ACI Committee 211 for normal-weight concrete[7] is explained here. The other popular method used in the United States is the PCA method. This method is explained in a manual pub-

TABLE 3.6 Maximum Permissible Water-Cement Ratio for Concrete When Strength Data from Field Experience or Trial Mixtures Are Not Available

	Absolute water–cement ratio by weight	
Specified compressive strength f'_c, psi* (MPa)	Non-air-entrained concrete	Air-entrained concrete
2500 (17)	0.67	0.54
3000 (21)	0.58	0.46
3500 (24)	0.51	0.40
4000 (28)	0.44	0.35
4500 (31)	0.38	†
5000 (34)	†	†

*28day strength.
†For strengths above 4500 psi (31 MPa) (non-air-entrained concrete) and 4000 psi (28 MPa) (air-entrained concrete), concrete proportions shall be established by using trial mixes.
Source: From ACI Building Code.[6]

lished by Portland Cement Association.[8] The details of the procedure used in Britain, which is similar to the methods used in western Europe, Australia, and Asia, can be found in Neville.[9] For the sake of brevity, lightweight and heavyweight concretes are not discussed here. The details can be found in reports published by ACI Committee 211.[7,10]

The water–cement ratio is the most influential factor that affects strength. Hence the design charts and tables developed for mix proportioning are geared toward obtaining the minimum water–cement ratio that would produce a workable concrete. The following is the step-by-step procedure to estimate the quantities of various ingredients.

1. *Collection of background information:* The following information should be collected before starting the computations:
   - Sieve analysis data for fine and coarse aggregate including fineness modulus
   - Bulk specific gravity of aggregates, cement, and admixtures in solid (powder) form
   - Dry-rodded unit weight of coarse aggregate
   - Moisture content of fine and coarse aggregates
   - Ratio of solid-to-liquid contents of liquid (or slurry) admixtures
   - Special conditions such as permissible maximum water–cement ratio, minimum cement content, minimum air content, minimum slump, maximum size of aggregate, and strength requirements at early age
2. *Selection of slump:* If the slump is not specified, choose an appropriate value from Table 3.7. A minimum possible value should be chosen within the specified range.
3. *Selection of maximum size for aggregate:* For the same volume fraction a large maximum size of well-graded aggregate provides the least void space, requiring the least amount of mortar content. Hence the maximum possible aggregate size, consistent with the type of application, should be chosen. The maximum size should satisfy the following restrictions:

   $\ngtr \frac{1}{5}$ narrowest dimension between sides of form

   $\ngtr \frac{3}{4}$ of minimum clear spacing between reinforcing bars

   $\ngtr \frac{1}{3}$ depth of slab

4. *Estimation of amount of water and air:* The amount of water required to produce a given slump depends on the aggregate properties, the concrete temperature, and the amount of entrained air. The aggregate properties that influence the slump are maximum size, shape, and grading. The admixture can also influence the slump, the most notable being a high-range water-reducing admixture. Cement content, within the normal range, does not influence the slump. Table 3.8 can be used to estimate the approximate amount of water needed. It provides guidelines for both air-entrained and non-air-entrained concrete. Note that the table gives only approximate values, and the influence of any admixtures other than air-entraining admixtures is not considered. Only trial batches can establish the actual water and the corresponding air content.

TABLE 3.7 Recommended Slump for Various Types of Construction

Type of construction	Maximum slump*	Minimum slump
Reinforced foundation walls and footings	3 in (75 mm)	1 in (25 mm)
Plain footings, caissons, and substructure walls	3 in (75 mm)	1 in (25 mm)
Beams and reinforced walls	4 in (100 mm)	1 in (25 mm)
Building columns	4 in (100 mm)	1 in (25 mm)
Pavements and slabs	3 in (75 mm)	1 in (25 mm)
Mass concrete	2 in (50 mm)	1 in (25 mm)

*May be increased I in (25 mm) for methods of consolidation other than vibration.
Source: From Ad Committee 211.[7]

TABLE 3.8 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregates

Slump, in	Water, lb/yd³ of concrete for nominal maximum aggregate sizes							
	⅜ in[a]	½ in[a]	¾ in[a]	1 in[a]	1½ in[a]	2 in[a,b]	3 in[b,c]	6 in[b,c]
	Non-air-entrained concrete							
1–2	350	335	315	300	275	260	220	190
3–4	385	365	340	325	300	285	245	210
6–7	410	385	360	340	315	300	270	—
>7[a]	—	—	—	—	—	—	—	—
Approximate amount of entrapped air in non-air-entrained concrete, %	3	2.5	2	1.5	1	0.5	0.3	0.2
	Air-entrained concrete							
1–2	305	295	280	270	250	240	205	180
3–4	340	325	305	295	275	265	225	200
6–7	365	345	325	310	290	280	260	—
>7[a]	—	—	—	—	—	—	—	—
Recommended average[d] total air content for level of exposure, %								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5[e,f]	1.0[e,f]
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5[e,f]	3.0[e,f]
Severe exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5[e,f]	4.0[e,f]

[a]The quantities of mixing water given for air-entrained concrete are based on typical total air content requirements as shown for moderate exposure. These quantities of mixing water are for use in computing cement contents for trial batches at 68–77 °F. They are maximum for reasonably well-shaped angular aggregates graded within limits of accepted specifications. Rounded aggregate will generally require 30 lb less water for non-air-entrained and 25 lb less for air-entrained concretes. The use of water producing chemical admixtures (ASTM C494) may also reduce mixing water by 5% or more. The volume of the liquid admixtures is included as part of the total volume of the mixing water. The slump values of more than 7 in are only obtained through the use of water-reducing chemical admixture; they are for concrete containing nominal maximum-size aggregate not larger than tin.

[b]The slump values for concrete containing aggregate larger than 1/2 in are based on slump tests made after removal of particles larger than 1½ in by wet screening.

[c]These quantities of mixing water are for use in computing cement factors for trial batches when 3- or 6-in nominal maximum-size aggregate ta used. They are average for reasonably well-shaped coarse aggregate, well-graded from coarse to fine.

[d]Additional recommendations for air content and necessary tolerances on air content for control in the field are given in a number of ACt documents, including ACI 201, 345, 3lg, 301, and 302. ASTM C94 for ready-mixed concrete also gives air-content limits. The requirements in other documents may not always agree exactly, so in proportioning concrete consideration must be given to selecting an air content that will meet the needs of the job and also the applicable specifications.

[e]For concrete containing large aggregates that will be wet-screened over the 1½-in sieve prior to testing for air content, the percentage of air expected in the 1½-in material should be as tabulated in the 1½-in column. However, initial proportioning calculations should include the air content as a percent of the whole.

[f]When using large aggregate in low cement factor concrete, air entrainment need not be detrimental to strength. In most cases the mixing water requirement is reduced sufficiently to improve the water–cement ratio and to thus compensate for the strength-reducing effect of air-entrained concrete. Generally, therefore, for these large nominal maximum sizes of aggregate, air contents recommended for extreme exposure should be considered even though there may he little or no exposure to moisture and freezing.

[g]These values are based on the criteria that 9% air is needed in the mortar phase of the concrete. if the mortar volume will be substantially different from that determined in this recommended practice, it maybe desirable to calculate the needed air content by taking 9% of the actual mortar volume.

Note: ho 25.4 mm; 1 lb/yd³ = 0.59 kg/m³.

Source: From ACI Committee 211.

TABLE 3.9 Relationships between Water-Cement Ratio and Compressive Strength of Concrete

Compressive strength at 28 days, psi* (MPa)	Water-cement ratio, by weight: Non-air-entrained concrete	Water-cement ratio, by weight: Air-entrained concrete
6000 (41)	0.41	—
5000 (35)	0.48	0.40
4000 (28)	0.57	0.48
3000 (21)	0.68	0.59
2000 (14)	0.82	0.74

*Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table 3.g. For a constant water–cement ratio, the strength of concrete is reduced as the air content is increased. Strength is based on 6- by 12-in cylinders moist cured 28 days as 73.4 ± 3°F (23 ± 1 .7°C) in accordance with Sec. 9(b) of ASTM C3 I, *Making and Curing Concrete Compression and Flexure Test Specimens in the Field*.

Source: From Act Committee 211.[7]

5. *Selection of water–cement, or water–cementitious-materials, ratio*. Water-cement ratio is a classical term. When mineral admixtures such as fly ash or silica fume are used, they could be considered as part of the cementitious materials. In this case the water is computed based on the weight of the cementitious (cement + fly ash or silica fume) materials, rather than just the weight of cement. As mentioned earlier, the amount of water used should satisfy both strength and durability requirements. Information given in Table 3.9 can be used as an approximate first step to establish the water–cement ratio. Since the strength is also affected by factors such as aggregate and cement types and by the properties of other cementitious materials, the values shown in Table 3.9 should be used only as a guideline. It is highly desirable to develop a water–cement (cementitious materials) ratio for the particular type of materials to be used in the proposed work.

   If the structure to be built is going to be exposed to severe environmental conditions, water–cement (cementitious materials) ratios should be limited to the values shown in Table 3.10. As mentioned earlier, a lower water content typically reduces permeability and improves overall durability.

6. *Computation of cement content:* Once the amount of water and the water–cement ratio are established, the computation of the amount of cement becomes a simple division of water content by water–cement ratio. The cement content should also satisfy any special minimum cement content stipulated in the specification.

TABLE 3.10 Maximum Permissible Water-Cement Ratios for Concrete in Severe Exposures

Type of structure	Structure wet continuously or frequently and exposed to freezing and thawing*	Structure exposed to seawater or sulfates†
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 1-in (25 mm) cover over steel	0.45	0.40
All other structures	0.50	0.45

*Concrete should also be air-entrained.

†If sulfate-resisting cement (type It or type v of ASTM C150) is used, the permissible water–cement ratio may be increased by 0.05.

Source: From ACI Committee 211.[7]

7. *Estimation of coarse aggregate content:* Typically the use of more coarse aggregate per unit volume of concrete leads to better economy. The larger the size of the particles in coarse aggregate and the finer the sand, the more volume fraction of coarse aggregate can be incorporated without sacrificing workability. The volume fractions of coarse aggregate that will produce a workable mix for various maximum aggregate sizes and fineness moduli of sand are shown in Table 3.11.

   For the chosen maximum aggregate size, say 1 in (25 mm), and the fineness of the sand to be used, say 2.6, the table provides the volume fraction of coarse aggregate in the dry-rodded form, 0.69. For 1 yd^3 (0.76 m^3) of concrete, the volume of coarse aggregate is 0.69 × 27, or 18.63 ft^3 (0.52 m^3). The corresponding weight is obtained by multiplying 18.63 by the dry-rodded unit weight. The values shown in Table 3.11 can be reduced by up to 10% to improve the workability for special circumstances, such as pumping or concreting members with congested reinforcement.

8. *Estimation of fine aggregate content:* At the completion of step 7, the amounts of all ingredients, except fine aggregate, have been estimated. Hence if the unit weight of fresh concrete is known, the weight of fine aggregate can be estimated by subtracting the total weight of all other ingredients from the weight of fresh concrete. This type of computation is called the weight method. In the absence of previous experience, a first estimate of the unit weight of concrete can be obtained using Table 3.12. The table covers both air-entrained and non-air-entrained concrete. Medium-rich concrete and a coarse aggregate specific gravity of about 2.7 have been assumed for developing Table 3.12. Even rough estimates of unit weight were found to provide satisfactory results for trial mixes.

   There is another procedure, called the absolute volume method, in which the volume of fine aggregate is computed by subtracting the volumes of all other ingredients from the unit volume of fresh concrete. This method is considered more accurate, but the specific gravity of all ingredients is needed prior to the computation.

9. *Adjustments for aggregate moisture:* In most cases the stock aggregates retain some moisture. The computations of aggregate weights in steps 7 and 8 are based on saturated surface-dry conditions. Hence the weight of moisture present in the aggregate should be accounted for. The easiest way to make the correction is to adjust the weight of aggregates for moisture. For example,

TABLE 3.11 Volume of Dry-Rodded Coarse Aggregate* per Unit Volume of Concrete for Different Fineness Moduli of Sand

Maximum size of aggregate, in (mm)	Fineness modulus of sand 2.40	2.60	2.80	3.00
⅜ (9)	0.50	0.48	0.46	0.44
½ (13)	0.59	0.57	0.55	0.53
¾ (19)	0.66	0.64	0.62	0.60
1 (25)	0.71	0.69	0.67	0.65
1½ (38)	0.75	0.73	0.71	0.69
2 (50)	0.78	0.76	0.74	0.72
3 (76)	0.82	0.80	0.78	0.76
6 (152)	0.87	0.85	0.83	0.81

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C29, Unit Weight of Aggregate." These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction the volume may be increased by about 100%. For more workable concrete, such as may sometimes he required when placement is to be by pumping, it maybe reduced by up so 10%.

Source: From Ad Committee 211.[7]

TABLE 3.12 First Estimate of Weight of Fresh Concrete

Maximum size of aggregate, in (mm)	Concrete weight,* lb/yd^3	
	Non-air-entrained concrete	Air-entrained concrete
⅜ (9)	3840 (2266)	3710 (2189)
½ (13)	3890 (2295)	3760 (2218)
¾ (19)	3960 (2336)	3840 (2266)
1 (25)	4010 (2366)	3850 (2272)
1½ (38)	4070 (2401)	3910 (2307)
2 (50)	4120 (2431)	3950 (2331)
3 (76)	4200 (2478)	4040 (2384)
6 (152)	4260 (2513)	4110 (2425)

*Values calculated for concrete of medium richness (550 lb of cement per cubic yard or 325 kg per cubic meter) and medium slump with aggregate specific gravity of 2.7. Water requirements based on values for 3 to 4 in (75 to 100 mm) of slump are given in Table 3.8. If desired, the estimated weight may be refined as follows when necessary information is available. For each 10-lb (4.5-kg) difference in mixing water from Table 3.8 values for 3 to 4 in (75 to 100 mm) of slump, correct the weight per cubic yard by 15 lb (6.8 kg) in the opposite direction; for each 100-lb (45.4-kg) difference in cement content from 550 lb (250 kg), correct the weight per cubic yard by 15 lb (6.8 kg) in the same direction; for each 0.1 by which aggregate specific gravity deviates from 2.7, correct the concrete weight by 100 lb (45.4 kg) in the same direction.

Source: From ACI Committee 211.[7]

if the moisture content of coarse aggregate is 2%, the amount of coarse aggregate needed for the batch should be increased by 2%. The actual amount of water, which is 2% of the weight of the coarse aggregate, should be subtracted from the water to be used for the mix. The reduction in water is necessary to maintain the water–cement ratio chosen in step 5. Similar adjustments should also be made for fine aggregate.

10. *Trial batch adjustments:* Since the estimation of the various ingredients is only approximate, adjustments are needed to obtain the mix that satisfies the workability and strength requirements. Fresh concrete should be tested for slump, segregation of aggregates, air content, and unit weight. The hardened concrete, cured under standard conditions, should be tested for strength at the specified age. The test methods are described in Sec. 3.3. In some instances it may take several trials to obtain a satisfactory mix. The following guidelines may be used for the adjustment of ingredients. The recommended numerical values are for 1 ft^3 (0.028 m^3) of concrete.

- If the slump of the trial mix is not correct, increase or decrease the estimated water by 10 lb (4.5 kg) for each 1-in (25-mm) required increase or decrease of slump.
- If the desired air content is not achieved, adjust the admixture content. Since the amount of air content influences the slump, the water content should also be changed with the change in air-entraining admixture. Change the water content by 5 lb (2.3 kg) for each 1% of air.
- Adjust the yield by using the unit weight of fresh concrete.

The following example further illustrates the mix proportioning process.

Example 3.2 To compute the mix proportions of normal-weight concrete, we use these specifications:

Required (28-day) average compressive strength f'_c	4100 psi (28 MPa)
Type of construction	Reinforced concrete footing
Slump	Minimum 3 in (75 mm)
Exposure condition	Below ground; no freezing, no exposure to chemicals
Maximum size of aggregate	1.5 in (38 mm)

Solution

1. *Background information on properties of constituent materials:*

Cement	ASTM type I; bulk density 196 lb/ft^3 (3136 kg/m^3)
Coarse aggregate	Maximum size 1.5 in (38 mm)
	Bulk density 168 lb/ft^3 (2690 kg/m^3)
	Dry-rodded unit weight 100 lb/ft^3 (1600 kg/m^3)
	Moisture content 2.0% over SSD condition
Fine aggregate	Bulk density 160 lb/ft^3 (2560 kg/m^3)
	Fineness modulus 2.8
	Moisture content 3.0% over SSD condition

2. *Selection of slump:*

$$\text{Specified minimum} = 3 \text{ in } (75 \text{ mm})$$

The specified minimum value is consistent with the 3 in (75 mm) recommended for reinforced concrete foundation walls and footings in Table 3.7.

3. *Maximum aggregate size:*

$$\text{Specified maximum} = 1.5 \text{ in } (38 \text{ mm})$$

4. *Estimation of amount of water and air:* Since there is no freezing or exposure to chemicals, non-air-entrained concrete is assumed to be adequate. Using Table 3.8, the amount of water for 1 yd^3 (0.76 m^3) = 300 lb for 3-to 4-in slump (136 kg^3 for 75- to 100-mm slump).
5. *Selection of water–cement ratio:* Using Table 3.9, the water–cement ratio for the required average strength is 0.56. The value of 0.56 is obtained by interpolating linearly between 4000 and 5000 psi (27 and 34 MPa). Note that this value is only a first estimate.

6. *Cement content:*

$$\text{Cement content} = \frac{\text{amount of water}}{\text{water–cement ratio}}$$

$$= \frac{300}{0.56} = 536 \text{ lb/yd}^3 \text{ } (316 \text{ kg/m}^3)$$

7. *Coarse aggregate content:* Using Table 3.11:

Volume fraction of dry-rodded aggregate for 1.5-in maximum-size aggregate and a sand fineness modulus of 2.8	0.71
Volume of coarse aggregate per cubic yard of concrete	0.71×27 ft^3 = 19.2 ft^3 (0.54 m^3)
Weight of coarse aggregate (since dry-rodded unit weight is 100 lb/ft^3 or 1600 kg/m^3)	19.2×100 1920 lb/yd^3 (1133 kg/m^3)

8. *Estimation of fine aggregate:*

Estimated unit weight of concrete, from Table 3.12	4070 lb/yd^3 (2401 kg/m^3)
Weight of cement (step 6)	536 lb/yd^3 (316 kg/m^3)

Weight of coarse aggregate (step 7)	1920 lb/yd^3 (1133 kg/m^3)
Weight of water (step 4)	300 lb/yd^3 (177 kg/m^3)
Weight of fine aggregate	4070 – (536 + 1920 + 300) 1314 lb/yd^3 (775 kg/m^3)

9. *Adjustments for aggregate moisture:* The moisture content of coarse aggregate above SSD is 2%. In order to obtain 1920 lb of dry-rodded weight, we have to use 1.02 × 1920 lb of stock sample.

Weight of stock sample of coarse aggregate	1.02 × 1920 lb/yd^3 = 1958 lb/yd^3 (1155 kg/m^3)
Weight of stock sample of fine aggregate	1.03 × 1314 lb/yd^3 = 1353 lb/yd^3 (798 kg/m^3)
Free water from coarse aggregate	1958 – 1920 = 38 lb/yd^3 (22 kg/m^3)
Free water from fine aggregate	1353 – 1314 = 39 lb/yd^3 (23 kg/m^3)
Water to be added	300 – 39 – 38 223 lb/yd^3 (132 kg/m^3)

The final weights of the constituent materials per cubic yard (0.76 m^3) are:

Cement	536 lb (243 kg)
Fine aggregate	1353 lb (614 kg)
Coarse aggregate	1958 lb (888 kg)
Water	223 lb (101 kg)
Total	4070 lb (1846 kg)

Note that the unit weight of concrete is the same even after the corrections are made for the moisture contents of aggregates.

10. *Trial batches:* Trial batches should be made using about 0.1 yd^3 (0.076 m^3) of concrete to determine the properties of fresh and hardened concrete. If the slump is too high or too low, corrections can be made immediately. Since it takes 28 days to obtain strength results, it may be more efficient to make two or three trial mixes with various water contents rather than waiting for the results from a single mix.

3.3 QUALITY ASSURANCE

Quality assurance procedures are needed in order to ascertain that the concrete placed in the actual structures satisfies the required specifications. As mentioned earlier, the quality of the concrete is influenced by a large number of variables. Hence continuous monitoring of properties is needed. The quality and physical properties of the constituent materials, namely, cement, aggregates, water, and admixtures, should also be checked periodically. ASTM standards are available for the test procedures needed to evaluate the properties of constituent materials and concrete. The properties of constituent materials were discussed in Sec. 3.1. This section deals with the test methods for fresh and hardened concrete and the quality assurance procedures. The frequency of testing depends primarily on the type of structure. For buildings, samples should be taken at least once a day or once for every 150 yd^3 (115 m^3) of concrete. If large surface areas are being constructed, the ACI code recommends at least one test for each 5000 ft^2 (464 m^2). For structures such as nuclear containment buildings where failure could result in disastrous consequences, more stringent quality control is needed.

3.3.1 Tests for Fresh Concrete

The most universally used test for fresh concrete is the slump test. It measures only the consistency of concrete. However, this test is used as an indicator of workability. The slump test is also used to assure the uniformity from batch to batch. The other two tests used for fresh concrete are the V-B test and the compaction factor test. These two tests are used primarily in the laboratory environment, whereas the slump test is used both in the laboratory and in the field.

Slump Test

The slump test is covered in ASTM C143. In this test a sample of freshly mixed concrete is placed inside a mold, which has the shape of the frustrum of a cone. The concrete is compacted using a standard procedure and the mold is raised to allow the concrete to slump. The amount by which the concrete slumps is measured in inches or millimeters and is called the slump value. The following are the pertinent details of this test.

The test equipment consists of (1) mold (Fig. 3.5), (2) a tamping rod, (3) a ruler with a reading accuracy of at least 0.125 in (3 mm), and (4) a nonabsorbent rigid pan. The mold shown in Fig. 3.5 should be clean and free of any projections such as rivets or dents on the inside surface. The top and bottom faces should be parallel to each other and perpendicular to the vertical axis. The tamping rod consists of a ⅝-in (16-mm)-diameter and 24-in (610-mm)-long steel bar with hemispherically rounded ends.

The test procedure consists of the following steps:

1. Dampen the mold and place it on a rigid, flat surface. The surface should be moist and nonabsorbent. Stand on the two foot pieces in order to hold the mold in place during the filling operation.
2. Pour concrete until the mold is filled to one-third of the volume. One third of the volume is reached when the mold is filled to a height of 2⅝ in (66 mm). Rod this bottom layer of concrete with 25 strokes of the tamping rod. Apply approximately half of the strokes near the perimeter and progress spirally toward the center. The bottom layer should be rodded throughout its depth.
3. Pour concrete to fill the mold to two-thirds of its volume. To reach this volume, the mold should be filled to a height of 6⅛ in (155 mm) from the bottom. Rod this layer using 25 strokes. The strokes should just penetrate into the underlying bottom layer.
4. Fill the remaining part of the mold, which forms the top layer. Again rod this top layer with 25 uniformly distributed strokes. The top level of concrete should stay slightly above the top surface of the mold. When concrete subsides below the top surface because of compaction, add more concrete. The strokes of the tamping rod should just penetrate into the second layer.

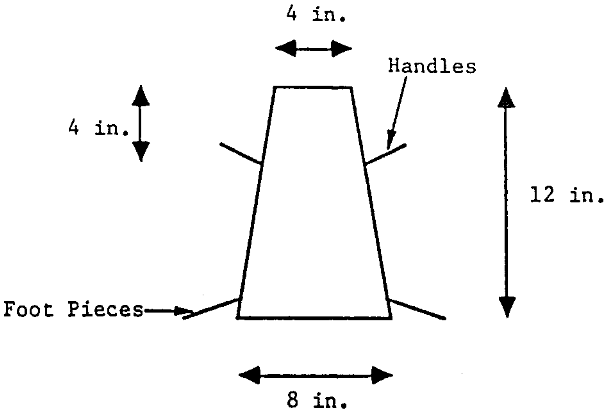


FIGURE 3.5 Salient features and dimensions of slump cone.

5. Strike off the excess concrete on the top using the tamping rod. A screeding and rolling motion should be used to obtain a relatively smooth top surface.
6. Remove the mold by lifting it in a vertical direction. Lateral or torsional motion should be avoided. The lifting operation should be complete in 5 ±2 s. The entire operation of filling and lifting the mold should be completed without interruption in 2.5 mm.
7. When the mold is removed, the concrete will slump down. If the concrete is stiff, it might move only a fraction of an inch. If the concrete is of flowing consistency, the whole mass will collapse. Measure the difference between the original 12 in (300 mm) and the height of the slumped concrete and record it to the nearest 0.25 in (6 mm). This value is called the slump of the concrete. If the concrete slides to one side, the slump value should be disregarded and the test repeated.

V-B Test

The equipment for the V-B test consists of a vibration table, a cylindrical pan, and a glass or plastic disk, Fig. 3.6. The glass or plastic disk is attached to a free-moving rod which serves as a reference end point. The cone is placed in the cylindrical pan and filled with concrete. The cone is removed, the disk is brought into position on the top of the concrete, and the vibrating table is set into motion. When the table starts vibrating, the concrete in the conical shape remolds itself into a cylinder. The time required for remolding the concrete into cylindrical shape, until the disk is completely covered with concrete, is recorded in seconds and reported as V-B time. Since the V-B test is conducted using vibration, the V-B time is a better indicator of workability when in the actual construction the concrete is compacted using vibration. However, this test is not easily adoptable for testing on the construction site.

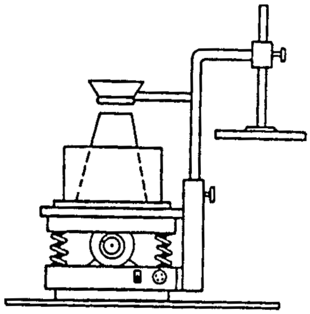


FIGURE 3.6 V-B apparatus.

Compaction Factor Test

This test measures the amount of compaction achieved when the concrete mixture is subjected to a standard amount of compacting work. The result is expressed as a compacting factor, which is a ratio of the density actually achieved in the test to the density of the same concrete under fully compacted conditions. This test, which was developed in Britain, is not as popular as the other two tests.

The three tests mentioned primarily provide an indication of the workability of concrete. In the case of air-entrained concrete the air content of fresh concrete has to be measured to assure that the concrete has the specified amount. Other properties measured using fresh concrete include unit weight, setting time, and concrete temperature. The air content can be determined using either the pressure or the volumetric method. These tests are described in ASTM C23 1 and C 173, respectively. The gravimetric method is covered in ASTM C 138. Only the volumetric method should be used for lightweight concrete. The unit weight, setting time, and temperature measurements are covered in ASTM C 138, C403, and C 1064, respectively.

3.3.2 Tests for Hardened Concrete

Hardened concrete should meet the minimum-strength requirements and should be durable. The compressive strength test is the most widely used quality control test. Various types of theoretical and empirical relationships have been developed to relate the compressive strength to other properties of concrete so that other tests need not be conducted for every situation. However, test methods exist for determining strength at various modes of loading, such as tension, flexure, shear, and torsion, and to ascertain durability under freeze-thaw conditions. The most commonly used tests and the corresponding ASTM standards are as follows:

Compressive strength of cylindrical concrete specimens (ASTM C39)

Flexural strength of concrete (ASTM C78, C293)

Splitting tensile strength (ASTM C496)

Modulus of elasticity and Poisson's ratio (ASTM C469)

Length change of hardened paste or mortar (ASTM C 157)

Creep under compression (ASTM C512)

Air void parameter (ASTM C457)

Resistance of concrete to rapid freezing and thawing (ASTM C666)

Resistance to scaling (ASTM C672)

Only the compressive strength test is described here. The details for the other tests can be found in the appropriate ASTM standards.

3.3.3 Compressive Strength Test

The compressive strength test can be conducted using cylinders, cubes, or prisms. In the United States, 6- by 12-in (150- by 300-mm) cylinders are the most popular test specimens for obtaining the compressive strength. In recent years 4- by 8-in (100- by 200-mm) cylinders have also been used, especially for high-strength concrete. In special circumstances smaller [3- by 6-in (75- by 150-mm)] and larger [(up to 24- by 48-in (600- by 1200-mm)] cylinders are also being be used. The cylinders are typically made in the laboratory for trial mixes. The cylinders made for quality control are usually made on the construction site. In either case, standard procedures should be followed for the casting, curing, capping, and testing of cylinders. This section presents the salient features of the procedure to be used for the preparation and testing of cylinders in the laboratory. For field cast and cured specimens appropriate ASTM standards should be followed for the sampling, making, and curing of cylinders.

Mixing, Molding, and Curing of Concrete Test Specimens

The concrete can be mixed either by hand or by machine. Hand mixing should be avoided as much as possible because it is very difficult to achieve uniform mixing. At least 10% more concrete than needed for molding should be mixed. Hand mixing, which is not to be used for air-entrained and no-slump concrete, and quantities exceeding 0.25 ft^3 (0.007 m^3) can be achieved by using the following steps:

1. Mix cement and fine aggregate in a watertight, clean, and damp metallic pan until the contents are thoroughly blended.
2. Add the coarse aggregate to the cement-sand mix and mix until the coarse aggregates are uniformly dispersed.
3. Add water and the admixtures, if any, and mix the contents until they become a homogeneous mass.

The mixing sequence for machine mixing is as follows. It is assumed that a drum-type mixer is used for mixing.

1. Place coarse aggregate and half the mixing water in the mixer and mix the contents for 30 s.
2. Add fine aggregate, cement, and the remaining water and mix for 3 min.
3. Stop the mixer and rest the mixture for 3 min.
4. Mix for additional min.

If admixtures are used, they should be mixed with water before starting the mixing process. The open end of the mixer should be covered with a pan to avoid evaporation. Typically, some concrete sticks to the sides of the drum. This concrete contains more mortar than the discharged concrete. Hence the concrete used for making samples could contain less mortar. This is particularly true when small quantities are mixed. This could be avoided by mixing a similar batch of concrete and disposing of the contents. The test batch is then mixed without cleaning the mixer. This process is called buttering the mixer. If the exact amount of mortar that sticks to the sides can be established, the mix can be adjusted to contain this excess mortar. This process is more difficult because it is not easy to establish the accurate amount of mortar that will stick to the sides.

Molding of specimens should be done in a place that is very near to the storing place where the specimens will be kept for the first 24 h. Molds should be made of a material which is nonabsorbent and nonreactive with concrete. They should be dimensionally stable and watertight. Reusable molds should be coated lightly with mineral oil for easy removal of the specimens.

The mixed concrete is placed in the mold in layers and compacted to form the test specimens. For a cylinder height of 12 in (300 mm) or smaller, casting should be done in three equal layers if compaction is done by rodding. If compaction is done by vibration, two layers should be used. For larger cylinders, more layers may be needed. If the slump is greater than 3 in (75 mm), compaction by rodding is preferable. If the slump is less than 1 in (25 mm), vibration should be used for compaction. If the slump is in the range of I to 3 in (25 to 75 mm), either rodding or vibration can be used. If the cylinder diameter is 4 in (100 mm) or less, only external vibration should be used. For larger cylinders, either internal or external vibration can be used.

The following points should be observed when compaction is done by rodding.

For 3-by 6-or 4-by 8-in (75- by 150- or 100- by 200-mm) cylinders, use a ⅛-in (9-mm)-diameter 12-in (300-mm)-long metal rod with hemispherical ends. For 6- by 12-in (150- by 300-mm) cylinders use ⅝-in (15-mm)-diameter 12-in (300-mm)-long metal rod with hemispherical ends.

Each layer should be rodded 25 strokes, uniformly distributed over the cross section. The bottom layer should be rodded throughout the depth. While rodding the upper layers, allow the rod to penetrate about ½ in (12 mm) into the underlying layer for 3-by 6- and 4-by 8-in (75- by 150- and 100- by 200-mm) cylinders and about 1 in (25 mm) for 6- by 12-in (150- by 300-mm) cylinders.

After rodding each layer, tap the outside of the mold about 15 times with a rubber mallet to close any holes left by the tamping rod and to release large entrained air bubbles. Spade the top of the concrete lightly before placing the subsequent layer.

If the compaction is done by vibration, fill the mold in a number of layers of equal height and vibrate them. Place all the concrete for each layer before starting the vibrating equipment. When adding the final layer, do not overfill more than 0.25 in (6 mm). The duration of vibration required depends on the workability of the concrete and the effectiveness of the vibration. Compaction can be assumed to be complete if the top surface is smooth. Overvibration should be avoided. Each layer should be vibrated to the same extent. When using an internal vibrator, use three insertions for each layer. Allow the vibrator to penetrate through the layer being vibrated and approximately 1 in (25 mm) into the underlying layer. The vibrator should be pulled out slowly so as to avoid air pockets. After vibrating each layer, tap the side 10 to 15 times with a rubber mallet to release entrapped air bubbles. When external vibration is used, ensure that the mold is held securely against the vibrating surface.

At the end of consolidation, strike off the top surface with a trowel. Flatten the top surface such that it is level with the rim of the mold and has no depressions or projections larger than 0.125 in (3 mm). The top surface of the freshly made cylinder may be capped with a stiff cement paste.

After finishing, cover the specimens with a nonabsorptive and nonreactive plate or an impervious plastic sheet. Remove the specimens from the mold 24 ± 8 h after casting. Cure the specimens at 73 ± 3°F (23 ± 2°C) from the time of removal until testing. Curing can be done by immersing the specimens in lime-saturated water or in a room maintained at 100% relative humidity by using moist sprays.

Capping of Cylindrical Specimens

The cylinders should be capped before testing to assure two flat surfaces that are perpendicular to the axis of the cylinders (ASTM C617). The capping material should be at least as strong as the concrete being tested. Freshly made specimens can be capped with cement paste. This is not usually done because it is very difficult to achieve the required accuracy. Normally capping is done after the cylinders are cured. Common capping materials are high-strength gypsum cement or molten sulfur. Standard equipment is available for melting the sulfur and for the alignment of caps so that they are perpendicular to the axis of the cylinder.

For high-strength concrete, with compressive strength greater than 12,000 psi (80 MPa), grinding of the ends is recommended. Grinding prevents the interference of the capping materials.

The tolerance for level surfaces is 0.002 in (0.05 mm). The caps should be about 0.125 in (3 mm) thick. They should not be more than 0.31 in.(8 mm) thick. Gypsum plaster should cure at least for 4 h prior to testing. The minimum curing period for sulfur caps is 2 h.

Test Apparatus

The test apparatus consists of a testing machine, scale, and calipers. If the stress-strain relationship is measured, then a compressometer setup is also needed. Most of the testing machines are driven by hydraulic fluid pressure. These machines can be operated so as to apply the load at a constant rate at a certain number of psi per minute or at a constant displacement rate. Some of the machines with servo control mechanisms can be run under different controls such as displacement or strain control.

A compressometer setup can be used to measure the deformation and, hence, the strain at a given load. This device is needed to obtain the stress-strain relationship and the Young's modulus of elasticity. The setup consists of two yokes (rings) (Fig. 3.7). The bottom ring (yoke) is attached to the cylinder using three screws. The top ring is attached to the cylinder using two screws placed at diametrically opposite points. This ring can rotate about the two screws. One end of the rotatable ring is connected to the bottom ring using a pivot rod. The pivot rod does not allow any translation but allows the ring to rotate. A dial gauge or other deformation-measuring instrument, such as an LVDT, is attached on the other side (Fig. 3.7). When the cylinder deforms, the two points where the screws of the top ring are attached move downward. Since one end cannot move due to the pivot rod, the ring rotates. Due to symmetry, if the deformation along the gauge length is S, the dial gauge will record 2S, hence improving the accuracy of the measurements, specially for small deformations. The rings are placed in proper position using spacing bars. Normally the gauge length for 6- by 12-in (150- by 300-mm) cylinders is 6 in (150 mm).

Specimen Preparation

As mentioned earlier, the cylinders should be capped so that the ends are flat and perpendicular to the axis of the cylinder. The diameter of one cylinder should not vary by more than 2% of that of the companion cylinder. If the difference is more than 2%, the cylinders should be discarded. The diameter should be measured to the nearest of 0.01 in (0.25 mm) by averaging two diameters measured at

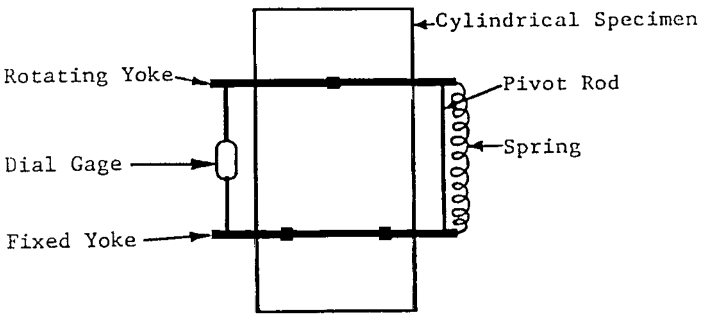


FIGURE 3.7 Compressometer arrangement.

right angles to each other at about midheight of the specimen. If the length-to-diameter ratio is less than 1.8 or more than 2.2, or if the height is used to compute the volume of the cylinder, the height should be measured to the nearest of "0.05 diameter." The test specimen should be in saturated surface-dry condition. The specimens should be tested at the specified age. The following are the ASTM permissible tolerances:

Test age, days	Permissible tolerance, hours
1	±0.5
3	±2.0
7	±6.0
28	±20.0
90	±48.0

Test Procedure

1. Clean the upper and lower bearing blocks and place the specimen on the lower block. Carefully align the axis of the cylinder with the center of thrust of the spherically seated upper block. Most testing machines have concentric circles marked on the bearing plates, and hence centering is not a difficult task.
2. Start the machine and raise the lower block so that the top block comes in contact with the cylinder. Once the top and bottom plates are touching the cylinder, lock the top plate to prevent its rotation.
3. Start applying load without shock. The loading should be a continuous process. For hydraulically operated machines the loading rate should be in the range of 20 to 50 psi/s (138 to 344 kPa/s). The loading rate should be constant. Adjustments should not be made, even when the specimen begins to fail.
4. Record the maximum load, type of failure, and appearance of the concrete. Five types of failures shown in Fig. 3.8, cover most of the failure modes.
5. Compute the compressive strength by dividing the maximum load by the area of cross section. If the length-to-diameter ratio is less than 1.8, apply the following correction factors. Cylinders with lower length-to-diameter ratios resist more loads due to a different strain distribution along the length.

Length/diameter	1.75	1.50	1.25	1.00
Correction factor	0.98	0.96	0.93	0.87

The values for other length-to-diameter ratios can be interpolated. These correction factors are applicable for normal-strength concrete with strengths in the range of 2000 to 6000 psi (14 to 42 MPa) and lightweight concrete with densities in the range of 100 to 120 lb/ft^3 (1600 to 1920kg/m^3). Report the compressive strength to the nearest 10 psi (0.1 MPa).

To obtain the modulus of elasticity the following additional steps are needed.

6. Prior to testing for the modulus of elasticity determine the compressive strength of concrete.
7. Attach the compressometer to the cylinder and place the specimen in the machine.
8. Load the specimen at a rate of 35 ± 5 psi/s (240 ± 34 kPa/s) to approximately 40% of the ultimate load. If the companion cylinders are not available, load the cylinders until the longitudinal strain reaches a value shown in Table 3.13. Immediately upon reaching the designated load, reduce the load to zero at the same rate at which it was applied.
9. Reload the specimen and record loads and deformations at predefined intervals. A sufficient

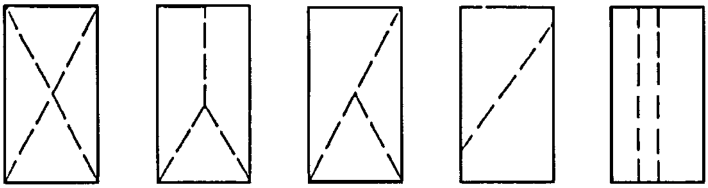


FIGURE 3.8 Types of fractures.

number of readings should be taken to establish the stress-strain relationship. Load the specimen up to about 50% of its capacity. Reduce the load to zero and repeat the measurements. If the two sets of measurements are the same, then proceed to compute the modulus E_c. If there is a significant difference between the two sets, an additional set of readings should be taken.

10. After obtaining a consistent set of load-deformation information, remove the compressometer and test the cylinder to failure.
11. Compute the stress by dividing the load by the area and the strain by dividing the deformation by the gauge length. Plot the stress-strain curve. The curve should be approximately linear, at least up to 40% of the failure load.
12. Compute E_c using the equation

$$E_c = \frac{S_2 - S_1}{\epsilon_2 - 0.00005} \tag{3.10}$$

where E_c = modulus of elasticity
S_1 = stress corresponding to a longitudinal strain of 0.00005
S_2 = stress corresponding to 40% of ultimate load
ϵ_2 = longitudinal strain corresponding to S_2.

Report the modulus of elasticity to the nearest 50,000 psi (0.5 GPa).

TABLE 3.13 Maximum Strain Values

Strain at age indicated × 10^{-6}			
Unit weight at time of test, lb/ft^3 (kg/m^3)		7 days or more	Less than 7 days
205	(3282) and over	300	200
165–204	(2642–3266)	375	250
135–164	(2161–2626)	450	300
115–134	(1841–2145)	525	350
105–114	(1681–1825)	600	400
95–104	(1521–1665)	675	450
85–94	(1361–1505)	750	500
75–84	(1201–1345)	825	550

3.3.4 Statistical Variation of Compressive Strengths and Quality Control

Due to the influence of various factors, there is always a variation in the strengths of concrete. The variations occur in all modes of loading, including compression, splitting tension, flexure, shear, and torsion. In most construction practices the variations of compressive strength are used for quality control. Hence only compressive strength variations and their tracking procedures are discussed here. In some instances involving pavements, flexural strengths are measured and used for quality assurance. The statistical procedure used for compressive strength can also be used for flexural strength.

As mentioned in Sec. 3.2.1, the variation of compressive strength is usually assumed to follow a normal distribution. The properties of the normal curve, sample average, and sample standard deviation are used to compute the required average compressive strength to satisfy the specified strength requirements. Hence at the beginning of a project, the strength level of the concrete being produced is based on the calculation of the required average strength. This hypothetical production strength is based on the assumption that the variables affecting the strength of concrete will be the same in the future as they have been in the past. As the data become available, the average of the actual production strength will replace the hypothetical average strength and the standard deviation. If the average and the standard deviation obtained during the project are about the same as the values used in the computation, the project average strength should be carefully maintained.

If the project average strength is smaller than the required average strength while the standard deviation is the same, the percentage of tests below the specified strength will be greater than the acceptable value, and steps must be taken to increase the average strength of the concrete. The average strength should also be increased if the standard deviation of the project is greater than the assumed standard deviation. On the other hand, if the project average is higher, or if the standard deviation is lower; the average strength could be reduced.

Continuous evaluation of the data is necessary in order to assure that the concrete being placed satisfies the specified strength requirements. An updated determination of the average strength and the standard deviation will provide an indication of how well the quality control procedures are working. At any given time the approximate percentage of tests falling below the specified strength can be calculated using the equation

$$p = \frac{\bar{x} - f'_c}{S} \tag{3.11}$$

where p = probability factor
$\bar{x}$ = average strength
f'_c = specified strength
S = standard deviation

Note that Eq. (3. 11) is a transformed version of Eq. (3. 1). The variables have been changed to permit using the project average strength x and the project standard deviation S. In Eq. (3. 11), if p = 2.33, the probability of the cylinder strength (average of two cylinders) falling below f'_c is 1%. Probabilities for other values can be estimated using statistical tables in statistics books or tables provided in the report of ACI Committee 214.[5]

In order to satisfy the equations of ACT code 3 18-92 [Eqs. (3.2) and (3.3)], the project average and the standard deviation should satisfy the following two inequalities:

$$1.34 < \frac{\bar{x} - f'_c}{S} \quad \text{or} \quad \frac{\bar{x} - f'_c}{S} \geq 1.34 \tag{3.12}$$

$$2.33 < \frac{\bar{x} - (f'_c - 500)}{S} \quad \text{or} \quad \frac{\bar{x} - (f'_c - 500)}{S} \geq 2.33 \tag{3.13}$$

Quality control charts are quite often used for a visual picture of concrete performance. Three typical quality control charts used in the industry are shown in Fig. 3.9. Various forms of other charts are also being used. With the advent of tabletop computers it is very easy to develop, maintain, and update these charts. The charts can also be transferred from location to location using phone lines.

Figure 3.9(*a*) shows the variations of (1) the individual cylinder strength, (2) the average of two cylinders, and (3) specified and required average strengths. The number of low tests can be easily picked out from this chart. Note that the number of low tests is computed using the average of two cylinders (solid line). If the volume of concrete produced requires more than one test per day, the average of all the tests (instead of two) can be plotted for that day. The charts can also be plotted using calendar dates.

Figure 3.9(*b*) and (*c*) is plotted using the values of Fig. 3.9(*a*). Each point in Fig. 3.9(b) represents the average of the previous five tests. The number of tests used to calculate this moving average depends on the type of job and the number of tests per day. In Fig. 3.9(*b*) some of the high variabilities of individual tests are suppressed. This chart can be used to identify the influence of major factors such as seasonal changes and changes in materials. Figure 3.9(*c*) shows the moving average range of the previous 10 groups of cylinders. Considerable change in this chart is an indication of high variability.

The control charts are valuable tools not only for the current project, but also for future projects. As discussed earlier, good records can be used for the computation of f'_{cr} and mix proportions instead of trial mixes, thus saving a considerable amount of time and effort.

The variability caused by the test procedure is always a concern in quality control. It is always advisable to separate the variability caused by the testing procedure from variabilities caused by other factors such as change in material properties because the variability in testing does not represent a variability in the strength of the concrete used in the actual construction. The following procedure can be used to estimate the magnitude of variation due to testing.

A test consists of all the cylinders made under identical conditions. The cylinders should be made using the same sample of concrete, cured at the same conditions, and tested at the same age. If

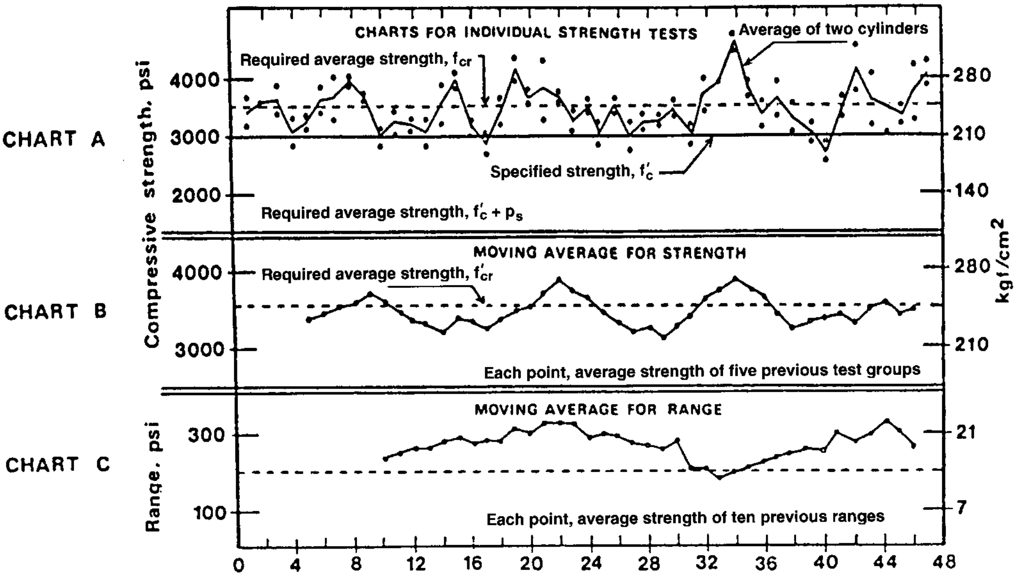


FIGURE 3.9 Quality control charts for concrete production and evaluation. (A) Individual strength tests. (B) Moving average for strength. (C) Moving average for range. (*From ACI Committee 214.*[11])

it is assumed that two or more test cylinders made from the same sample of concrete and tested at the same age should have the same strength, variations in the strengths of these cylinders can be attributed to the testing procedures. However, since differences in casting and curing could also make a difference, only a major part (and not 100%) of the variation can be attributed to testing. Differences between cylinders cast from the same sample are called within-test variations. The within-test standard deviation S_{wt}, can be calculated using the equation

$$S_{wt} = \frac{R}{d_2} \tag{3.14}$$

where R = average range for all tests of a class of concrete
d_2 = factor based on number of cylinders within test

The values of d_2 are 1.128, 1.693, and 2.059 for two, three, and four cylinders, respectively. The range is the difference between the highest and lowest values of strength.

The within-test coefficient of variation V_{wt} can be computed using the equation

$$V_{wt} = \frac{S_{wt}}{\bar{x}} \times 100 \tag{3.15}$$

where $\bar{x}$ is the average strength for the class of concrete.

If V_{wt} is less than 1.5%, the field control testing can be considered excellent. If the value is greater than 4%, the within-test variation should be considered as being poor, and errors in testing may be a major contributing factor to strength variation. If S_{wt} is between 1.5 and 2.0, 2 and 3, or 3 and 4, the testing performance is considered very good, good, or fair, respectively.

3.3.5 Accelerated Strength Tests

In modern-day construction, large volumes of concrete are placed in a single day. In some cases, such as slip-formed construction, it is possible to complete a substantial portion of a structure in a single day. For example, in the case of the CN Communication Tower in Toronto, Canada, the slip-formed construction procedure was used to complete almost 20 ft/day (6 in/day). Therefore one cannot wait 28 days to ascertain the strength. If the strength were found to be unsatisfactory after 28 days, hundreds of feet of structure would have to be taken down. Accelerated strength test procedures were developed for use in such situations. Using these procedures potential 28-day strengths can be estimated in 1 or 2 days. The four procedures, procedure A (warm-water method), procedure B (boiling-water method), procedure C (autogenous curing method), and procedure D (high-temperature and -pressure method), are recognized in ASTM C 684. This section deals with brief descriptions of these methods and their use in quality assurance.

Warm-Water Method (Procedure A)

In this method the cylinders are placed in warm water right after casting. The specimens are cured in their molds in the water maintained at 95 ± 5°F (35 ± 3°C) for a period of 23.5 ± 0.5 h. After this curing period of about 24 h the cylinders are capped and tested to determine their compressive strengths. An extensive study conducted by the U.S. Corps of Engineers established that this is a reliable method for routine quality control of concrete. The primary limitation of this method is that the strength gain is not substantial as compared to 24-h moist cured samples.

Boiling-Water Method (Procedure B)

In this method the cylinders are stored at 70 ± l0°F (21 ± 5°C) for the first 23 ± 0.25 h and then placed in boiling water for a period of 3.5 h ±5 mm. The specimens are allowed to cool for 1 h and tested. The strength increase provided by this type of accelerated curing is much higher than for the warm-water method, and hence the specimen can be transported to the laboratory site without being damaged. This method is the most commonly used one among the four methods.

Autogenous Method (Procedure C)

In this procedure the accelerated curing effect is obtained by using the heat of hydration. The cylinders are placed in an insulated container right after casting to retain the heat generated by hydration. The cylinders are tested after curing for 48 ± 0.25 h and a rest period of 30 mm at room temperature. The strength gain obtained in this method is lower than that obtained by the boiling-water method. This procedure was found to be less accurate than procedures A and B. However, it was used successfully in the CN tower in Toronto, Canada. The project, which was completed in 1974, involved the placement of about 51,000 yd^3 (39,000 m^3) of concrete.

High-Temperature and -Pressure Method (Procedure D)

This procedure is limited to concrete containing aggregates smaller than 1 in (25 mm). Wet sieving can be used for concrete containing larger aggregates. Sealed 3- by 6-in (75- by 150-mm) cylinders are cured at a temperature of 300 ± 5°F (149 ± 3°C) and a pressure of 1500 ± 25 psi (10.3 ± 17 MPa) for a period of 5 h ± 5 min. The curing process starts right after casting. In most cases capping is not required because of the presence of end plates and external pressure. Hence the specimens can be tested within 15 min after curing. For specimens that need capping, testing is done after 30 min. This procedure needs sophisticated equipment and hence is more expensive compared to the three other procedures.

A summary of all four procedures is presented in Table 3.14. This table shows the curing medium, the temperature and duration of curing, and the age at testing for all four procedures.

3.3.6 Quality Control Using Accelerated Strength Tests

The most important use of accelerated test data is quality control. These tests permit rapid adjustment of batching and mixing. In the current practice, the accelerated strength results are used to estimate 28-day strengths because of the traditional use of 28-day strength for design purposes. A correlation between the chosen accelerated strength and the 28-day strength should be established before starting the project. The mix proportions and materials used for developing the correlation equation should be the same as the materials and mix proportions to be used for the project.

ACI Committee 214[12] recommends a minimum of 30 data sets for establishing the correlation between the 28-day strength and the accelerated strength. The 28-day strength range should include

TABLE 3.14 Accelerated Curing Procedures

Procedure	Molds	Accelerated curing medium	Curing begins	Duration of curing	Age at testing
A Warm water	Reusable or single use	Warm water, 950F (35°C)	Immediately after casting	23½ h ± 30 in	24 h ± 15 min
B Boiling water	Reusable or single use	Boiling water	23 h ± 15 mm after casting	3½ h ± 5 min	28½ h ± 15 min
C Autogenous	Single use	Heat of hydration	Immediately after casting	48 h ± 15 min	49 h ± 15 min
D High temperature and pressure	Reusable	External heat and pressure, 300°F (149°C)	Immediately after casting	5 h ± 5 mm	s5¼ h ± S mm

Source: From ASTM C654, 1993.[2]

the specified strength and should not fall below 75% of the specified strength. The 28-day strength is normally expressed as a linear function of accelerated strength using the equation

$$y = ax + b \tag{3.16}$$

where y = 28-day strength
x = accelerated strength
a, b = constants

The constants a and b are obtained using statistical correlation. The correlation coefficient should be at least 0.8. A typical correlation curve is shown in Fig. 3.10. The 95% confidence limits in Fig. 3.10 show the variations of 28-day strength that can be expected 95% of the time. For example, if the accelerated strength is 3000 psi (21 MPa), the expected 28-day strength is about 5700 psi (40 MPa). Of the 28-day strengths 95% can be expected to be in the range of 4800 to 6500 psi (33 to 45 MPa). An interpretation of the accelerated strength test results and their use for quality assurance follows.

Interpretation of Test Results

Accelerated strength test results can be interpreted using the same procedures as those used for 28-day strength results (Sec. 3.3.4). The required average strength f'_{cr} can still be computed using the equation

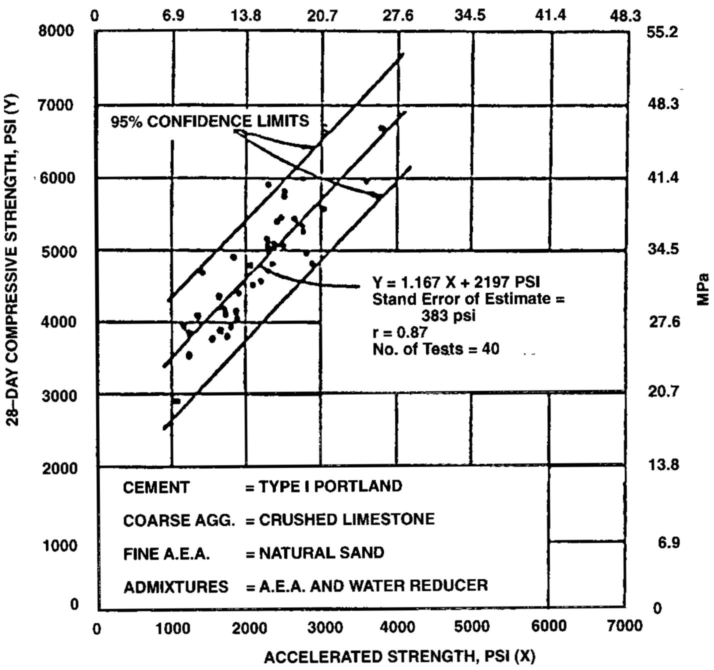


FIGURE 3.10 Relationship between accelerated and 28-day compressive strengths of concrete—Data obtained using boiling-water method (ASTM procedure B). (*From ACI Committee 214.*[12])

$$f'_{cr} = f'_c + t\sigma \tag{3.17}$$

where f'_c = specified design strength
t = constant depending on proportion of tests that may fall below f'_{cc} (Table 3.15).
σ = a standard deviation of data set used for prediction of f'_c

Table 3.15 presents the t values for a number of low test values ranging from 1 in 2 to 1 in 1000. Permissible low tests should be chosen primarily based on the type of structure, as explained earlier.

The required average strength can be established based on either specified accelerated strength or specified 28-day strength and the correlation equation between accelerated and 28-day strengths. The following examples illustrate the computation procedure.

Example 3.3 The specifications require an accelerated strength of 2000 psi (13.8 MPa). Compute the required average (accelerated) strength f'_{cr} if the acceptable number of low tests is 1 in 100. The standard deviation from past records for accelerated strength tests is 500 psi (3.4 MPa).

Solution From Table 3.15, the value oft for 1 in 100 low tests is 2.33

$$\begin{aligned} f'_{cr} &= f'_c + t\sigma \\ &= 2000 + 2.33(500) \\ &= 3165 \text{ psi (21.8 MPa)} \end{aligned}$$

Note that f'_{cr}, f'_c, and σ correspond to accelerated strengths.

Example 3.4 The specifications require a 28-day compressive strength of 4500 psi (31 MPa). Compute the required accelerated average strength using the following information. The relationship between 28-day strength y and accelerated strength x is

$$y = 1.167x + 2197$$

The acceptable number of low tests is 1 in 10, and the standard deviation for accelerated strengths is 410 psi (2.83 MPa)

Solution The specified 28-day strength is 4500 psi (31 MPa). Using the correlation equation, the corresponding accelerated strength is

TABLE 3.15 Values oft for the Equation $f'_{cr} = f'_c + t$

Number	Likelihood of low test results, %	t
1 in 1000	0.1	3.09
1 in 500	0.2	2.88
1 in 100	1.0	2.33
1 in 50	2.0	2.06
1 in 25	4.0	1.75
1 in 20	5.0	1.65
1 in 10	10.0	1.28
1 in 5	20.0	0.84
1 in 2	50.0	0.00

Source: From ACI Committee 214.[12]

$$x = \frac{y - 2197}{1.167}$$

$$= \frac{4500 - 2197}{1.167} = 1973 \text{ psi (13.6 Mpa)}$$

For 1 low test in 10, $t = 1.28$,

$$(f'_{cr})_{\text{accelerated}} = 1973 + 1.28(410) = 2498 \text{ psi}$$

Hence the required average accelerated strength is 2500 psi (17 MPa).

If the number of pairs of data used for the regression line relating accelerated and 28-day strengths is less than 30, special statistical procedures can be used for the prediction of f'_{cr}. The procedure can be found in the report of ACI Committee 214.[12] The number of pairs should be at least 10.

3.3.7 Nondestructive Tests

Nondestructive tests are valuable tools for evaluating the properties of in situ concrete. These methods can be used to estimate the strength, durability, or elastic properties of concrete. In addition they can also be used to estimate the location and condition of reinforcement, and for locating cracks, large voids, and moisture content. The test methods can be classified as:

Surface hardness methods

Penetration resistance techniques

Pull-out tests

Ultrasonic pulse velocity method

Maturity concepts

Electromagnetic methods

Acoustical methods

These methods are described briefly in this section.

Surface Hardness Methods

In surface hardness methods the hardness of the surface measured, using the size of the indentation or the amount of rebound, is taken as an indicator of the strength of concrete. The most popular method in this category is the rebound hammer test. This test is also known as Schmidt rebound hammer, impact hammer, or sclerometer test. The rebound hammer, shown in Fig. 3.11, consists of a spring-loaded mass and a plunger. When the plunger is pressed against the concrete, it retracts against the force imparted by a spring. When the spring is retracted to a certain position, it releases automatically. Upon release the mass rebounds, taking a rider with it along a guide scale. The distance traveled by the mass, expressed as a percentage of the initial extension of the spring, is called the rebound number. ASTM C805 covers the procedure for conducting the rebound hammer test.

The rebound number, which is a measure of the hardness of the concrete surface, can be empirically related to the compressive strength of concrete. However, in certain circumstances the surface hardness may not represent the strength of the concrete inside the structure. For example, the presence of a large aggregate immediately underneath the plunger would result in a large rebound number. A large void underneath the plunger, on the other hand, would provide an unusually low rebound number. Other factors that influence the rebound number include the type of aggregate, smoothness of the surface, moisture condition, size and age of the specimen, degree of carbonation,

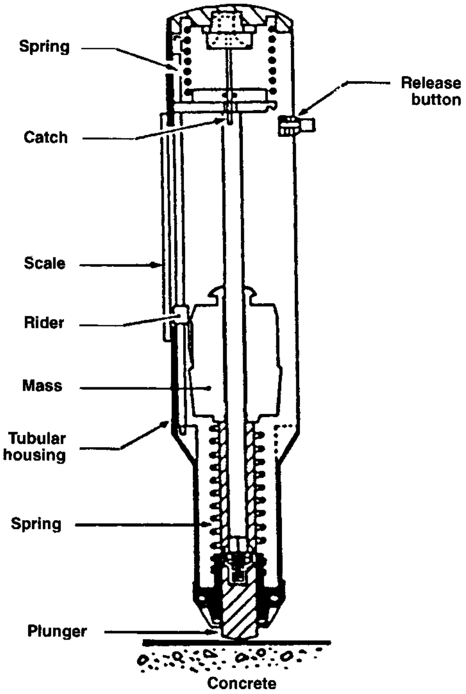


FIGURE 3.11 Major components of rebound hammer.

and position of the hammer (vertical versus inclined or horizontal). The hammer should be used only against a smooth surface. Troweled surfaces should be rubbed smooth using a carborundum stone. If the concrete being tested is not part of a large mass, it has to be supported so that the specimen does not move during the impact. Concrete in the dry state tends to record a higher rebound number. Because of gravity, the rebound number of floor concrete would be smaller than that of the soffit concrete, even though both concretes are similar. The rebound number of inclined and vertical surfaces would fall somewhere in between.

The rebound hammer is best utilized for checking the uniformity of concrete in a large structure or for comparing the quality of concrete in similar structural components such as precast beams. It can also be used for estimating the strength of concrete that is being cured for the purpose of removing formwork. If the rebound numbers have to be used for estimating strength, correlation should be established between the rebound number and the compressive strength for each type of concrete used on a site. If mix proportions or constituent materials are changed, then a new set of tests should be conducted and a new correlation equation obtained.

Typically there is a large variation in rebound numbers. At least 10 to 12 readings should be taken in each location. ASTM C805 provides guidelines for averaging the rebound numbers. In certain cases some individual readings might have to be omitted from the average. If proper calibration is used, the accuracy of prediction of concrete strength is about ±20% for laboratory specimens and ±25% for in situ concrete.

Penetration Resistance Techniques

In this method the penetration resistance of concrete is used as the indicator of its strength. The most commonly used test is the Windsor probe test. In this test a hardened alloy probe is fired (or driven) by a driver using a standard charge of powder. The exposed length of the probe is taken as a measure of penetration resistance. It is assumed that the compressive strength of concrete is proportional to its penetration resistance. Here again the hardness of the aggregates plays an important role. A correlation has to be developed for a particular concrete if this method is to be used for predicting strengths.

The probes are driven in sets of three in close vicinity, and the average value is used for estimating the strength. The test procedure is covered in ASTM C803. This test is more expensive than the rebound hammer test, but much less expensive than core tests. This test, which is considered to be more accurate than the rebound hammer test because the measurement is not made just on the surface, is also an excellent tool for determining the uniformity of concrete and the relative rate of strength gain at early age for the purpose of removing formwork.

Pull-Out Tests

In a standard pull-out test the concrete strength is estimated using the force required to pull out a specially shaped steel insert whose enlarged end has been cast into the fresh concrete. The specifications of the test are covered in ASTM C900. Because of the shape of the insert, a lump of concrete, in the shape of a frustrum of a cone, is pulled out along with the insert. In most cases the fracture occurs at about a 45° angle. Even though the failure occurs due to tension and shear, the strength computed using an idealized area of the frustrum was found to be approximately equal to the shear strength of concrete. An approximate linear correlation seems to exist between pull-out strength and compressive strength. The ratio of pull-out strength to compressive strength decreases slightly for concrete with higher compressive strengths.

The pull-out test is more accurate than the penetration or the rebound hammer test because it is based on the actual failure load. However, this test, which is more involved, has to be planned in advance, and the damaged area has to be repaired. In some cases, such as estimation of strength for form removal, pulling out the assembly may not be necessary. The test can be stopped when a predetermined force is reached, assuring sufficient strength for removing the forms.

Other forms of tests similar to the standard pull-out test are still being developed. Those test methods include the pull-out of inserts placed in drilled holes, pulling out a wedge anchor using torque, and break-off tests. In the break-off test the flexural strength of concrete is determined by applying a transverse force on the cylinder created by inserting a tube in the fresh concrete. Inserts placed in drilled holes can be used for existing structures. But their success is still to be established.

Ultrasonic Pulse Velocity Method

In this method the longitudinal wave velocity in concrete is used to estimate the compressive strength and check the uniformity of concrete. An exciter is used to initiate a pulse which is picked up at another designated location. The distance between the two locations divided by the time required for the travel is the pulse velocity. Correlation relationships have been developed between pulse velocity, compressive strength, and modulus of elasticity. The relationships are affected by a number of variables, such as the moisture condition of the specimen, type and volume fraction of the aggregate, water–cement ratio, and age of specimen. In general, pulse velocity alone cannot be used to estimate the strength, unless a correlation exists for the particular type of concrete being tested. But the method is an excellent tool for quality control measures. The test method is covered in ASTM C597. The pulse velocity method can be used for both laboratory and field tests. In field tests, the presence of reinforcement and the vibration of elements being tested (such as piers under a roadway in use) might pose problems. A combination of rebound hammer and pulse velocity methods is sometimes used to evaluate both the surface characteristics and the quality of concrete located well inside the surface.

Maturity Concepts

It is well established that the strength development of concrete depends on duration, curing temperature, and pressure. If moist curing is done at atmospheric pressure, then the combination of duration and curing temperature could be used to estimate the maturity and hence the compressive strength. A number of relationships relating strength and maturity exist in the published literature. Maturity meters are also available for use in the field and the laboratory. This concept can be used to estimate the early strength of structural members for form removal. The correlation between maturity and strength should be established before starting the actual project.

Electromagnetic and Acoustical Methods

Various forms of magnetic, electrical, and acoustical techniques have been tried to determine the properties of concrete such as dynamic modulus, presence of cracks, honeycombing, and measurement of cover to reinforcement. These methods have not attained common acceptance so far.

3.A.3.8 Core Tests

If there is a reason to believe that the concrete in place may not have the specified strength, nondestructive tests discussed in the previous section can be used to determine the uniformity. If the location in question behaves very similar to other locations, the quality of the concrete could be satisfactory. However, if the tests indicate variability, cores might have to be taken to determine the strength. The procedure for evaluation using cores is covered in ASTM C42.

Typically, cores are drilled using diamond drills. A number of factors should be considered in evaluating core strengths. The following are some of the important points:

- Typically core strengths are lower than standard cylinder strengths. The differences could be more significant for high-strength concrete.
- The strength of the core could depend on its position in the structure. Cores taken near the top of a structural element are typically weaker than cores taken from the bottom.
- Cores taken from very thick sections could contain microcracks due to excessive heat of hydration and hence register lower strengths.
- The presence of large aggregates in small cores could result in erroneous strengths.

3.4 MECHANICAL PROPERTIES

Strength, stiffness, and dimensional stability constitute the core of the mechanical properties. Strength can be measured under various modes of loading such as compression, tension, flexure, shear, and torsion. This section deals with these basic mechanical properties of concrete. Properties such as the durability of concrete exposed to various chemicals or to freezing and thawing, and permeability are also very important for some structures. These properties are not covered in this book for lack of space. The reader is referred to the literature.

3.4.1 Compressive Strength

Compressive strength is the most commonly used design parameter for concrete. In most cases the concrete is specified using its compressive strength measured at 28 days. In some instances 56-day strength or minimum early strength is specified. Up to around 1960, the compressive strength of concrete was limited to about 6000 psi (42 MPa). The advent of new admixtures as well as mixing, placing, and compacting techniques led to the development of higher strengths. Concrete with a specified compressive strength of 12,000 psi (84 MPa) was used in Water Tower Place in Chicago, which was topped off in 1972. The development of high-range water-reducing admixtures (superplasticizers) resulted in routine use of high-strength concrete. Concrete used in a Seattle building in the late 1980s had an average strength in excess of 20,000 psi (135 MPa). This section deals with the various factors that affect the compressive strength of concrete. An understanding of the influence of the various factors is needed for effectively proportioning, making, and casting concrete in the field.

The major factors that influence compressive strength are water–cement ratio; aggregate-cement ratio; maximum size of aggregate; grading, surface texture, shape, strength, and stiffness of aggregate particles; degree of compaction; curing conditions; and testing parameters. The admixtures used can also influence the strength by improving workability and through better compaction.

Water-Cement Ratio

The relation between compressive strength and water–cement ratio was established in 1918 by Duff Abrams. For a fully compacted concrete, he found that the strength f'_c can be expressed as

$$f'_c = \frac{k_1}{k_2 w/c} \tag{3.18}$$

where w/c = water–cement ratio

k_1. k_2 = empirical constants

If the amount of air voids does not exceed 1% by volume, the concrete can be considered as fully compacted concrete. Typical variations of compressive strength and the influence of compaction are shown in Fig. 3.12. From this figure it can be seen that compaction plays an important role. The use of admixtures that improved workability led to better compaction, resulting in higher and higher strengths in the 1980s. Typical variations of compressive strength for water–cement ratios ranging

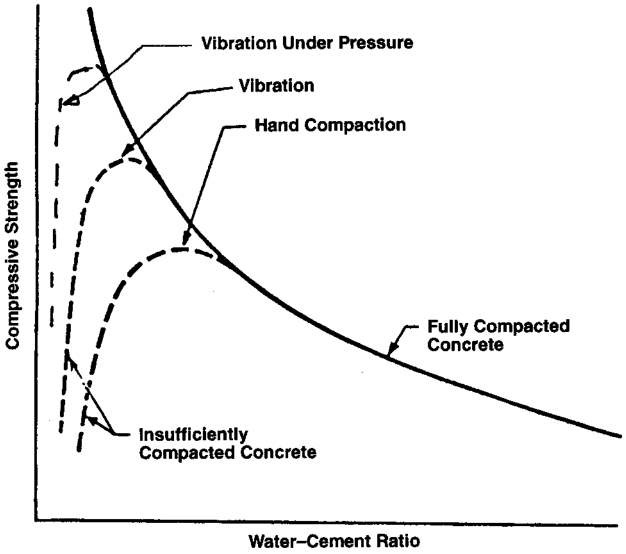


FIGURE 3.12 Relationship between strength and water–cement ratio of concrete.

from 0.35 to 1.2 are shown in Fig. 3.13. In field construction the commonly used water–cement ratios vary from 0.25 to 0.65. As mentioned in Sec. 3.2, the water–cement ratio should be restricted to 0.4 for obtaining durable concrete. If the water–cement ratio is less than 0.4, in most cases some form of admixture is needed for obtaining a workable concrete.

The amount of voids present in concrete controls its strength. In concrete with a high water- cement ratio, the excess water results in more voids and hence lower strength. Improperly compacted concrete also has higher voids, resulting in low strength. Hence a balanced approach should be used in mix proportioning. A water–cement ratio of about 0.4 is needed for complete hydration of the cement. However, complete hydration of cement does not produce the highest strength. The presence of unhydrated cement as inert particles was found to provide better strength. In most cases the lower limit for the water–cement ratio is 0.28. However, water–cement ratios lower than 0.28 have been used with superplasticizers and other admixtures for producing very high-strength concrete.

Aggregate-Cement Ratio

The aggregate-cement ratio affects the strength of concrete if the strength 15 about 5000 psi (35 MPa) or more. The influence of the aggregate-cement ratio is not as significant as that of the water–cement ratio, but it has been found that for a constant water–cement ratio, leaner mixes provide higher strengths, as shown in Fig. 3.14. The increase in strength could be due to absorption of water by the aggregate and hence a lower effective water–cement ratio. In addition leaner mixes have lower amounts of total water and paste content and hence lower amounts of voids.

A more recent study indicates that the strength increases with an increase in the cement content if the volume of aggregates is less than 40%. But the trend reverses in the aggregate volume ratios of 40 to 80% (Fig. 3.15).

Cement Type and Age

The degree of cement hydration determines the porosity of the hydrated cement paste and hence the compressive strength. Under standard curing conditions type III cement hydrates faster than type I

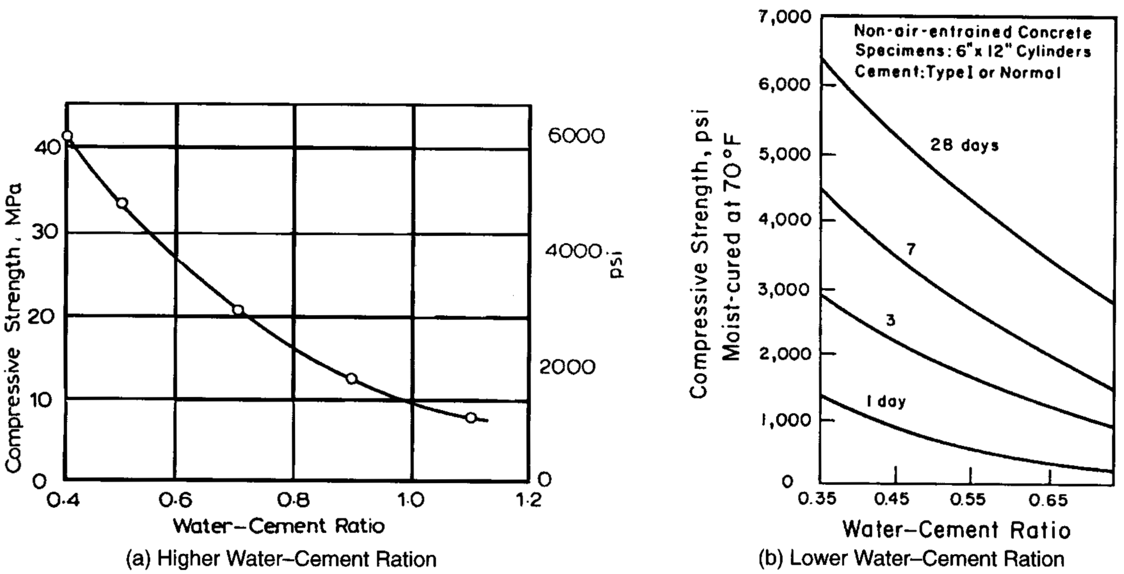


FIGURE 3.13 Influence of water–cement ratio and moist curing on concrete strengths. (a) Higher water–cement ratio. (b) Lower water–cement ratio. (*From PCA.*[8])

cement. Hence at early ages type III cement provides higher strengths. The variations in strength, for type I and III cements at 1, 3, 7, and 28 days are shown in Fig. 3.16. The bands shown in this figure cover the majority of the data obtained in the laboratories. Relative strength gains for three water–cement ratios are shown in Fig. 3.17. From this figure it can be seen that early strength gain is higher for lower water–cement ratios. Empirical relationships are available in the literature for predicting 28-day strengths based on the results of 1-, 3-, or 7-day strengths and vice versa.

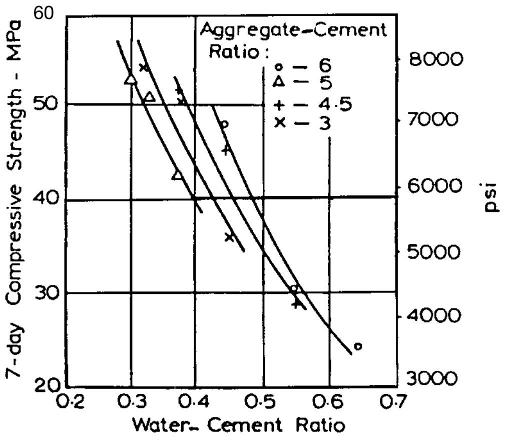


FIGURE 3.14 Influence of aggregate-cement ratio on strength of concrete. (*From Singh.*[13])

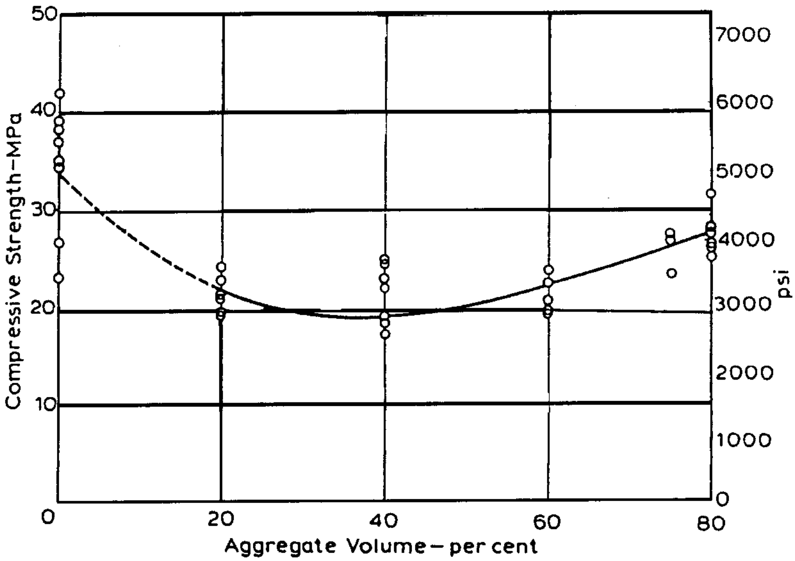


FIGURE 3.15 Influence of aggregate content on strength. (*From Stock et al.*[14])

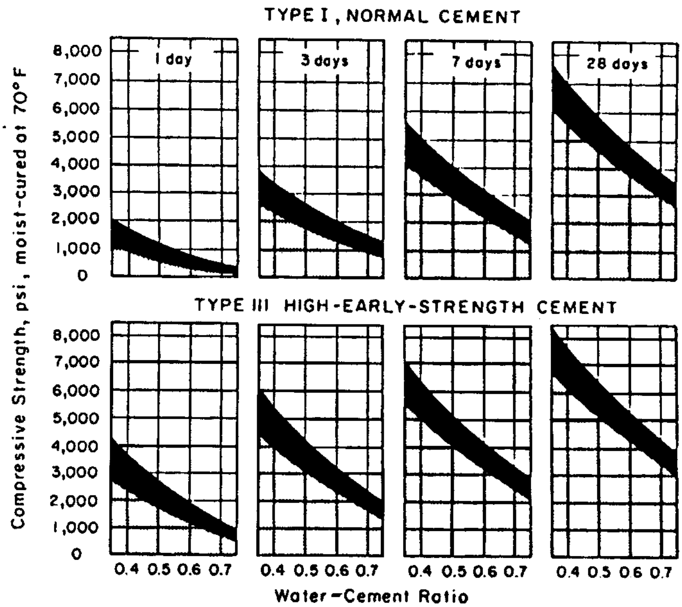


FIGURE 3.16 Influence of water–cement ratio, duration of moist curing. and cement type on strength. (*From PCA.*[8])

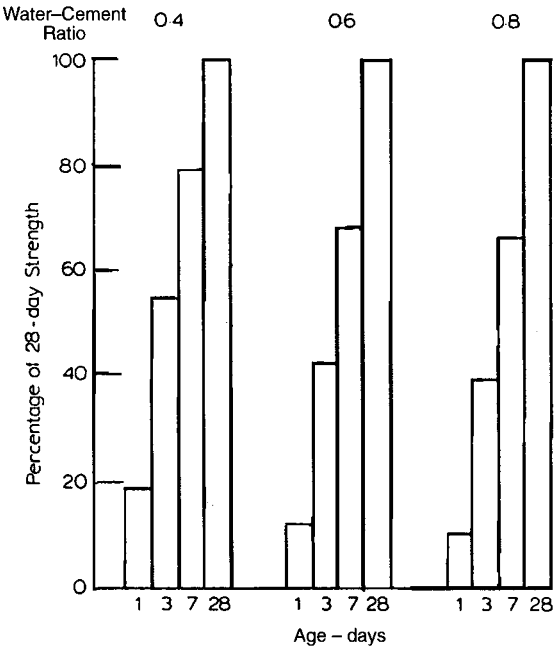


FIGURE 3.17 Relative strength gain of concrete with time for different water–cement ratios.

At normal temperatures ASTM type II, IV, and V portland cements hydrate at a slower rate, and hence concrete containing these cements will have lower early strengths. The results presented in Table 3.16 show typical relative strengths at 1, 7, and 28 days for various cements. After 90 days the variation in strength between cement types is negligible.

Coarse Aggregate

Typically, aggregates are stronger than the matrix and hence do not control the fracture strength. However, factors such as aggregate size, shape, surface texture, and mineralogy can influence the

TABLE 3.16 Approximate Relative Strength of Concrete as Affected by Type of Cement

Type of portland cement		Compressive strength, % of strength of type I* 1 day	7 days	28 days
I	Normal or general-purpose	100	100	100
II	Moderate heat of hydration and moderate sulfate resisting	75	85	90
III	High early strength	190	120	110
IV	Low heat of hydration	55	65	75
V	Sulfate resisting	65	75	85

*Compressive strength is 100% for all cements at 90 days.

Source: From PCA.[8]

workability, degree of compaction, and formation of gel around the aggregate. Consequently it can affect the compressive strength.

The maximum size of aggregate has considerable influence, especially at low water–cement ratios, as shown in Fig. 3.18. Concrete with large-size aggregates requires less water, and hence the same water–cement ratio provides better compaction, resulting in higher strength. On the other hand, larger sizes provide weaker transition zones around the aggregate, resulting in lower strength. These opposing influences are water–cement ratio dependent and provide more pronounced effects at lower water–cement ratios. In most cases the strength can be expected to go down with an increase in the maximum size of the aggregate.

If water–cement ratio and maximum size of aggregate are kept constant, aggregate grading influences the consistency of the concrete and hence the strength. An increase in fines typically increases the water demand, and if the amount of water is not increased, consistency decreases. The decrease in strength was found to be as high as 12%.

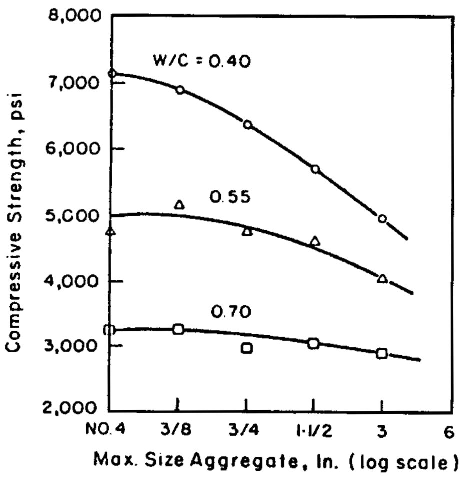


FIGURE 3.18 Influence of aggregate size and water–cement ratio on strength. (*From Cordon and Gillespie.*[15])

Aggregates with rough texture were found to result in early high strengths as compared to aggregates with smooth textures. The rough surface provides a better bond between matrix and aggregate, especially at early stages when the hydration is not complete. At later stages the influence diminishes. Aggregates with smooth surfaces are easier to work with and hence provide a better final product.

The mineralogical composition of aggregates was also found to influence the concrete strength. Calcareous aggregates tend to provide better strength than siliceous aggregates. Fig. 3.19 shows the compressive strengths obtained using various types of aggregates. It can be seen that the strength variation could be as high as 50%.

Air Content

Air is entrained in concrete to improve its durability. The air bubbles tend to improve the workability of fresh concrete and hence improve the compaction. But the presence of air bubbles in the hardened concrete increases its porosity and reduces the density of the composite. Hence air entrainment leads to a decrease in compressive strength.

The decrease in strength due to air entrainment was found to depend on both the water–cement ratio and the cement content. As the water–cement ratio decreases, the strength loss increases. Hence the strength loss is considerable for high-strength concrete. When the cement content is reduced, the influence of the air content also decreases. In fact the air content improves the strength slightly if the cement content is very low. In the normal strength range, about a 2% loss in compressive strength can be expected for each 1% increase in air content.

The air content was found to improve the workability of lightweight concrete. This is particularly true for mixes of low paste content.

Curing Conditions

Concrete should be moist cured for at least 28 days to obtain best results. Premature drying can reduce the strength by more than 50%. In terms of curing, the major factors are time, humidity, temperature, and pressure.

A longer curing time under moist conditions always provides better results. A 100% relative humidity is the best condition. This can be achieved by pooling water, placing wet burlaps, or making

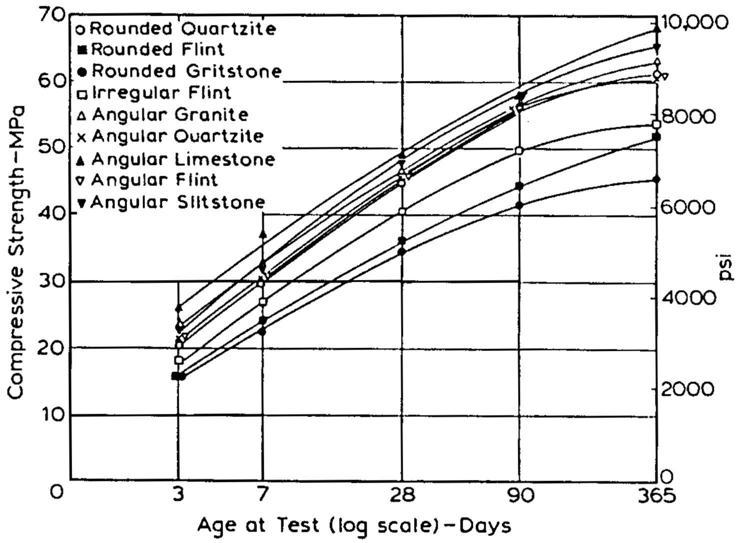


FIGURE 3.19 Influence of aggregate type on strength. (*From a report by the Building Research Station, London, 1969.*)

other arrangements. Figure 3.20 shows the influence of moist curing. It can be seen that 3 days of moist curing provides 50% improvement over air-dried samples. It is preferable to moist cure for 28 days. In any circumstance, the moist curing should be done for at least 7 days.

Typically higher temperatures provide faster curing. At about 12°F (–11°C) the cement stops hydrating. Hence at this temperature there may not be any increase in strength, even after long periods of time. Early age strength increase can be accelerated by using warm water. Temperatures higher than 100°F (380C) are not normally used because the increase in acceleration beyond this tempera-

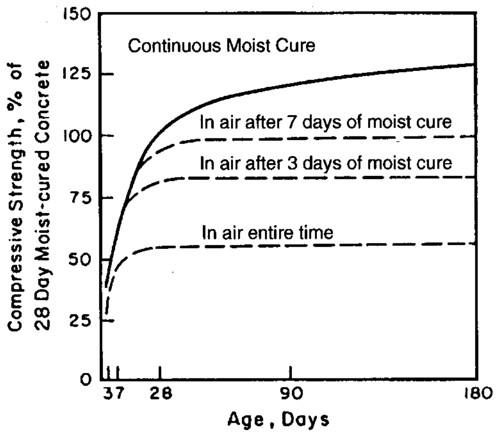


FIGURE 3.20 Influence of curing conditions on strength. (*From PCA.*[8])

ture is not high enough for economic reasons and the initial high temperature could result in lower long-term strength. However, precast elements are sometimes steam cured in order to reduce turn-around time and speed up reuse of forms.

Higher pressure typically accelerates the curing. But the increase is not significant for commercial applications. Hence pressure curing is used only for steam curing.

Test Conditions

In North America 6-by 12-in (150- by 300-mm) cylinders are the standard test specimens, even though 4- by 8-in (100- by 200-mm) cylinders are sometimes used for high-strength concrete. Other types of specimens include cubes and prisms. In the case of cylinders and prisms, the minimum length-to-diameter ratio (or longer dimension versus lateral dimension) should be at least 2 to avoid end effects. If specimens with lower length-to-diameter ratios are used, correction factors should be applied for determining the compressive strength. ASTM specifications provide guidelines, as explained in Sec. 3.2. Cylinders tend to record lower strengths than cubes. The ratio of cylinder to cube strength is about 0.85. Larger samples tend to record lower strengths as compared to smaller samples. Other test conditions that influence strength include end conditions, rate of loading, moisture condition of the specimen, and condition of the plattens of the machine.

Cylinders should be capped properly, making sure that the ends are plane, parallel to each other, and perpendicular to the longitudinal axis. The testing machine should have capability to allow slight rotation of the ends. The machine should also be calibrated periodically to avoid errors in the measurement of loads.

3.4.2 Tensile Strength

The tensile strength can be measured using uniaxial tension specimens or cylinders. Uniaxial tension specimens are very difficult to test because special gripping devices are needed. The most common practice is to test a 6- by 12-in (150- by 300-mm) cylinder along the longitudinal axis. This test is called the splitting tension test, and the strength value obtained is the splitting tensile strength.

There is a relationship between compressive and tensile strength. But the relationship is not well established. Tensile strength is about 10% of compressive strength for normal-strength concrete. As the compressive strength increases, the ratio decreases. Table 3.17 presents typical tensile strength values for compressive strengths varying from 1000 to 9000 psi (7 to 60 MPa). A number of factors, including type and shape of coarse aggregate, properties of fine aggregate, grading of aggregate, air

TABLE 3.17 Relation between Compressive, Flexural, and Tensile Strengths of Concrete

Strength of concrete, psi (MPa)			Ratio, %		
Compressive strength	Modulus of rupture	Splitting tensile strength	Modulus of rupture to compressive strength	Splitting tensile strength to compressive strength	Splitting tensile strength to modulus of rupture
1000 (6.9)	230 (1.6)	110 (0.8)	23.0	11.0	48
2000 (13.8)	375 (2.6)	200 (1.4)	18.8	10.0	53
3000 (20.7)	485 (3.3)	275 (1.9)	16.2	9.2	57
4000 (27.6)	580 (4.0)	340 (2.3)	14.5	8.5	59
5000 (34.5)	675 (4.7)	400 (2.8)	13.5	8.0	59
6000 (41.3)	765 (5.3)	460 (3.2)	12.8	7.7	60
7000 (48.2)	855 (5.9)	520 (3.6)	12.2	7.4	61
8000 (55.1)	930 (6.4)	580 (4.0)	11.6	7.2	62
9000 (62.0)	1010 (7.0)	630 (4.3)	11.2	7.0	63

Source: From Price.[16]

entrainment, and age at testing, affect the ratio between tensile and compressive strengths. A number of empirical relations are available for estimating the tensile strength. Most of these equations are of the form

$$f_t = k(f'_c)^n \tag{3.19}$$

where f'_c = compressive strength
f_t = tensile strength
k, n = empirical constants

Concretes containing lightweight aggregate typically have lower tensile strengths.

3.4.3 Modulus of Rupture

The flexural strength of concrete is called modulus of rupture. The strength values are determined using 4- by 4- by 14- or 6- by 6-by 20-in (100- by 100- by 350- or 150- by 150- by 550-mm) prisms subjected to four-point loading. When analyzing reinforced concrete beams and slabs for flexural loading, the modulus of rupture is used to compute cracking load and deflection. Since the beams are in the flexure mode, the modulus of rupture is more representative than splitting or direct tensile strengths. Typical values of flexural strength are presented in Table 3.17.

For design purposes, the modulus of rupture f_r can be estimated using the following equation:

$$f_r = 7.5\sqrt{f'_c} \tag{3.20}$$

In most cases this equation provides a conservative estimate. For high-strength concrete, other forms of equations have been proposed. Nevertheless, Eq. (3.20) provides a good estimate for design purposes. For lightweight concrete a constant smaller than 7.5 should be used.

Most factors that influence the ratio of tensile to compressive strength also influence the modulus of rupture. The magnitude of influence is slightly lower for flexural strength.

3.4.4 Shear Strength

Concrete is seldom subjected to pure shear. But when structural members are loaded under various modes, the concrete in those members could be subjected to a shear force. It is extremely difficult to measure direct shear strength. Torsion or deep beam specimens can be used to measure shear strength indirectly.

Equations for the prediction of shear strength are not well established. Most researchers agree that the shear strength is proportional to the square root of compressive strength. But the constant of proportionality is not established. For beams subjected to bending and shear, the ACI code allows a shear stress of $4\sqrt{f'_c}$ for uncracked sections. However, this value is useful only for beams subjected to shear. A number of researchers have tested specimens under torsion to establish the shear strength. But the results are still inconclusive in terms of having a single equation for predicting shear strength.

3.4.5 Modulus of Elasticity

The modulus of elasticity of a material is the slope of the initial linear portion of the stress-strain curve. Since the stress-strain curve of concrete is nonlinear, there are three types of moduli, namely, the tangent, the secant, and the chord modulus. The secant modulus is the most commonly used parameter for design purposes. It is defined as the slope of the line joining the origin and the point on the stress-strain curve corresponding to 40% of compressive strength. For high-strength concrete the initial portion of the curve is almost linear, and hence the tangent and the secant moduli are same. Since measurement of the modulus is a more involved process, its value is normally estimated using the compressive strength and the unit weight of concrete.

The ACI code recommends the following equations for normal-strength concrete[4]:

$$E_c = 33\, W_c^{1.5}\sqrt{f_c'} \tag{3.21}$$

where E_c = secant modulus
E_c = unit weight of concrete
f_c' = compressive strength of concrete

For normal-weight concrete which has a unit weight of about 145 lb/ft^3 (2321 kg/m^3) E_c can be computed using the following equation:

$$E_c = 57{,}000\sqrt{f_c'} \tag{3.22}$$

For high-strength concrete the aforementioned equations were found to overestimate the value of the modulus. The equation recommended for high-strength concrete is

$$E_c = (40{,}000\sqrt{f_c'} + 10^6)\left(\frac{W_c}{145}\right)^{1.5} \tag{3.23}$$

A number of other equations are also available in the literature. The primary factors that influence the modulus of elasticity are aggregate type, amount of paste, mineral admixtures such as fly ash and silica fume, and moisture conditions of the specimen. Tests should be conducted carefully using ASTM procedures. The modulus value is very sensitive to test conditions such as rate of loading.

3.4.6 Shrinkage

Shrinkage is a phenomenon in which concrete shrinks under no load due to the movement and evaporation of water. There are two types of shrinkage known, plastic and drying shrinkage. Plastic shrinkage occurs during the initial and final setting times, extending to several hours after the initial placement of concrete. Major factors that affect plastic shrinkage are area of exposed surface, surface and concrete temperature, and surface air velocity.

Drying shrinkage occurs in hardened concrete, and it can continue for up to 2 years. It occurs due to the evaporation of water and the movement of water within, resulting in a more compact matrix.

If concrete is immersed in water, a small amount of expansion known as swelling can be observed. However, the expansion cannot completely reverse the shrinkage. Shrinkage is a time-dependent process. In the first few months it is much larger than at later ages. A number of factors influence the shrinkage. These factors are briefly discussed in the following. More details can be found in the literature dealing with concrete.

Aggregate Type and Volume Fraction

The aggregates restrain the shrinkage of cement paste because they do not shrink. Hence the concrete with lower cement content has less shrinkage. In addition the elastic properties and other characteristics of the aggregate also influence the shrinkage because the amount of restraint proivided by the aggregate depends on its elastic modulus and how much force it can transmit along the interface. Typical shrinkage values for different aggregate-cement and water–cement ratios are shown in

Table 3.18. From this table it can be seen that shrinkage can be reduced by as much as four times by increasing the aggregate content.

In terms of type of aggregates, quartz provides the least amount of shrinkage. Aggregates that produce more shrinkage in progressive order are limestone, granite, basalt, gravel, and sandstone. Lightweight aggregate concretes typically shrink more than normal-weight concrete.

Water-Cement Ratio

As the water–cement ratio increases, shrinkage increases, as shown in Table 3.18. This should be expected because more water leads to more evaporation and movement. Water-cement ratios higher than 0.7 could lead to excessive shrinkage.

Exposure Conditions

The relative humidity, temperature, and air movement to which the element is exposed affect both the rate of shrinkage and the total (ultimate) shrinkage. Higher relative humidity, lower temperature, and low air velocity decrease shrinkage.

Size of Member

As the size of the member increases, the rate of evaporation decreases and hence the rate of shrinkage is lower. As the concrete matures, the amount of water lost to evaporation also decreases, because water cannot move freely in matured concrete. This results in a decrease in ultimate shrinkage.

Type of Cement

Cements that hydrate faster tend to produce more shrinkage. Special cements, called shrinkage-compensating cement, are available for reducing shrinkage. ASTM type K cement is called expansive cement, as it provides increases in volume rather than a decrease, or shrinkage.

Admixtures

As mentioned in Sec. 3. 1, a number of mineral and chemical admixtures are used in concrete to obtain certain properties in the fresh and hardened states. Accelerating admixtures tend to increase the rate of shrinkage whereas retarding admixtures will decrease the rate of shrinkage. Since concrete with water-reducing admixtures tends to have lower water–cement ratios, both the rate and the ultimate shrinkage for this concrete decreases. The effect of mineral admixtures depends on the type and volume fraction. Air-entraining agents were found to have little effect on shrinkage.

Carbonation

Carbonation, which occurs due to a reaction between the carbon dioxide present in the atmosphere and the cement paste, tends to produce shrinkage which is called carbonation shrinkage. This phe-

TABLE 3.18 Typical Values ($\times 10^{-6}$) of Shrinkage after 6 Months for Mortar and Concrete Specimens*

	Water-cement ratio			
Aggregate-cement ratio	0.4	0.5	0.6	0.7
3	800	1200	—	—
4	550	850	1050	—
5	400	600	750	850
6	300	400	550	650
7	200	300	400	500

*Based on 5-by 5-in (125- by 125-mm) prisms, stored at a relative humidity of 50% soda temperature of 70°F (21°C).

Source: From Lea.[17]

nomenon occurs only near the exposed surface. Carbonation tends to occur over a longer period of time than drying shrinkage. At humidities less than 50%, carbonation decreases drastically whereas drying shrinkage accelerates.

External Restraints

External restraints reduce shrinkage. The most common restraints are the reinforcing bars. Reinforcement tends to resist movement, and hence the total magnitude of shrinkage decreases. Because of the restriction in movement, a stress state occurs in reinforced concrete elements due to shrinkage. Typically reinforcement is subjected to compression and the surrounding concrete is subjected to tension. If the concrete is weak and its cross section is not sufficient to withstand the tensile forces, it might crack.

Mixing, Placing, Consolidation, and Curing

The way the concrete is mixed, placed in position, compacted, finished, and cured affects the quality of the concrete and hence the shrinkage. If these operations are not done properly, the resulting concrete could be porous, sustaining higher shrinkage strains. Curing is very important because the presence of moisture on the exposed surface reduces the evaporation loss considerably, resulting in reduced shrinkage.

Reduced shrinkage strain is better for the integrity of the concrete elements. Shrinkage increases the long-term deflections of beams and slabs, as well as the crack widths. In prestressed concrete shrinkage produces a loss in prestress. In composite construction and indeterminate structures, shrinkage produces a redistribution of stresses. These effects should be considered in the structural design.

3.4.7 Creep

The phenomenon of creep has a number of similarities with shrinkage. Most factors that affect shrinkage also affect creep. Shrinkage occurs under no load whereas creep occurs under a state of sustained stress. Typical variations of creep and shrinkage strains with time are shown in Fig. 3.2 1. Figure 3.2 1(*a*) shows the variation of shrinkage strain obtained using an unloaded specimen. If the specimen is sealed and subjected to sustained stress, the variation of strain with respect to time is shown in Fig. 3.21(*b*). Since the specimen is sealed, the shrinkage is essentially eliminated. As soon as the specimen is loaded, there is an elastic response producing an elastic or instantaneous strain. The strain continues to increase under the sustained stress. This additional strain is called creep strain. If the unsealed specimen is kept under sustained stress, both creep and shrinkage phenomenon occur. In addition, more shrinkage occurs because of the stress. The stress aids the movement of water, resulting in additional shrinkage strain.

As mentioned earlier, the factors that affect shrinkage also affect creep. The influences are similar in most cases. The factors that have less effect on creep than on shrinkage are type of cement and carbonation. The following additional factors influence the creep strain.

Level of Stress

There is a proportionality between the magnitudes of sustained stress and creep if the level of stress is less than 50% of compressive strength. Hence the creep strain is proportional to the elastic strain under normal working load conditions. However, if the level of stress increases beyond 70% of compressive strength, excessive creep strain occurs, leading to failure.

Time of Loading

The time of loading influences the creep strain because the strength of concrete increases with time. For example, if the concrete is subjected to sustained load at 7 days, it undergoes more creep strain as compared to concrete loaded at 28 days. It should be noted that the level of stress should be the same for both loading conditions. The influence of the time at loading decreases after 28 days and

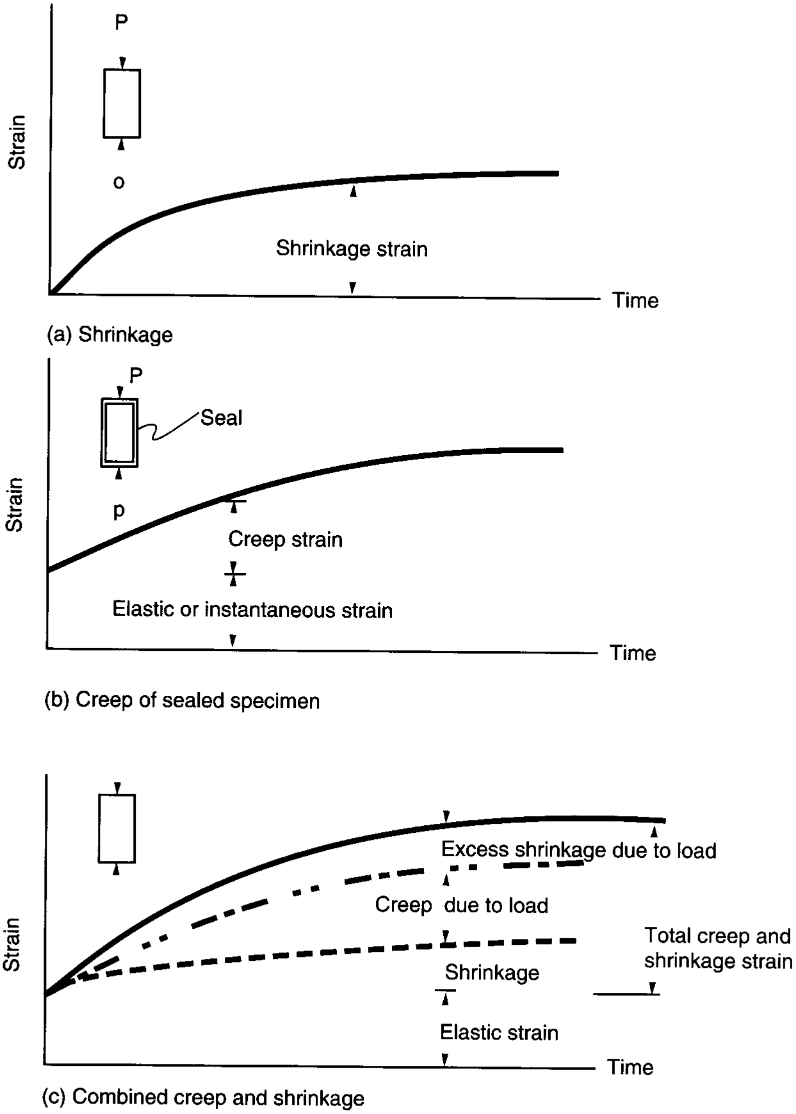


FIGURE 3.21 Typical variations of creep and shrinkage strains with time. (a) Shrinkage. (b) Creep of sealed specimens. (c) Combined creep and shrinkage.

becomes almost insignificant if the time at loading exceeds 56 days. This should be expected because the change in compressive strength after 56 days is negligible for normal concrete.

As is the case of shrinkage strain, creep strain increases deflections and crack widths, causes loss of prestress, and results in a redistribution of stresses in indeterminate structures and composite members. In the case of reinforced concrete columns, the redistribution of stresses could result in yielding of steel and buckling of eccentrically loaded columns.

Unloading typically leads to elastic and creep recovery. As in the case of shrinkage, creep recovery is not complete, leading to some amount of permanent deformation.

3.4.8 Estimation of Creep and Shrinkage Strains

A number of rheological models have been developed to simulate creep and shrinkage. These models consisting of Kelvin solid and Maxwell fluid can be used to predict the behavior of concrete subjected to various modes of loading such as constant stress, constant strain, and loading for a certain amount of time. These models can predict both creep and creep recovery. However, most designers prefer to use simple empirical equations for the prediction of creep and shrinkage strains.

A number of empirical equations exist for the prediction shrinkage and creep at a given time under load. The equations recommended by ACI Committee 209[18] are

$$(\epsilon_{sh})_t = \frac{t}{35 + t}(\epsilon_{sh})_u \tag{3.24}$$

$$\gamma = \frac{t}{10 + t^{0.6}}(C_u) \tag{3.25}$$

where $(\epsilon_{sh})_t$ = shrinkage strain at time
$(\epsilon_{sh})_t$ = ultimate shrinkage strain
γ_t = creep coefficient
$(C)_u$ = ultimate creep coefficient

For a given concrete the ultimate creep coefficient and ultimate shrinkage strain have to be assumed. These values can also be obtained using short-term readings taken at, say, 14, 28, or 56 days.

The equations recommended by the Comité Euro-International du Béton (CEB), Paris, France,[19] involve the use of coefficients for various exposure conditions and hence are more accurate. The shrinkage creep strains are computed using the following equation:

$$\epsilon_{sh} = \epsilon_c K_b K_t K_e \tag{3.26}$$

where

$$\epsilon_{cr} = \phi \text{ (elastic strain)} \tag{3.27}$$

$$\phi = K_c K_b K_d K_t' K_e' \tag{3.28}$$

where ϵ_{sh}, K_b, K_e, K_t, K_c, K_d, K_t', and K_e' are coefficients. The values for these coefficients can be obtained using Figs. 3.22 and 3.23.

3.4.9 Behavior under Multiaxial Stresses

In some structures such as two-way slabs and nuclear reactor containment vessels the concrete is subjected to biaxial and triaxial stresses. Another example for triaxial state of stress is off-shore oil platforms. Concrete located near the bottom of the sea is subjected to water pressure and external load, resulting in a triaxial state of stress.

In the case of biaxial loading, the strength of concrete increases by about 20% under biaxial compression. Figure 3.24 shows the failure envelope for concrete subjected to biaxial loading tn tension and compression. The increase in strength under biaxial loading is normally neglected in de-

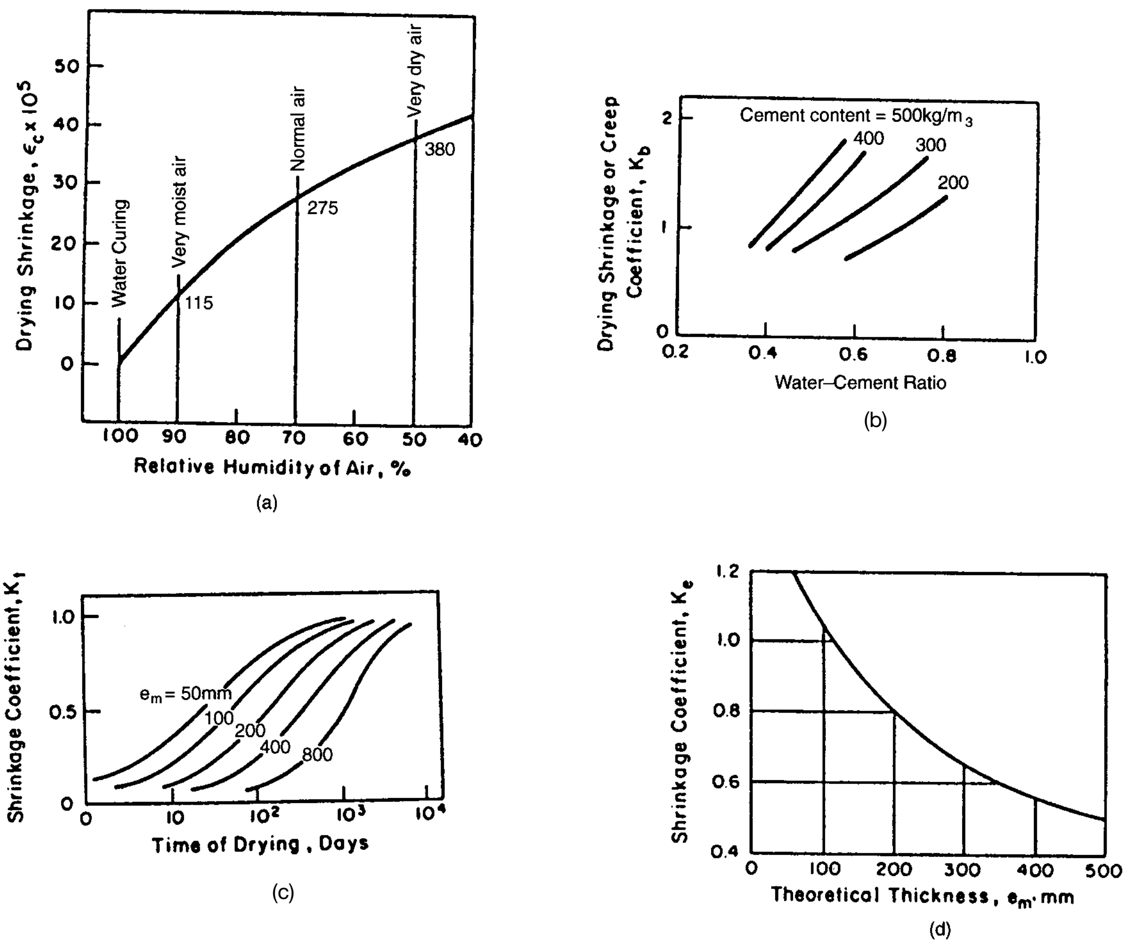


FIGURE 3.22 Coefficients ϵ_c, K_b, K_t, and K_e for CEB method. (*From CEB/FIP.*[19])

sign computations. Hence structural components subjected to biaxial loading can be designed using uniaxial strengths.

Concrete subjected to triaxial loading can withstand much higher stresses as compared to uniaxial strength, as shown in Fig. 3.25. In general, triaxial compression improves the compressive strength of leaner or low-strength concrete more than that of a stronger or high-strength concrete. Failure theories have been developed for the triaxial state of stress. The most popular ones are known as octahedral shear stress theory and Mohr-Coulomb failure theory. Some researchers have also developed empirical relationships between minor axial stress σ_3 and major axial stress σ_1, which creates failure. Under the uniaxial state of stress σ_3 is zero, and hence σ_1 is the compressive strength f'_c. One such empirical equation is

$$\sigma_1 = f'_c + 4.8\,\sigma_3 \tag{3.29}$$

where f'_c is the compressive strength.

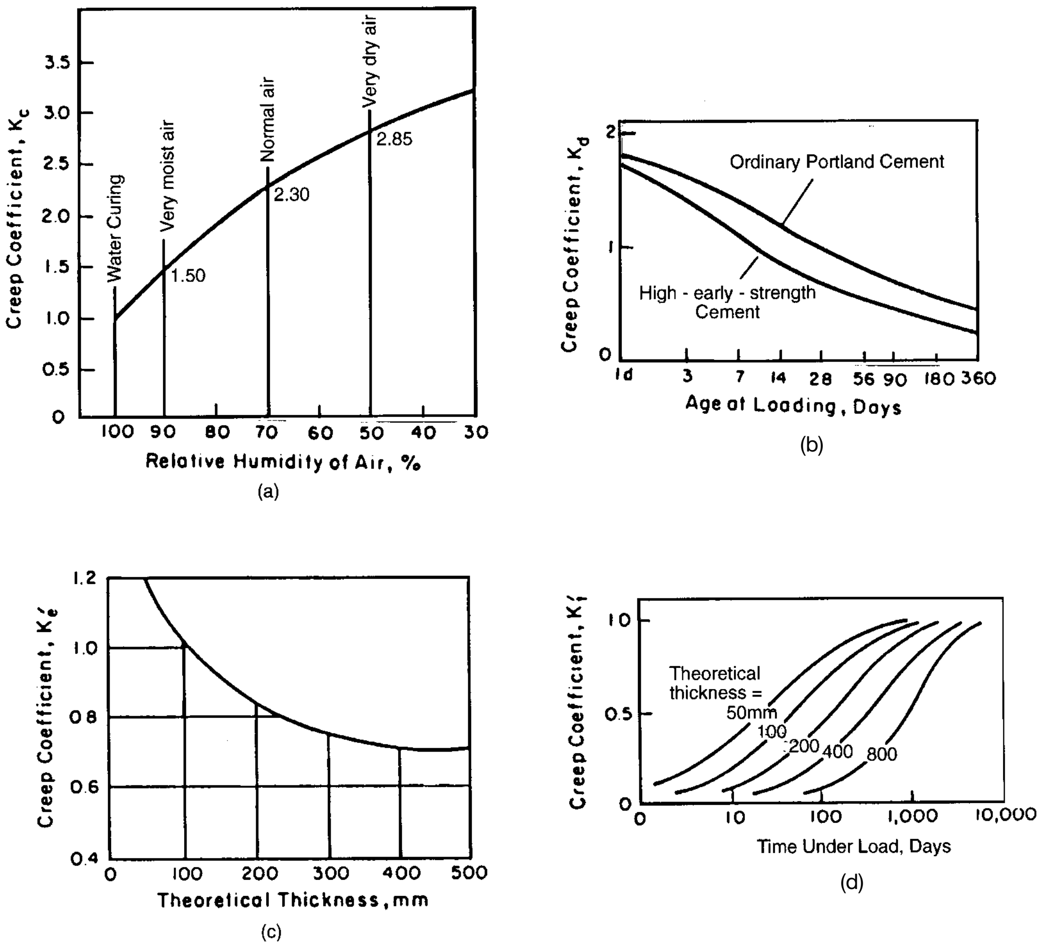


FIGURE 3.23 Coefficients K_c, K_d, K_e', and K_t' for CEB method. (*From CEB/FIP.*[19])

For example, concrete with a compressive strength of 4000 psi (28 MPa) can be expected to withstand 8800 psi (61 MPa) under a lateral pressure of 1000 psi (6.89 MPa) ($\sigma_1 = 4000 + 4.8 \times 1000$). More detailed information regarding various theories and stress-strain behavior can be found in the literature.

3.4.10 Fatigue Loading

In fatigue loading, the structural components are subjected to varying stresses. Typical examples are bridge beams, offshore structures subjected to wave loads, and machine foundations. Extensive research have been conducted on the behavior of concrete subjected to fatigue loading. The following major conclusions, arrived at by the various investigators, may be useful for the designer.

- Concrete can withstand 10 million cycles if the stress range (difference between maximum and minimum stresses) is less than 55% of compressive strength and the minimum stress is about zero.

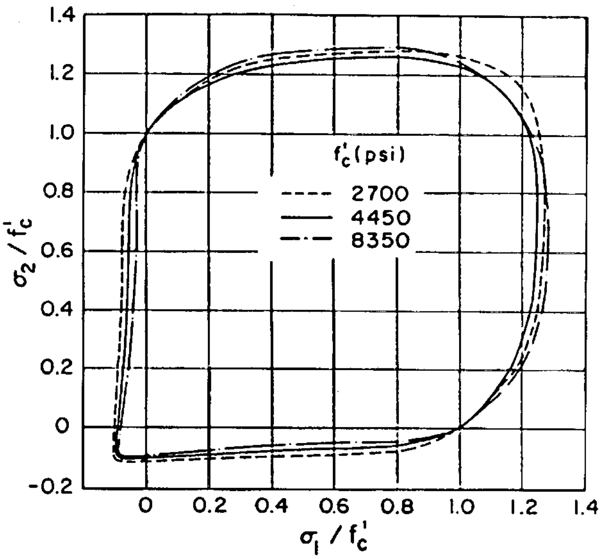


FIGURE 3.24 Biaxial stress interaction curves. (*From Kupfel et al.*[19])

- The fatigue strength decreases with an increase in minimum strength.
- Components subjected to different intensities of fatigue loading can be designed using Minor's hypothesis.
- Frequency of loading of between 70 and 900 cycles has little effect on fatigue strength, provided the stress range is less than 70% of compressive strength.
- The variables of mix proportion, such as water–cement ratio and cement content, affect fatigue strength in the same way as static compressive strength. Behavior in tension and flexure is about the same as behavior in compression. For example, the flexural fatigue strength at a stress range of 55% of static flexural strength (modulus of rupture) is about 10 million cycles.
- The stress gradient increases the fatigue life.
- Rest periods do not affect the fatigue life significantly.
- The creep strain under fatigue loading is higher than the creep strain under the static load that corresponds to the maximum fatigue load. In other words, even though the average stress is lower for fatigue load conditions the creep strain is higher.

3.5 COLD-WEATHER CONCRETING

When the temperature falls below 20°F (–6°C), the hydration of cement becomes extremely slow. Therefore concrete placed at low temperatures should be protected until it gains sufficient strength to resist freezing action. Normally the minimum recommended compressive strength is 500 psi (4 MPa). Hydration of cement also reduces the degree of saturation because the chemical action consumes the water.

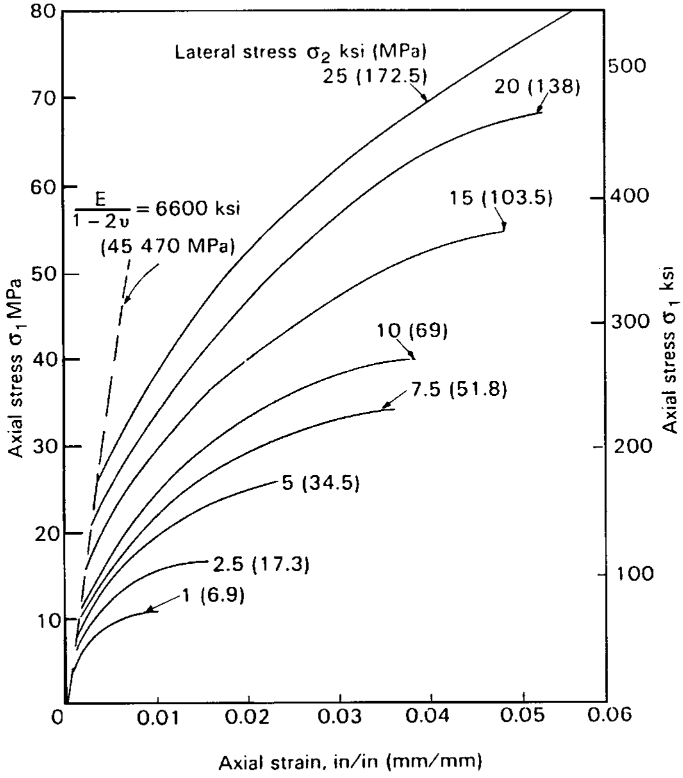


FIGURE 3.25 Behavior in triaxial compression.

The practice recommended by ACI Committee 306 for cold-weather concreting is shown in Table 3.19. The committee recommends that the concrete be maintained at a certain minimum temperature for lb 3 days, depending on whether it is the conventional or the high-early-strength type. For moderately and fully stressed members longer durations are recommended. The recommended minimum temperature depends on the exposure temperature and the thickness of the specimen. Since thicker specimens dissipate the heat of hydration more slowly, they could be maintained at a lower temperature than thin sections. Non-air-entrained concrete should be protected for at least twice the number of days because it is much more susceptible to freeze-thaw damage than air-entrained concrete. In most cases the concrete could be maintained without using external heat sources if proper care is taken to maintain the temperature of the ingredient materials and the insulation is properly placed. The effect of frozen ground, reinforcing bars, and formwork should be considered in computing the temperature requirements of the ingredients. For very thin sections external heat may be needed to maintain the recommended concrete temperature. Temperatures higher than 70°F (21 °C) are not normally recommended.

Most specifications are written based on minimum compressive strength. It is assumed that if the concrete has the specified compressive strength, it is durable. This may not be true in all cases, particularly for concrete exposed to cold weather in its fresh state. The frost action could impart considerable internal damage, making the concrete less durable. If durability is a main consideration the concrete should be protected for longer periods than recommended in Table 3.19.

The temperature of the fresh concrete can be controlled by controlling the temperature of the

TABLE 3.19 Recommended Concrete Temperatures for Cold-Weather Construction, Air-Entrained Concrete

	Sections < 12 in (0.3 in) thick		Sections 12–36 in (0.3–0.9 in) thick		Sections 36–72 in (0.9–1.8 in) thick		Sections > 72 in (1.8 in) thick	
	°F	°C	°F	°C	°F	°C	°F	°C
Minimum temperature for fresh concrete as mixed in weather indicated								
Above30°F(–l°C)	60	16	55	13	50	10	45	7
0 to 30°F(–18to1°C)	65	18	60	16	55	13	50	10
Below 0°F (–18°C)	70	21	65	18	60	16	55	13
Minimum temperature for fresh concrete *as placed and maintained*	55	13	50	10	45	7	40	5
Maximum allowable *gradual* drop in temperature in first 24 h after end of protection	50	28	40	22	30	17	20	11

Source: From ACt 306 R-SS.[21]

constituent materials, namely, cement, aggregates, and water. Since the specific heat of water is 1.0 as compared to 0.22 for cement and aggregates, it is more efficient to heat the water. In addition it is easier to heat the water than the other ingredients used in concrete.

If the outside temperature is above freezing, aggregates are not usually heated. In temperatures below freezing, heating of fine aggregates could be sufficient. Coarse aggregate is heated only as a last resort because it is more difficult to heat loosely packed materials. Fine aggregates are generally heated by circulating hot air or steam through pipes that are embedded in them. In any case, the final temperature of the freshly mixed concrete should be maintained at the specified level. The temperature of fresh concrete T can be estimated using the following equation:

$$T = \frac{0.22(T_a W_a + T_c W_c) + T_w W_w + T_{wa} W_{wa}}{0.22(W_a + W_c) + W_w + W_{wa}} \qquad (3.30)$$

where T_a, T_c, T_w, and T_{wa} are the temperatures of aggregate, cement, water, and free moisture in aggregates, respectively, and W_a, W_c, W_w, and W_{wa} are the weights of aggregate, cement, water, and free moisture in aggregates, respectively. All the temperatures are in °F and the weights are in pounds. For SI units the temperatures are expressed in °C and the weights in kilograms. The temperatures of concrete should be checked using thermometers. Both mercury and bimetallic thermometers are available for that purpose.

3.6 HOT-WEATHER CONCRETING

Concreting in hot weather leads to a rapid hydration rate and evaporation loss, resulting in microcracks and inferior final product. Since lower humidity and higher wind velocity also lead to rapid evaporation, hot weather for concreting purposes is taken as a combination of temperature, relative humidity, and wind velocity. If the relative humidity is high, such as near sea shores, special precautions may not be needed up to about 85°F (29°C). If the humidity is very low, such as in desert conditions, precautions might be needed even at temperatures lower than 80°F (27°C). The combined effect of temperature, relative humidity, and wind velocity can be judged using the amount of water that evaporates from fresh concrete. Nomograms are available for estimating the rate of evaporation for a given site condition.

Solar radiation can also affect the properties of fresh concrete in a number of ways. If the ingredients are stored outside, they can absorb heat during transport or during the waiting period. Heating of reinforcement and formwork can further aggravate the situation.

Fresh Concrete

Hot weather adversely affects workability. For the same workability, or to improve the workability, more water is needed at higher temperatures. Unfortunately the excess water added results in reduced strength and less durable concrete. Hence it is advisable to use water-reducing admixtures such as superplasticizers rather than excess water. Researchers have shown that superplasticizers can be used effectively to increase workability without adversely affecting strength and durability.

Concrete stiffens much faster at higher temperatures. The rate of hydration approximately doubles for every 18°F (10°C) increase in temperature. Hence the workability decreases much faster at high temperatures. The change in workability can be best represented by the loss in slump values. Figure 3.26 shows the variation of slump at two different temperatures. It can be seen that slump loss is much more rapid at 95°F (35°C) than at 73°F (23°C). Set-retarding admixtures may be used to delay the stiffening of concrete.

When the concrete is placed in position, vibrated, and the top surface finished to the desired texture, a certain amount of water rises up to the surface. This water, called bleed water, should be kept to a minimum. The bleed water does help to prevent shrinkage cracking. However, at high temperatures the bleed water may evaporate rapidly, causing excessive shrinkage cracks.

Hardened Concrete

Concrete placed and cured at higher temperatures develops higher strengths at the early ages of maturity. But the final strength, measured after 28 days, decreases with an increase in placing and curing temperature. The reduction in strength occurs both in the compression and the flexure mode. Typical variations of compressive strengths at various temperatures are shown in Fig. 3.27.

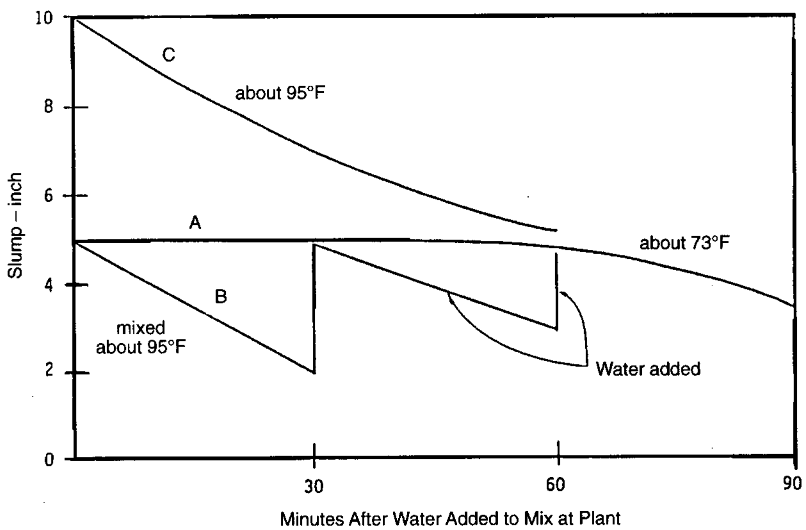


FIGURE 3.26 Influence of temperature on slump variation with time. (*From Shilstone.*[22])

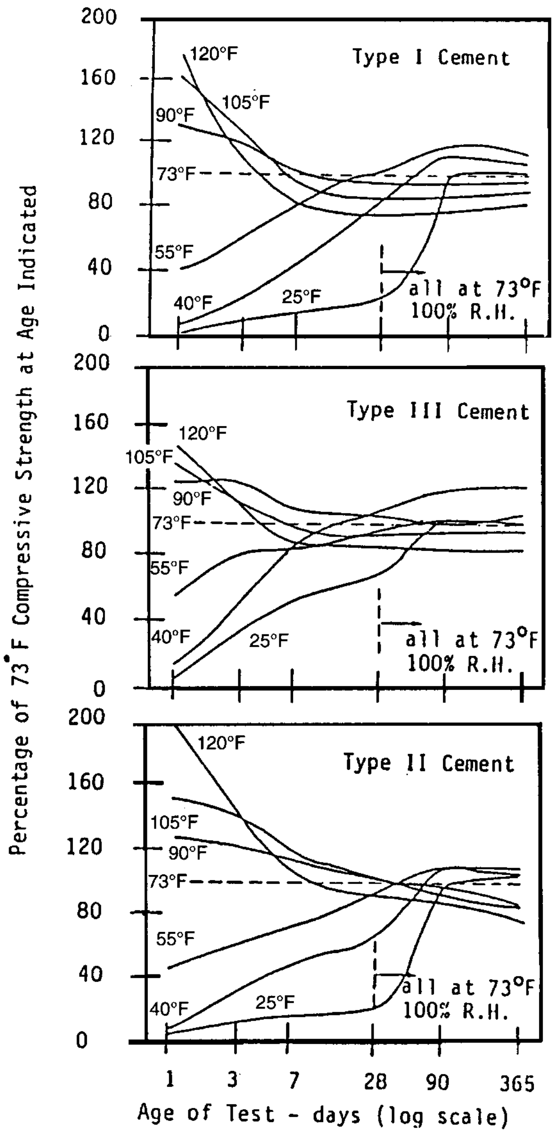


FIGURE 3.27 Effect of temperature on compressive strength for different types of cement. (*From Kliegar.*[23])

Concrete cast during hot weather develops more shrinkage cracks. This results in more permeable and less durable concrete. In general, concrete placed at high temperatures without taking special precautions can be considered weak under both thermal (freeze-thaw) and chemical attacks.

In general, creep of concrete increases with the increase in temperature. Concrete exposed to temperatures higher than 90°F (32°C) could undergo considerably more creep strains. In certain cases the higher creep strains should be taken into consideration in structural design.

Precautionary Measures for Hot-Weather Concreting

The best approach is to cast concrete at temperatures lower than 85°F (29°C). In some instances concrete could be cast during the evening or night hours rather than the daytime. If concrete has to be placed at higher temperatures, the best way to avoid problems is to keep the concrete temperature low. This can be achieved by cooling the aggregates or the water.

Using ice in the mixing water is the most efficient way to lower the concrete temperature. The approximate temperature of the concrete can be calculated using Eq. (3.30). If ice is used as part of the mixing water, the equation can be modified as follows:

$$T = \frac{0.22(T_a W_a + T_c W_c) + T_w W_w + T_{wa} W_{wa} - 112\, I}{0.22(W_a + W_c) + W_w + W_{wa} + I} \tag{3.31}$$

where I is the weight of ice in pounds.

The amount of ice needed to obtain a certain concrete temperature can be computed by the following modified form of Eq. (3.31):

$$I = \frac{0.22[W_a(T_a - T) + W_c(T_c - T)] + W_{wa}(T_a - T) + W_{wa}(T_w - T)}{112 + T} \tag{3.32}$$

Shaved ice can be added to the mixer as part of the mixing water. If block ice is used, it should be crushed before adding. In either case, ice must be completely melted at the end of the mixing cycle. Water from melted ice must be considered as part of the total mixing water, keeping the water–cement ratio the same.

Under certain circumstances, liquid nitrogen has been shown to be economical and practical, especially if low concrete temperatures are needed. However, this method can be used only when nitrogen manufacturing facilities are available locally. Liquid nitrogen can be used to cool the aggregates, the water, or the concrete mix. Experience shows that a combination provides a repeatable and quality mix.

Some of the proven techniques helpful for hot-weather concreting are listed here. The recommendations are not elaborated upon for lack of space.

- Keep the concrete temperature low, preferably below 85°F (29°C).
- Provide shade over stockpiles of coarse and fine aggregates.
- Use chilled water or partly replace mixing water with ice for mixing concrete, or use liquid nitrogen to cool the concrete.
- Keep the cement content to the minimum required for strength and durability.
- Use appropriate water-reducing, superplasticizing, and set-retarding admixtures after establishing, under site conditions, dosage and compatibility with the cement used.
- Paint all mixing and conveying equipment for concreting with reflective or light-colored paint.
- Precool the forms, reinforcement, and surroundings prior to concrete placement.
- Place concrete at night.
- Place concrete in layers of optimum thickness for efficient compaction and avoidance of cold joints.
- Wet or moist curing and membrane protection are necessary after the concrete has hardened.

Retempering of Concrete in Hot Weather

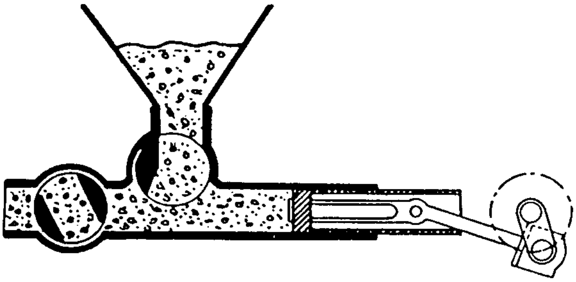
Normally, retempering of concrete to improve its workability is considered a bad option. But recent research shows that the retempering technique can be used successfully without adverse effects. A study conducted using water and superplasticizer for retempering concretes, mixed at temperatures up to 140°F (60°C), led to the following conclusions.

- Additional water and cement are needed (maintaining the same water–cement ratio) at higher ambient temperatures to achieve the same slump. The additional water demand is very high for concretes with low water-to-cement ratios (0.4) at temperatures higher than 104°F (40°C). However, for concretes with higher water–cement ratios (0.5 and 0.6) there is only a slight increase in the water required to maintain the same slump for a temperature range of 86 to 140°F (30 to 60°C).
- For all concretes the quantity of retempering water required to restore their initial slumps, after an elapse of a 30-min period, increases with an increase in the ambient temperature for both first and second retemperings. The quantity of water needed for second retempering is significantly higher than that for first retempering at all temperatures. Concretes with lower water–cement ratios (0.4) need considerably higher quantities of retempering water at higher temperatures.
- The cohesiveness and finishability of concrete seems to be better after retempering than after initial mixing.
- Slump loss is considerably higher for a concrete with a water–cement ratio of 0.4 than for concretes with water–cement ratios of 0.5 and 0.6. After retempering the rate of slump loss is higher for all concretes. The rate of slump loss is not significantly higher at higher temperatures.
- No appreciable change in the unit weight of fresh concrete occurs after first and second retemperings at all temperatures tested. For concretes with low water–cement ratios (0.4) an increase in the ambient temperature causes a decrease in the plastic unit weight.
- There is no apparent change in the entrapped air either due to a temperature increase or due to retempertngs.
- All hardened concrete properties (compressive strength, splitting tensile strength, flexural strength, static modulus of elasticity, dynamic modulus, pulse velocity, and dry unit weight) are affected similarly by an increase in the ambient temperature from 86 to I 40°F (30 to 60°C) and due to first and second retemperings. There is a successive, though not significant, reduction in the strength and modulus values after first and second retemperings. There is a slight reduction (less than 5%) as the temperature increases from 86 to 1400F (30 to 60°C). The concretes most affected by temperature increase are those with a water–cement ratio of 0.4.
- There is no positively recognizable change in the relationships between the various properties of hardened concrete either due to an increase in temperature from 86 to 1400F (30 to 60°C) or due to two retemperings.
- There is no significant difference in the properties, particularly in the case of compressive strength, of concretes mixed, cast, and cured under identical conditions and with two different agents, namely, superplasticizer and water. The observed difference between the two is less than 15%. Moreover the variation is not consistent. A statistical analysis conducted using regression equations relating the compressive strength and other properties, such as splitting tensile strength, flexural strength, pulse velocity, static modulus, dynamic modulus, and dry unit weight, confirms the observation that the two different retempering agents have the same influence on the properties of concrete.

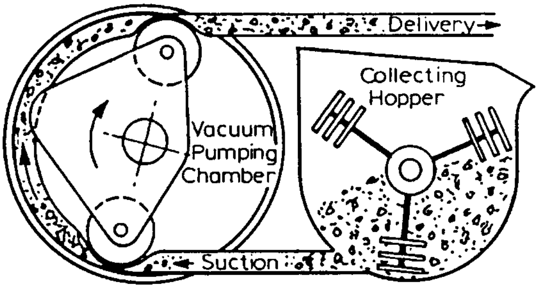
3.7 PUMPING OF CONCRETE

In modern construction, pumping of concrete has become quite common. Pumping has a number of advantages, such as providing a continuous supply of concrete, access to hard to reach places, and economy . The pumping system consists of a hopper, a concrete pump, and pipes that can be connected and dismantled easily.

Two typical pumps are shown in Fig. 3.28. Direct-acting horizontal piston pumps shown in Fig. 3.28(a) are more common. The semirotary valves allow the passage of coarse aggregate particles. Concrete, which is fed into the hopper from the mixer, gets sucked into the pipes by gravity and the



(a)



(b)

FIGURE 3.28 Concrete pumps. (a) Direct-acting horizontal piston pump. (b) Squeeze-type pump.

vacuum action created during the suction stroke. These pumps can pump up to 1500 ft (457 in) horizontally and 140 ft (43 in) vertically.

The squeeze-type pumps shown in Fig. 3.28(b) are normally smaller and truck-mounted. In this pump concrete placed in the collecting hopper is fed by rotating blades into a pliable pipe located in the pumping chamber. The pumping chamber, which can maintain a vacuum of about 26 in (660 mm) of mercury, supplies continuous feed to the delivery pipe. Delivery is normally done using folding boom consisting of 3- and 4-in (75- and 100-mm) pipes. Squeeze pumps can pump up to 300 ft (91 in) horizontally and 100 ft (30 in) vertically.

Different pump and pipe sizes are available. Squeeze pumps can deliver up to 25 yd^3 (19 m^3) per hour whereas piston pumps can deliver up to 80 yd^3 (61 m^3) per hour. Piston pumps work with pipes up to 9 in (230 mm) in diameter. The pipe diameter should be at least three times the maximum size of the aggregate. Hence even squeeze pumps with 3-in (75-mm)-diameter pipes can handle most concrete used for buildings.

Two types of blockages can occur in pumping. The first occurs due to segregation, the second due to excessive friction. When the concrete has too much water, the solid particles consisting of aggregates cannot be carried through by the liquid medium because the water escapes through the mix. Since water is the only medium pumpable in its natural state, the aggregates stay behind and get clogged. If the mix is cohesive, then the water will carry the solid particles with it.

When the mix is very cohesive, the friction between the walls and the mix becomes high and the pump cannot overcome this friction, which will result in a blockage. This type of failure is more common in high-strength concrete mixes and in mixes containing a high proportion of very fine material such as crushed dust or fly ash.

The optimum mix is the one that produces maximum internal frictional resistance within the ingredients and minimum frictional resistance against the pipe walls. Void sizes should also be minimum. For concrete containing 0.75-in (19-mm) maximum size aggregate, fine aggregate should be in the range of 35 to 40%. Of these 15 to 20% should pass through ASTM sieve 50, or finer than 300 μm.

Concrete should be mixed well before feeding it into the hopper. In some cases additional mixing is done in the hopper using stirrers. The mix cannot be too harsh, too dry, too wet, or too sticky. A slump of 1.5 104 in (38 to 100 mm) normally produces satisfactory results. Since pumping provides some compaction, the mix at the delivery point could have a slump lowered by as much as 1 in (25 mm). When the concrete is at the correct consistency, a thin lubricating film forms near the surface of the pipes, allowing smooth flow of concrete.

The following are some of the additional factors to be considered in pumping concrete.

- Pumping is economical only if it can be used over long uninterrupted periods because considerable effort is needed for lubricating and cleaning at the beginning and end.
- A short piece of flexible hose near the end makes the placement easier. But the flexible hose could increase the friction loss.
- Bends should be kept to a minimum.
- Aluminum pipes should not be used because they react with alkalis in cement and generate hydrogen bubbles, weakening the hardened concrete.
- The shape of the aggregate influences the pumpability of the mix. Natural sands are preferable to crushed sands because of their spherical shape and continuous uniform grading.
- The presence of entrained air increases the pumping effort. If large amounts of entrained air are present, the entire movement of the piston could be wasted on compressing the air bubbles, resulting in no flow. In general air-entrained concrete can be pumped only shorter distances as compared to non-air-entrained concrete.
- Typically pumping lightweight aggregate concrete needs more effort. Sealing of the surface of the aggregates may be necessary, or special admixtures may be needed to pump lightweight concrete. Otherwise aggregates can absorb more water under pressure, making the mix stiffer and hence more difficult to pump. Some of the aggregate may also crush during the pumping process.
- Concrete with unsatisfactory conditions in the fresh state cannot be pumped. Pumpable concrete, in almost all cases, has the right consistency for placing and finishing.

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P • A • R • T • 4

REINFORCED CONCRETE FOUNDATIONS

SECTION 4A

FUNDAMENTALS OF REINFORCED CONCRETE

A. SAMER EZELDIN

NOTATIONS

a = depth of equivalent rectangular stress block as defined in Sec. 4A.2.4. 1

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral

A_g = gross area of concrete section g

A_s = area of tension steel reinforcement
area of steel per unit width of slab or plate

A'_s = area of compression steel reinforcement

A_{st} = total area of longitudinal reinforcement

b = width of cross section

b_w = web of cross section

c = distance from extreme compression fiber to neutral axis

d = distance from extreme compression fiber to centroid of tension reinforcement

d' = distance from extreme compression fiber to centroid of compression reinforcement
d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto
E_c = modulus of elasticity of concrete
E_s = Young's modulus of steel
EI = flexural stiffness of compression member
f_c = compressive strength in concrete
f'_c = cylinder compressive strength of concrete
f_r = modulus of rupture of concrete
f_s = calculated stress in reinforcement at service loads
f'_t = tensile splitting strength of concrete
f_y = specified yield strength of nonprestressed reinforcement
h = overall thickness of member
I_{cr} = moment of inertia of cracked section
I_e = effective moment of inertia for computation of deflection
I_g = gross moment of inertia
k = effective length factor for compression members
L = clear long span length
l = effective beam span
l_d = development length
l_e = equivalent embedded length of a hook
M = positive moment
M' = negative moment
M_a = moment of maximum service load in span
M_{cr} = cracking moment
M_n = nominal moment strength at section
P_b = nominal axial load strength at balanced strain conditions
P_c = critical load
P_n = nominal axial load strength at given eccentricity
P_0 = nominal axial load strength at zero eccentricity
P_u = factored axial load at given eccentricity; $\leq \phi P_n$, ϕ being a reduction factor
s = spacing of stirrups or ties
V_c = nominal shear strength provided by concrete
V_s = nominal shear strength provided by steel reinforcement
W = total uniform load per unit area
x = shorter overall dimension of rectangular part of cross section
x_1 = shorter center-to-center dimension of closed rectangular stirrup
y = longer overall dimension of rectangular part of cross section
y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension
y_1 = longer center-to-center dimension of closed rectangular stirrup

β_1 = factor, varying from 0.85 for f'_c = 4000 p5i 100.65 minimum; it decreases at a rate of 0.05 per 1000-psi strength above 4000 psi (27.6 MPa)

δ = moment magnification factor

ε_s = unit strain in reinforcement steel

ν = Poisson's ratio

ρ = ratio of nonprestressed tension reinforcement; A_s/bd for beam and $A_s/12d$ for slab

$\bar{\rho}_b$ = reinforcement ratio producing balanced strain conditions

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of spirally reinforced compressive member

ρ_t = active steel ratio; = A_s/A_t

ω = reinforcement index; = $\rho f_y/f'_c$

Σ^0 = sum of circumferences of reinforcing elements

$\Sigma x^2 y$ = torsional section properties

4A.1 INTRODUCTION

Concrete is obtained by mixing cement, water, fine aggregate, coarse aggregate, and frequently other additives in specified proportions. The hardened concrete is strong in compression but weak in tension, making it vulnerable to cracking. Also, concrete is brittle and fails without warning. In order to overcome the negative implications of these two main weaknesses, steel bars are added to reinforce the concrete. Hence reinforced concrete, when designed properly, can be used as an economically strong and ductile construction material.

In many civil engineering applications, reinforced concrete is used extensively as a construction material for structures and foundations. This chapter provides a basic knowledge of reinforced concrete elements (beams, columns, and slabs) subjected to flexure, shear, and torsion. Once the response of an individual element is understood, the designer will have the necessary background to analyze and design reinforced concrete systems composed of these elements, such as foundations and buildings.

Two popular methods are available for analyzing and designing the strength of reinforced concrete members. The first is referred to as the working stress design method. This method is based on limiting the computed stresses in members as they are subjected to service loads up to the allowable stresses. The second design method, called the strength method, is based on predicting the maximum resistance of a member rather than predicting stresses under service loads.

The strength method is the design method recommended by the current edition of the ACI code.1 Members are designed for factored loads that are greater than the service loads. The factored loads are obtained by multiplying the service load by load factors greater than 1. Table 4A.gives the load factors for various types of load. Using factored loads, the designer performs an elastic analysis to obtain the required strength of the members. Members are designed so that their design strength is equal to or greater than the required strength,[2–4]

$$\text{Required strength} \leq \text{design strength} \tag{4A. 1}$$

The design strength is used to express the nominal capacity of a member reduced by a strength reduction factor ϕ. The nominal capacity is evaluated in accordance with provisions and assumptions specified by the ACI code. Reduction factors for different loading conditions are presented in Table 4A.2. The design criteria just presented provide for a margin of safety in two ways. First, the required strength is computed by increasing service loads by load factors. Also, the design strength

TABLE 4A.1 Load Factors for Ultimate-Strength Design Method

Condition	U
1. Dead load D + live load L	$1.4D + 1.7L$
2. Dead + live + wind load W when additive	$0.75(1.4D + 7L + 1.7W)$
3. Same as item 2 when gravity counteracts wind-load effects	$0.9D + 1.3W$
4. In structures designed for earthquake loads or forces E, replace W by $1.1E$ in items 2 and 3	
5. When lateral earth pressure H acts in addition to gravity forces when effects are additive	$1.4D + 1.7L + 1.7H$
6. Same as item 5 when gravity counteracts earth-pressure effects	$0.9D + 1.7H$
7. When lateral liquid pressure F acts in addition to gravity loads, replace $1.7H$ by $1.4F$ in items 5 and 6	
8. Vertical liquid pressures shall be considered as dead loads D	
9. Impact effects, if any, shall be included in live loads L	
10. When effects T of settlement, creep, shrinkage, or temperature change are significant and additive	$0.75(1.4D + 1.4T + 1.7L)$
11. Same as item 10 when gravity counteracts T	$1.4(D + T)$
12. In no case shall U be less than given by item 1	

Source: From ACI.[1]

TABLE 4A.2 Reduction Factors for Ultimate-Strength Design Method

Kind of strength	Strength reduction factor ϕ
Flexural, with or without axial tension	0.90
Axial tension	0.90
Axial compression, with or without flexure:	
Members with spiral reinforcement	0.75
Other reinforced members	0.70
Exception: for low values of axial load, 4 may be increased in accordance with the following:	
For members in which f_y does not exceed 60,000 psi, with symmetrical reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_s$ to zero.	
For other reinforced members, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ or ϕP_{nb}, whichever is smaller, to zero.	
Shear and torsion	0.85
Bearing on concrete	0.70
Flexure in plain concrete	0.65

Source: From ACI.[1]

is computed by reducing the nominal strength by a strength reduction factor. This design criterion applies to all possible states of stress, namely, bending, shear, torsion, and axial stresses.

4A.2 FLEXURE BEHAVIOR

Four basic assumptions are made when deriving a general theory for flexure behavior of reinforced concrete members.

1. Plane sections before bending remain plane after bending.
2. The stress-strain relationships for the steel reinforcement and the concrete are known.
3. The tensile strength of the concrete is neglected after cracking.
4. Perfect bonding exists between the concrete and the steel reinforcement.

The simple reinforced concrete beam shown in Fig. 4A.1(*a*) exhibits the basic characteristics of flexural behavior. Depending on the magnitude of the bending moment, the beam's response will be in the elastic range, either uncracked or cracked, or in the inelastic range and cracked.

4A.2.1 Uncracked, Elastic Range

When the bending moment is smaller than the cracking moment, the strain and stress distributions at the maximum moment section are as shown in Fig. 4A.l(*b*). Stresses are related to strain by the modulus of elasticity of concrete E_c. The reinforcement in concrete makes only a minor contribution at this stage, because its strain is too small to achieve an appreciable resisting stress. The internal force couple formed by the tension and compression forces in concrete provides the required resistance to the externally applied moment. The principle of elastic theory and the transformed area concept are employed for computing stresses in concrete and in steel reinforcement.

The stress in concrete with the extreme fiber in tension is given by

$$f_t + \frac{M(h-c)}{I_t} \tag{4A.2}$$

and the stress in the steel reinforcement by

$$f_s + \frac{M(dh-c)}{I_t} n \tag{4A.3}$$

where $$n = \frac{E_s}{E_c}$$

and I_t is the moment of inertia of the transformed section.

4A.2.2 Cracked, Elastic Range

As the applied load is increased, the maximum moment reaches higher values, resulting in the maximum tension in concrete approaching the modulus of rupture. At this level, the tension cracks start forming on the tension face of the concrete section. The cracking moment can be computed using the equation

$$M_{cr} = \frac{f_r I_t}{y_t} \tag{4A.4}$$

Here I_t can be replaced by the gross moment of inertia I_g, neglecting the contribution of the steel reinforcement without appreciable error.

Under increasing loads and progressive cracking, the neutral axis shifts upward. Since concrete cannot resist the developed tensile stresses, the reinforcing steel is called upon to resist the entire

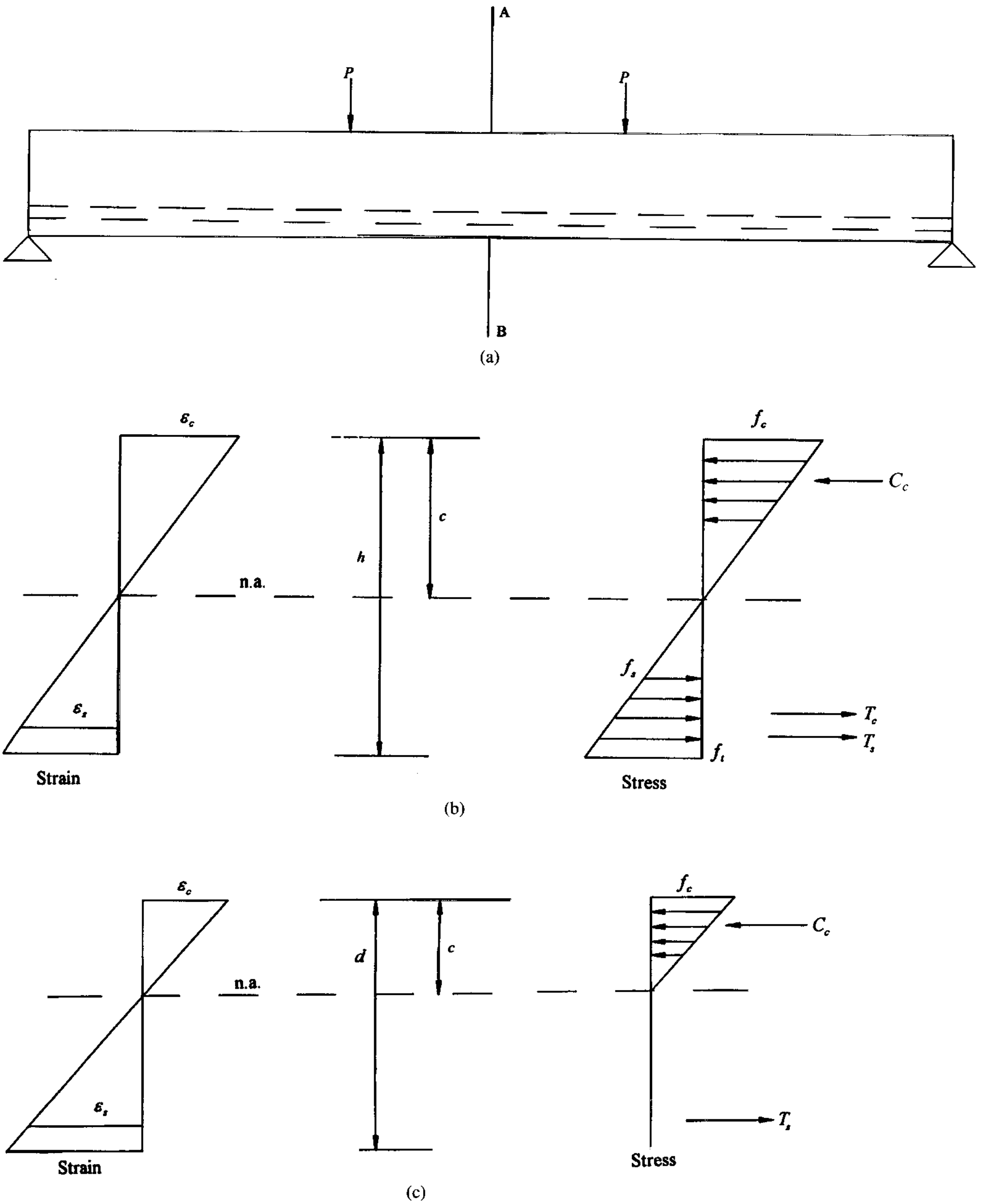
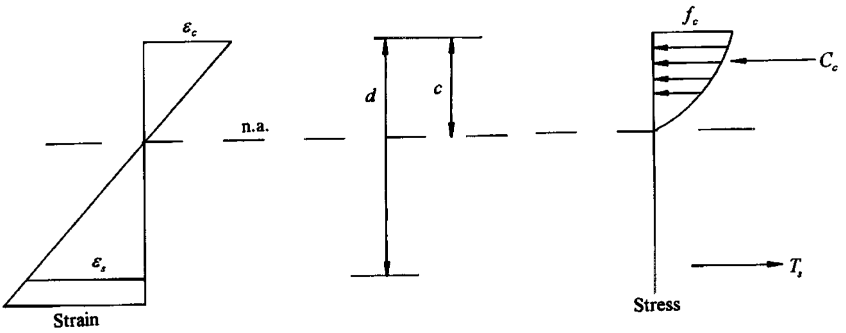
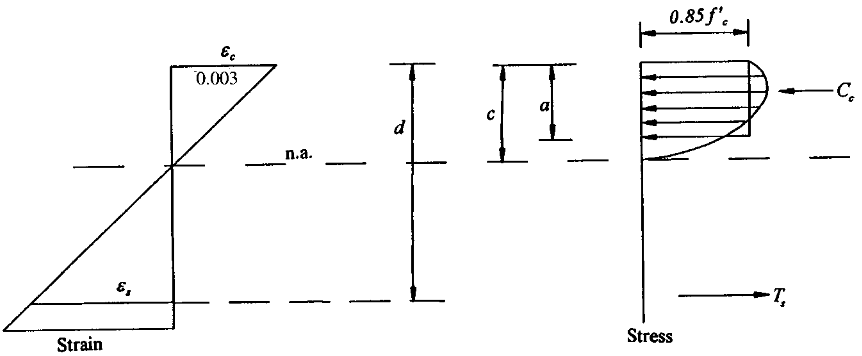


FIGURE 4A.1 Flexural behavior of simple reinforced concrete beam. (*a*) Reinforced concrete beam. (*b*) Uncracked, elastic stage. (*c*) Cracked, elastic stage.



(d)



(e)

FIGURE 4A.1 (*Continued*) (*d*) Cracked, inelastic stage. (*e*) Flexural strength.

tension force. Up to a compressive stress of about $0.5f'_c$, the linear relationship is still valid. Figure 4A.l(*c*) shows the response of a reinforced concrete section at this stage. The internal resisting couple is provided by a concrete compressive force C_c and a steel tensile force T_s. The moment equilibrium equation is given by

$$M = T_s\left(d - \frac{c}{3}\right) = C_c\left(d - \frac{c}{3}\right) \tag{4A.5}$$

The concept of transformed section can still be used to compute the stress in concrete and in steel reinforcement. However, at this stage the cracked concrete is assumed to make no contribution to I_t.

Example 4A.1 Calculate the bending stresses in the beam shown in Fig. X.1, using the transformed area method.

Given:

$f'_c = 4000$ psi (27.6 MPa)

$M = 900{,}000$ in · lb(l0l.7k N · m)

$E_s = 29{,}000{,}000$ psi (199,810 MPa)

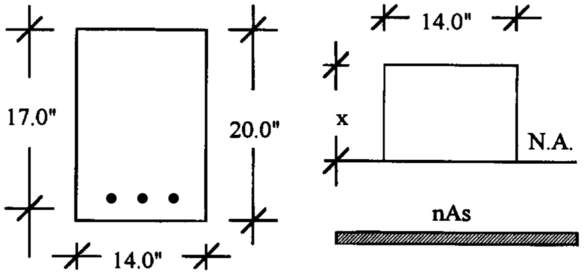


FIGURE X.1

b = 14.0 in (356 mm)

h = 20.0 in (508 mm)

d = 17.0 in (432 mm)

A_s = 3 no. 9 bars [3 in^2 (1935 mm^2)]

Solution

$$M_{cr} = \frac{f_r I_t}{y_t}$$

$$= \frac{(7.5\sqrt{4000})(14 \times 20^3/12)}{10} = 442{,}717 \text{ in} \cdot \text{lb} < 900{,}000 \text{ in} \cdot \text{lb} \qquad \therefore \text{ Section has cracked.}$$

$$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57{,}000\sqrt{f'_c}} = \frac{29 \times 10^6}{4 \times 10^6} = 7.2$$

Taking moments about the neutral axis,

$$14x\left(\frac{x}{2}\right) = nA_s(d - x)$$

$$7x^2 = 367.2 - 21.6x$$

Solving the quadratic equation we get

$$x = \frac{-21.6 \pm \sqrt{21.6^2 + 4 \times 7 \times 367.2}}{2 \times 7} = 5.86 \text{ in (148.8 mm)}$$

The moment of inertia of the transformed section is

$$I_t = \frac{bx^3}{3} + nA_s(d - x)^2$$

$$= \frac{14 \times 5.86^3}{3} + 7.2 \times 3(17 - 5.86)^2 = 3620 \text{ in}^4 \ (1506 \times 10^6 \text{ mm}^4)$$

The concrete top-fiber compression stress is

$$f_c = \frac{Mc}{I_t} = \frac{900{,}000 \times 5.86}{3620} = 1457 \text{ psi (10 MPa)}$$

The steel tension stress is

$$f_t = n\frac{Mc}{I_t} = \frac{7.2 \times 900{,}000(17 - 5.86)}{3620} = 19{,}941 \text{ psi (137.4 MPa)}$$

4A.2.3 Cracked, Inelastic Range, and Flexural Strength

With increasing loads the stresses in concrete exceed the $0.5f_c'$ value. The proportionality of stresses and strains ceases to exist, and nonlinear characteristics are observed, as shown in Fig. 4A.1(*d*).

It is a common practice to utilize an elastic-plastic idealization for the stress-strain relationship of steel. This implies that stresses and strains are related with the steel modulus of elasticity E_s up to the yield stress and its corresponding strain ε_y. At higher strain values, stresses in steel are taken equal to the yield stress f_y irrespective of the strain magnitude. However, the inelastic performance of concrete under load will result in a parabolic stress distribution. The diagram of the area of stress as well as the line of action of the resulting force have to be obtained in order to compute the stresses acting on the section.

The section reaches its flexural strength (nominal strength) when the extreme compressive fiber of concrete reaches its maximum usable strain. The ACI code recommends a maximum usable strain value of 0.003. If the properties of the compressive stress block just prior to failure are defined, the flexural strength can be computed [Fig. 4A.1(*e*)]. The ACI code allows the use of a simplified equivalent rectangular stress block to represent the stress distribution of the concrete. The rectangular block has a mean stress of $0.85f_c'$ and a depth a, where $a/c = \beta_1 = 0.85$ for $f_c' \leq 4000$ psi. β_1 is reduced incrementally by 0.05 for each 1000 psi of strength in excess of 4000 psi (27.6 MPa), provided that it does not go below 0.65. The reduction of β_1 is mainly due to less favorable properties of the stress-strain relationship for higher-strength concrete.

When the ultimate flexural capacity is reached, two types of failure can occur, depending on the amount of steel reinforcement. If a relatively low percentage of steel is used, the strain on the tension face will be beyond the yield strain ε_y. This triggers excessive deflections and wide cracks, providing a warning of imminent failure. This type of ductile failure is a desirable mode for flexural members. On the other hand, if a relatively high percentage of steel reinforcement is incorporated in the section, failure will occur prior to yielding of the reinforcement. Hence violent concrete crushing occurs in a sudden brittle manner. Because of the nonductile behavior of this mode of failure, it should be avoided. When the extreme compressive fiber of concrete reaches its maximum usable strain simultaneously with the steel reaching its yielding strain, the section is defined as a balanced section. The balanced section is used as a datum to identify ductile and nonductile reinforced concrete sections.

4A.2.4 Rectangular Sections with Tension Reinforcement Only

4A.2.4.1 Moment Capacity

From Fig. 4A.1(*e*), equating the horizontal forces C and T and solving for the depth of the compression block a, we obtain

$$0.85f_c'ba = A_s f_s \tag{4A.6}$$

$$a = \beta_1 c = \frac{A_s f_s}{0.85 f_c' b} \tag{4A.7}$$

Hence the nominal flexural strength is

$$M_n = A_s f_s \left(d - \frac{a}{2}\right) \tag{4A.8}$$

4A.2.4.2 Balanced Condition

From Fig. 4A.l(*e*), the linear strain distribution gives

$$\frac{c}{d} = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_y} = \frac{0.003}{0.003 + f_y/29 \times 10^6} = \frac{87{,}000}{87{,}000 + f_y} \tag{4A.9}$$

From Eqs. (4A.7) and (4A.9), we can write

$$c = \frac{a}{\beta_1} = \left(\frac{87{,}000}{87{,}000 + f_y}\right) d = \frac{A_s f_y}{0.85 f_c' b \beta_1}$$

Defining $\rho_b = A_s/bd$ we obtain

$$\rho_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\left(\frac{0.85 f_c'}{f_y}\right)\beta_1 \tag{4A.10}$$

Equation (4A.10) gives the balanced section reinforcement ratio. To ensure ductile failure, the ACI code limits the maximum tension reinforcement ratio to 75% of ρ_b. Reinforcement ratios higher than ρ_b produce nonductile failure, with steel reinforcement not yielding prior to the crushing of concrete.

4A.2.4.3 Ductile Tension Failure

For this condition Eqs. (4A.7) and (4A.8) can be written as follows:

$$a = \frac{A_s f_y}{0.85 f_c' ba} \tag{4A.1l}$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) \tag{4A.12}$$

4A.2.4.4 Nonductile Failure

The nominal flexural strength is obtained from

$$M_n = A_s f_s \left(d - \frac{a}{2}\right) \tag{4A.13}$$

where f_s is obtained from the following quadratic equation:

$$A_s(f_s)^2 + (87{,}000A_s)f_s - (0.85f'_c b) \times (87{,}000\beta_1 d) = 0 \tag{4A.14}$$

$$a = \frac{A_s f_s}{0.85f'_c b} \tag{4A.15}$$

4A.2.4.5 Minimum Percentage of Steel

If the nominal flexural strength of the section is less than its cracking moment, the section will fail immediately when a crack occurs. This type of failure occurs in very lightly reinforced beams without warning.

In order to avoid such a failure, the ACI code requires a minimum steel percentage ρ_{min} equal to

$$\rho_{min} = \frac{200}{f_y} \tag{4A.16}$$

Example 4A.2 Determine the nominal flexural strength M_n, of the rectangular section shown in Fig. X.2.

Given:

$f'_c = 4000$ psi (27.6 Mpa)
$f_y = 60{,}000$ psi (413.4 MPa)
$b = 14.0$ in (356 mm)
$h = 24.0$ in (610 mm)
$d = 21.5$ in (546 mm)
$A_s = 4$ no. 10 bars [5.08 in^2 (3277 mm^2)]

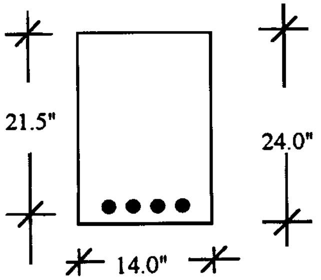


FIGURE X.2

Solution

$$\rho_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\left(\frac{0.85f'_c}{f_y}\right)\beta_1 = (87{,}000\backslash 87{,}000 + 60{,}000)(0.85 \times 4000\backslash 60{,}000)0.85 = 0.0285$$

$$\rho_{max} = 0.75\rho_b = 0.75 \times 0.0285 = 0.0213$$

$$\rho = \frac{A_s}{bd} = \frac{5.08}{14 \times 21.5} = 0.0169 < 0.0213 \qquad \therefore \text{Ductile failure.}$$

$$a = \frac{A_s f_y}{0.85f'_c b} = \frac{5.08 \times 60{,}000}{0.85 \times 4000 \times 14} = 6.4 \text{ in (16.3 mm)}$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right)$$

$$= 5.08 \times 60{,}000\left(21.5 - \frac{6.4}{2}\right) = 5.58 \times 10^6 \text{in} \cdot \text{lb (630.63 kN} \cdot \text{m)}$$

Example 4A.3 Determine the flexural strength of the rectangular section shown in Fig. X.3.

Given:

$f'_c = 5000$ psi (34.45 MPa)

$f_y = 60{,}000$ psi (413.4 MPa)

$b = 14.0$ in (356 mm)

$d = 20.0$ in (508 mm)

$h = 24.0$ in (610 mm)

$A_s = 8$ no. 10 bars [10.16 in^2 (6553.2 mm^2)]

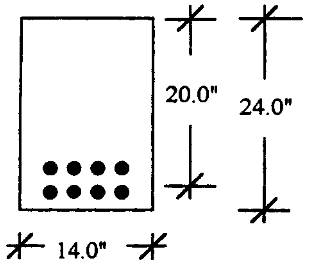


FIGURE X.3

Solution

$$\rho_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\left(\frac{0.85 f'_c}{f_y}\right)\beta_1 = \left(\frac{87{,}000}{87{,}000 + 60{,}000}\right)\left(\frac{0.85 \times 5000}{60{,}000}\right)0.8 = 0.0335$$

$$\rho = \frac{A_s}{bd} = \frac{10.16}{14 \times 20.0} = 0.036 > \rho_b \qquad \therefore \text{ Nonductile failure}$$

(The section does not satisfy ACI code requirements for ductility.)

Hence $f_s < f_y$. To compute f_s,

$$(A_s)f_s^2 + (87{,}000 A_s) f_s - (0.85 f'_c b \times 87{,}000 \beta_1 d) = 0$$

$$10.16 f_s^2 + (87{,}000 \times 10.16) f_s - (0.85 \times 5000 \times 14 \times 87{,}000 \times 0.8 \times 20) = 0$$

$$10.16 f_s^2 + 883{,}920 f_s - 8.28 \times 10^{10} = 0$$

Hence

$$f_s = \frac{-883{,}920 \pm \sqrt{883{,}920^2 + 4 \times 10.16 \times 8.28 \times 10^{10}}}{2 \times 10.16}$$

$$= 56{,}720 \text{ psi (390 MPa)} < 60{,}000 \text{ psi (413.4 MPa)}$$

$$a = \frac{A_s f_s}{0.85 f'_c b} = \frac{10.16 \times 56{,}720}{0.85 \times 5000 \times 14} = 9.68 \text{ in (24.59 mm)}$$

$$M_n = A_s f_s \left(d - \frac{a}{2}\right)$$

$$= 10.16 \times 56{,}720 \left(20 - \frac{9.68}{2}\right) = 8.73 \times 10^6 \text{ in} \cdot \text{lb (986.63 kN} \cdot \text{m)}$$

4A.2.5 Doubly Reinforced Sections

4A.2.5.1 Moment Capacity

Doubly reinforced sections contain reinforcement on the tension side as well as on the compression side of the cross section. These sections become necessary when the size of the rectangular section is restricted due to architectural or mechanical limitations such that the required moment is larger than the resisting design moment of singly reinforced sections.

The analysis of a doubly reinforced section is carried out by theoretically dividing the cross section into two parts, as shown in Fig. 4A.2. Beam 1 is comprised of compression reinforcement at the top and sufficient steel at the bottom to have $T_1 = C_1$. Beam 2 consists of the concrete web and the remaining tensile reinforcement.

The nominal strength of part 1 can be obtained by taking the moment about the tension steel,

$$M_{n1} = A_{s1} f_y(d - d')A'_s f_y(d - d') \tag{4A.17}$$

The nominal strength of part 2 is obtained by taking the moment about the compression force,

$$M_{n2} = (A_s - A_{s1})f_y\left(d - \frac{a}{2}\right) = (A_s - A'_s)f_y\left(d - \frac{a}{2}\right) \tag{4A.18}$$

where
$$a = \frac{(A_s - A_s 1)f_y}{0.85 f'_c b} \tag{4A.19}$$

Hence the nominal strength is

$$M_n = M_{n1} + M_{n2}$$

or
$$M_n = A'_s f_y(d - d') + (A_s - A'_s)f_y\left(d - \frac{a}{2}\right) \tag{4A.20}$$

This equation is only valid when A'_s reaches the yield stress prior to concrete crushing. This condition is satisfied if

$$\rho - \rho' \geq \left(\frac{0.85\beta_1 f'_c d'}{f_y d}\right)\left(\frac{87{,}00}{87{,}000 - f_y}\right) \tag{4A.21}$$

Otherwise the nominal strength equation is written as

$$M_n = A'_s f'_s(d - d') + (A_s f_y - A'_s f'_s)\left(d - \frac{a}{2}\right) \tag{4A.22}$$

where
$$a = \frac{A_s f_y - A'_s f'_s}{f'_c b} \tag{4A.23}$$

and $f'_s < f_y$.

The following iterative procedure can be followed to obtain f'_s:

1. For the first trial assume

$$f'_s = 87{,}000\left[1 - \frac{0.85\beta_1 f'_c}{(\rho - \rho')f_y}\frac{d'}{d}\right]$$

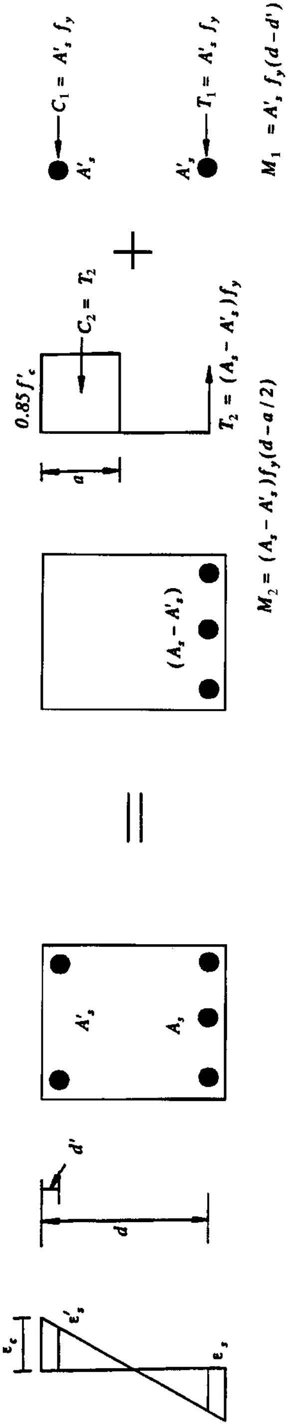


FIGURE 4A.2 Flexural behavior of a doubly reinforced concrete section.

2. Obtain the depth of the compression block,

$$a = \frac{A_s f_y - A_s' f_s'}{0.85 f_c' b}$$

3. Calculate the depth of the neutral axis, $c = a/\beta_1$.
4. Using similar triangles as in Fig. 4A.2, compute the strain ε_s' and then the adjusted compression stress of the steel, $f_s' = \varepsilon_s' E_s$.
5. Repeat steps 2 to 4 until an acceptable convergence is reached. Usually one trial will give acceptable results.

4A.2.5.2 Ductile Failure

To ensure ductile failure, the tension steel reinforcement ρ should be limited to

$$\rho \le 0.75\bar{\rho}_b + \rho_s' \frac{f_s'}{f_y} \tag{4A.24}$$

where $\bar{\rho}_b$ is the balanced steel ratio for a singly reinforced beam with a tension steel area of $A_{s1} = A_s - A'_s$.

Example 4A.4 Determine the nominal strength of the rectangular section shown in Fig. X.4.

Given:

$f_c' = 5000$ psi (34.45 MPa)
$f_y = 60{,}000$ psi (413.4 MPa)
$d = 26.0$ in (660 mm)
$b = 14.0$ in (356 mm)
$h = 30.0$ in (762 mm)
$d = 26.0$ in (660 mm)
$d' = 2.0$ in (51 mm)
$A_s = 8$ no. 9 bars [8 in^2 (5160 mm^2)]
$A_s' = 2$ no. 8 bars [1.58 in^2 (1019 mm^2)]

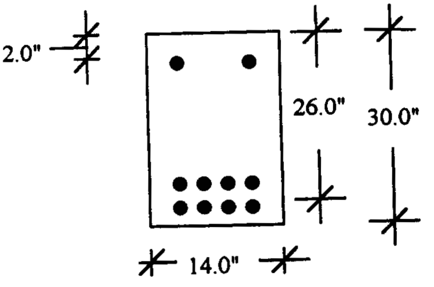


FIGURE X.4

Solution

$$\rho = \frac{A_s}{bd} = \frac{8.0}{14 \times 26.0} = 0.022$$

$$\rho' = \frac{A_s'}{bd} = \frac{1.58}{14 \times 26.0} = 0.0043$$

$$\bar{\rho}_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\left(\frac{f_c'}{f_y}\right)\beta_1 = \left(\frac{87{,}000}{87{,}000 + 60{,}000}\right)\left(\frac{0.85 \times 5000}{60{,}000}\right)0.8 = 0.0336$$

Check for yielding of steel in compression:

$$\rho - \rho_b = 0.022 - 0.0043 = 0.0177$$

$$\left(\frac{0.85\beta_1 f'_c d'}{f_y d}\right)\left(\frac{87{,}000}{87{,}000 - f_y}\right) = \left(\frac{0.85 \times 0.8 \times 5000 \times 2}{60{,}000 \times 26}\right)\left(\frac{87{,}000}{87{,}000 - 60{,}000}\right) = 0.014$$

$$\rho - \rho_b = 0.0177 > 0.0\ 14 \qquad \therefore \text{ Steel is yielding in compression.}$$

$$f'_s = f_y = 60{,}000 \text{ psi (413.4 MPa)}$$

Check for yielding of steel in tension:

$$0.75\bar{\rho}_b + \rho' \frac{f'_s}{f_y} = 0.75 \times 0.0336 + 0.0043 = 0.0295$$

$$\rho = 0.022 < 0.0295 \qquad \therefore \text{ Steel is yielding in tension.}$$

$$a = \frac{(A_s - A'_s) f_y}{0.85 f'_c b} = \frac{(8 - 1.58)60{,}000}{0.85 \times 5000 \times 14} = 6.47 \text{ in (16.4 mm)}$$

$$M_{n1} = A'_s f_y (d - d') = 1.58 \times 60{,}000(26 - 2) = 2.27 \times 10^6 \text{ in} \cdot \text{lb (256.5 kN} \cdot \text{m)}$$

$$M_{n2} = (A_s - A'_s) f_y \left(d - \frac{a}{2}\right)$$

$$= (8 - 1.58) \times 60{,}000\left(26 - \frac{6.47}{2}\right) = 8.76 \times 10^6 \text{ in} \cdot \text{lb (990.02 kN} \cdot \text{m)}$$

$$M_n = (2.27 + 8.76)10^6 = 11 \times 10^{\ 6} \text{ in} \cdot \text{lb (1246.52 kN} \cdot \text{m)}$$

Example 4A.5 Determine the nominal strength M5 of the rectangular section shown in Fig. X.5.

Given:

f'_c = 5000 psi (34.45 MPa)

f_y = 60,000 psi (413.4 MPa)

b = 14.0 in (356 mm)

h = 24.0 in (610 mm)

d = 21.0 in (S33 mm)

d' = 2.0 in (S1 mm)

A_s = 4 no. 10 bars [S08 in^2 (3277 mm^2]

A'_s = 3 no.7 bars [1.8 in^2 (1161 mm^2]

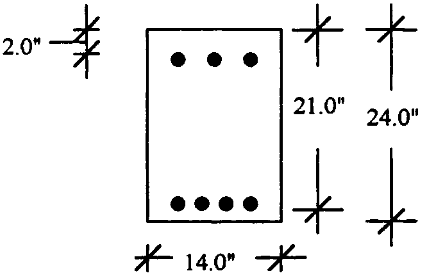


FIGURE X.5

Solution

$$\rho = \frac{A_s}{bd} = \frac{5.08}{14 \times 21.0} = 0.0173$$

$$\rho' = \frac{A'_s}{bd} = \frac{1.8}{14 \times 21.0} = 0.0061$$

$$\bar{\rho}_b = 0.0336$$

Check for yielding of steel in compression:

$$\rho - \rho' = 0.0173 - 0.0061 = 0.0112 < 0.0173 \qquad \therefore \text{ Steel did not yield in compression.}$$

Assume

$$f_s' = 87{,}000\left[1 - \frac{0.85\beta_1 f_c' d'}{(\rho - \rho')f_y d}\right]$$

$$= 87{,}000\left[1 - \frac{0.85 \times 0.8 \times 5000 \times 2}{0.0112 \times 60{,}000 \times 21}\right] = 45{,}000 \text{ psi (310 MPa)}$$

Start the iteration cycle,

$$a = \frac{A_s f_y - A_s' f_s'}{0.85 f_c' b} = \frac{5.08 \times 60{,}000 - 1.8 \times 45{,}000}{0.85 \times 5000 \times 14} = 3.76 \text{ in (95.5 mm)}$$

$$(f_s') \text{ adjusted} = 29 \times 10^6 \times 0.003\left(\frac{3.76/0.8 - 2}{3.76/0.8}\right) = 50{,}000 \text{ psi (344.5 Mpa)}$$

Take $f_s' = 50{,}000$ psi,

$$a = \frac{A_s f_y - A_s' f_s'}{0.85 f_c' b} = \frac{5.08 \times 60{,}000 - 1.8 \times 50{,}000}{0.85 \times 5000 \times 14} = 3.6 \text{ in (91.4 mm)}$$

$$M_n = (A_s f_y - A_s' f_s')\left(d - \frac{a}{2}\right) + A_s' f_s'(d - d')$$

$$= (5.08 \times 60{,}000 - 1.8 \times 50{,}000)\left(21 - \frac{3.6}{2}\right) + 1.8 \times 50{,}000(21 - 2)$$

$$= 5.83 \times 10^6 \text{ in} \cdot \text{lb}(658.88 \text{ kN} \cdot \text{m})$$

4A.2.6 Flanged Section

4A.2.6.1 Moment Capacity

Because rectangular beams are generally cast monolithically with concrete slabs, a full composite action between the slab and the beam is obtained. In the positive moment diagram, the slab is in compression and, hence, contributes to the moment strength of the section. The effective cross section of the beam has a T shape or an L shape, consisting of the rectangular beam as the web and a portion of the slab as the flange (Fig. 4A.3). The effective width of the slab contributing to the section strength has to satisfy the following requirements.

For the T shape (interior beam),

$$b \le 16h_f + b_w$$
$$\le b_w + l_c$$
$$\le 1_n/4 \qquad (4A.25)$$

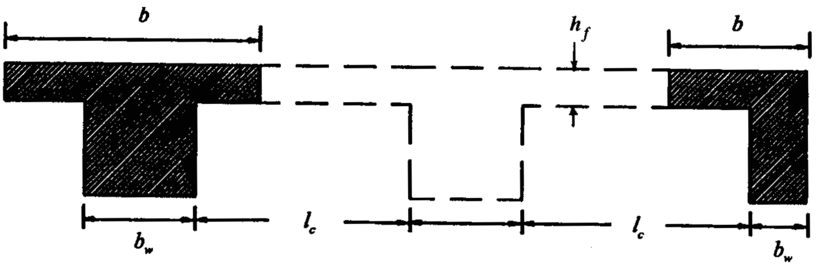


FIGURE 4A.3 Effective cross section of a flanged concrete beam. l_n = beam span length.

For the L shape (edge beam),

$$\begin{aligned} b &\le 6h_f + b_w \\ &\le b_w + l_c/2 \\ &\le 1_n/12 \end{aligned} \tag{4A.26}$$

Flanged beams possess a high compression capacity because of the large contribution of the concrete on the compression face. Hence the neutral axis generally lies in the flange. When this situation occurs, the section behaves as a rectangular singly reinforced section having a width b equal to the effective width of the slab. The flexural strength of this section is

$$M_n = A_s f_y \left(\frac{d - a}{2} \right) \tag{4A.27}$$

where $$a = \frac{A_s f_y}{0.85 f'_c b}$$

The neutral axis will fall below the flange if the tension force $A_s f_y$ is greater than the compression force capacity of the flange $0.85 f'_c b h_f$,

$$A_s f_y > 0.85 f'_c b h_f$$

Hence

$$a = \frac{A_s f_y}{0.85 f'_c b} > h_f \tag{4A.28}$$

In this case the analysis can be conducted by considering the resistance provided by the overhanging flanges and that provided by the remaining rectangular beam, as shown in Fig. 4A.4. Beam 1 consists of the overhanging flange area A_f, stressed to $0.85 f'_c$, giving a compressive force C_f, which acts at the centroid of the area of the overhanging flanges. To maintain equilibrium, beam 1 has a tensile steel area A_{sf} chosen such that

$$A_{sf} f_y = A_f (0.85 f'_c)$$

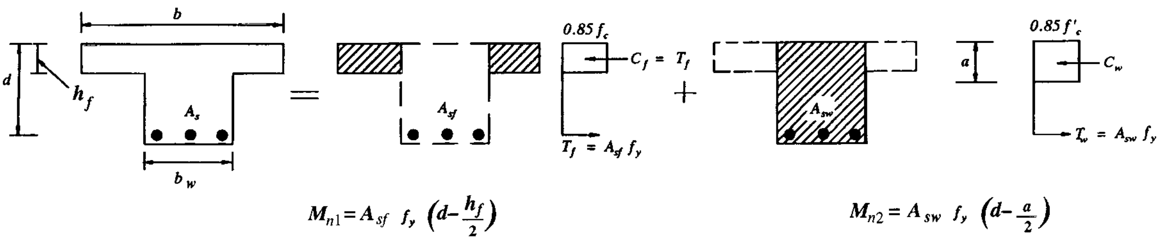


FIGURE 4A.4 Flexural behavior of concrete beam.

This steel area A_{sf} is a portion of the total area A_f and is assumed to be at the same centroid. The moment capacity of beam 1 is obtained by taking the moment about the tensile steel area A_{sf},

$$M_{n1} = 0.85f'_c(b - b_w)h_f\left(d - \frac{h_f}{2}\right) \tag{4A.29}$$

Beam 2 is a rectangular beam having a width b_w. The compressive force of the beam, $C_2 = 0.85f'_c b_w a$, acts through the centroid of the compression area. Equilibrium is maintained by utilizing the remaining tensile steel area $A_s - A_{sf} = A_{sw}$. The moment capacity is obtained by taking the moment about the compression force C_2,

$$M_{n2} = A_{sw} f_y\left(d - \frac{a}{2}\right) \tag{4A.30}$$

where
$$A_{sw} = A_s - A_{sf} = A_s - \frac{0.85f'_c(b - b_w)h_f}{f_y} \tag{4A.31}$$

and
$$a = \frac{(A_s - A_{sf})f_y}{0.85f'_c b_w} \tag{4A.32}$$

Hence the total nominal strength is

$$M_n = M_{n1} + M_{n2} = 0.85f'_c(b - b_w)h_f\left(d - \frac{h_f}{2}\right) + A_{sw}f_y\left(d - \frac{a}{2}\right) \tag{4A.33}$$

4A.2.6.2 Ductile Failure

To ensure ductile failure, the tension steel reinforcement ratio ρ should be limited to

$$\rho \le 0.75\frac{b_w}{b}(\bar{\rho}_b + \rho_f) \tag{4A.34}$$

where $\rho = A_s/bd$

ρ_b = balanced steel ratio for a rectangular section (b_w and h) with tension reinforcement, $A_{sw} = A_s - A_{sf}$

$$\rho_f = \frac{0.85f'_c(b - b_w)h_f}{f_y b_w d} = \frac{A_{sf}}{b_w d}$$

Example 4A.6 Determine the nominal flexural strength M_n, of the precast T beam shown in Fig. X.6.

Given:

$f'_c = 5000$ psi (34.45 MPa)

$f_y = 60{,}000$ psi (413.4 MPa)

$b = 36.0$ in(914 mm)

$bw = 12.0$ in (305 mm)

$h = 20.0$ in (508 mm)

$h_f = 2.0$ in (51 mm)

$d = 17.0$ in (432 mm)

$A_s = 6$ no. 9 bars [6 in^2 (3870 mm^2)]

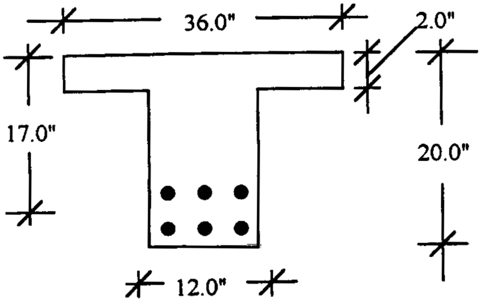


FIGURE X.6

Solution Check whether the tension force is greater than the compressive force,

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6 \times 60{,}000}{0.85 \times 5000 \times 36} = 2.35 \text{ in (59.7 mm)} > h_f = 2 \text{ in (51 mm)}$$

Hence the section will act as a T beam.

$$A_{sf} = \frac{0.85 f'_c (b - b_w) h_f}{f_y} = \frac{0.85 \times 5000(36 - 12)2}{60{,}000} = 3.4 \text{ in}^2 \text{ (2193 mm}^2\text{)}$$

$$a = \frac{(A_s - A_{sf}) f_y}{0.85 f'_c b_w} = \frac{(6 - 3.4)600{,}000}{0.85 \times 5000 \times 12} = 3.06 \text{ in (77.7 mm)}$$

$$M_{n1} = 0.85 f'_c (b - b_w) h_f \left(d - \frac{h_f}{2} \right)$$

$$= 0.85 \times 5000(36 - 12)2 \left(17 - \frac{2}{2} \right) = 3.26 \times 10^6 \text{ in} \cdot \text{lb (368.43 kN} \cdot \text{m)}$$

$$M_{n2} = A_{sw} f_y \left(d - \frac{a}{2} \right)$$

$$= (6 - 3.4)60{,}000 \left(17 - \frac{3.06}{2} \right) = 2.4 \times 10^6 \text{ in} \cdot \text{lb (271.24 kN} \cdot \text{m)}$$

Hence the total nominal strength is

$$M_n = M_{n1} + M_{n2} = (3.26 + 2.41)10^6 = 5.67 \times 10^6 \text{ in} \cdot \text{lb (639.67 kN} \cdot \text{m)}$$

4A.3 SHEAR BEHAVIOR

4A.3.1 Plain Concrete

Because structural members are usually subjected to shear stresses combined with axial, flexure, and tension forces rather than to pure shear stresses, the behavior of concrete under pure shear forces is not of major importance. Furthermore, even if pure shear is encountered in a member, a principal stress of equal magnitude will be produced on another inclined plane, leading to the failure of concrete in tension before its shearing strength can be reached.

Consider a small element A at the neutral axis of the beam presented in Fig. 4A.5(*a*). It can be shown that the case of pure shear [Fig. 4A.5(*b*)] is equivalent to a set of normal tension and compression stresses σ_1 and σ_2, respectively. Cracking of concrete will occur if the tension stress referred to as diagonal tension exceeds the tension strength of the concrete. As stated earlier, shear stresses are usually combined with other stresses. Considering a small element located below the neutral axis of the beam in Fig. 4A.5(*a*), two types of stresses occur, bending stresses and shear stresses. If the beam is behaving in the elastic range, these stresses can be obtained as follows:

$$\sigma = \frac{Mc}{I} \tag{4A.35}$$

$$\tau = \frac{VQ}{Ib} \tag{4A.36}$$

This element can be rotated at an angle ϕ to obtain the principal normal stresses σ_1 and σ_2. The magnitude of the principal stresses and their orientations [Fig. 4A.5(*c*)] are determined from the following expressions:

$$\sigma_1 = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{4A.37}$$

$$\sigma_2 = \frac{\sigma}{2} - \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{4A.38}$$

$$\tan 2\theta = \frac{\tau}{\sigma/2} \tag{4A.39}$$

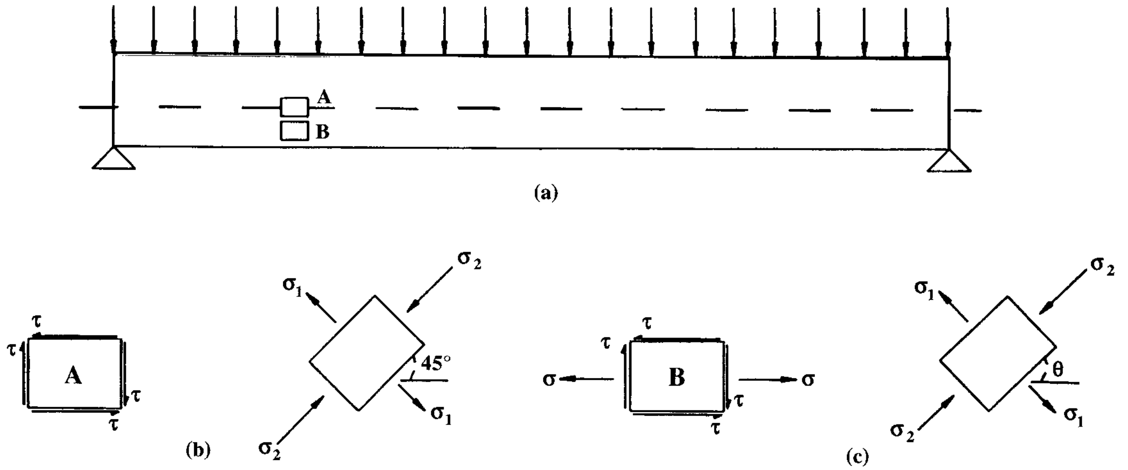


FIGURE 4A.5 (a) Shear behavior of concrete beam. (b) Pure shear. (c) Combined shear and bending.

It should be clear at this stage that, depending on the relative values of the bending moments and shear forces, the magnitudes of the principal stresses and their orientations will vary. When bending moments are relatively predominant compared to shearing forces, flexure cracks are observed. However, if the shear stresses are sufficiently higher than the bending stresses, inclined cracks of the web propagating from the neutral axis are to be expected.

4A.3.2 Reinforced Concrete Beams

The shear strength of concrete beams reinforced with steel bars resisting flexural loading is denoted by V_c. The failure plane shown in Fig. 4A.6 indicates that V_c is mainly provided by three sources—shear resistance of the uncracked concrete V_{uc} resistance by aggregate interlock V_a and dowel action provided by the longitudinal flexural steel reinforcements V_d. These three terms cannot be determined individually. Their combined total effect is evaluated empirically based on a large number of available test results. The ACT code suggests the following conservative equation to determine V_c:

$$V_c = 2\sqrt{f'_c}b_w d \tag{4A.40}$$

A less conservative expression, which takes into account the effects of the longitudinal reinforcement and the moment-to-shear ratio, may also be used:

$$V_c = \left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right)b_w d \le 3.5\sqrt{f'_c}b_w d \tag{4A.41}$$

In this expression $V_u d/M_u$ may not be taken as greater than 1.0. In spite of the fact that Eq. (4A.41) is less conservative, its complex form makes its use justifiable in cases of large numbers of similar members. If the shear strength is to be determined for lightweight concrete members, the term $\sqrt{f'_c}$ should be replaced with $f_{ct}/6.7 \le f'_c$, where f_{ct} is the split cylinder strength of concrete. If the f_{ct} value is not available, then the term $\sqrt{f'_c}$ is to be multiplied by 0.75 for all lightweight concrete and by 0.85 for sand lightweight concrete.

When axial compression exists, the ACT code permits the use of the following equation:

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}b_w d \tag{4A.42}$$

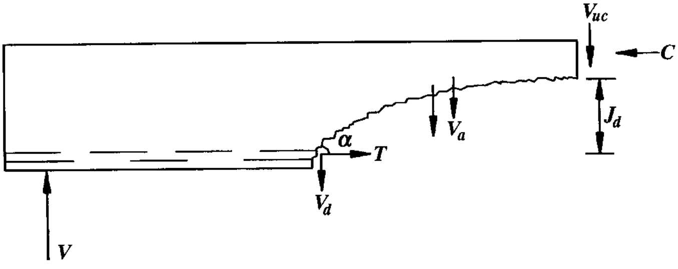


FIGURE 4A.6 Shear-resisting mechanism in reinforced concrete beams.

A more conservative equation is adopted by the ACI code for the case of axial tension,

$$V_c = 2\left(1 + \frac{N_u}{500A_g}\right)\sqrt{f'_c}\,b_w d \tag{4A.43}$$

where N_u is negative for tension and the ratio N_u/A_g is expressed in pounds per square inch.

When the factored shear force V_u is relatively higher compared to V_c, an additional web reinforcement is provided, as shown in Fig. 4A.7. Several theories have been presented to explain the behavior of web reinforcement. The truss analogy theory is being used widely to illustrate the contribution of web reinforcement to the shear strength of the reinforced concrete beams. According to this theory, the behavior of a reinforced concrete beam with shear reinforcement is analogous to that of a statically determinate paralleled chord truss with pinned joints.

If it is conservatively assumed that the horizontal projection of the crack is equal to the effective depth of the section d, it can be shown that the shear contribution of the vertical stirrups is

$$V_s = \frac{A_v f_y d}{s} \tag{4A.44}$$

If inclined stirrups are used, their contribution to the section shear strength is

$$V_s = \frac{A_v f_y d}{s}(\sin\alpha + \cos\alpha) \tag{4A.45}$$

where α is the angle between the stirrups and the longitudinal axis of the member.

The total nominal shear strength of a section is therefore

$$V_n = V_c + V_s \tag{4A.46}$$

The ACI code limits the maximum vertical spacing to $d/2 \le 24$ in (305 mm). If the shear resistance V_s of the web reinforcement exceeds $4\sqrt{f'_c}\,b_w d$, the maximum spacing limit is reduced by one-half to $d/4 \le 12$ in (305 mm). A minimum practical spacing that could be adopted is approximately 3 to 4 in (75 to 100 mm).

The code also provides maximum and minimum limits for the area of shear reinforcement. To avoid concrete crushing prior to the yielding of shear reinforcement, a maximum limit is set. This is provided by limiting the contribution of V_s to the shear resistance to

$$V_s \le 8\sqrt{f'_c}\,b_w d \tag{4A.47}$$

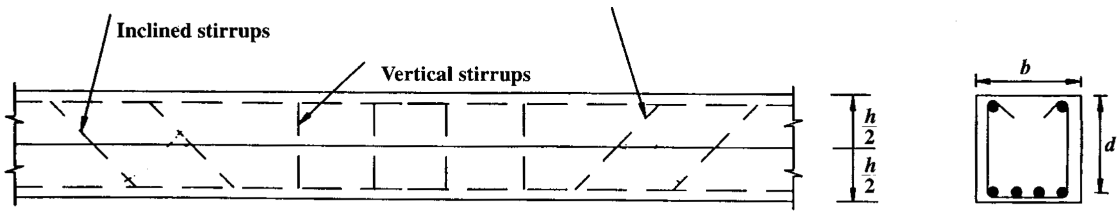


FIGURE 4A.7 Web reinforcement for shear resistance.

If a safe design calls for a higher V_s contribution than the limit set in Eq. (4A.47), the concrete section needs to be enlarged to increase the contribution of concrete to the shear resistance V_c.

The ACI code also requires a minimum shear reinforcement $(A_v)_{\min}$ if V_u exceeds $\phi V_c/2$ such that

$$(A_v)_{\min} = \frac{50 b_w s}{f_y} \tag{4A.48}$$

This requirement is necessary to avoid possible brittle failure after the formation of early-stage diagonal cracking. The yield strength of stirrups is limited by the code to 60,000 psi (413.4 MPa) to control the crack width and provide for aggregate interlock availability.

Example 4A.7 The rectangular cross section shown in Fig. X.7 is subjected to the following factored loads:

$$V_u = 70{,}000 \text{ lb } (311.5 \text{ kN})$$

$$M_u = 1 \times 10^6 \text{ in} \cdot \text{lb} (113 \text{ kN} \cdot \text{m})$$

Given:

$f_c' = 4000$ psi (27.S6 MPa)

f_y(stirrups) = 60,000 psi (413.4 MPa)

$b = b_w = 12.0$ in (305 mm)

$h = 28.0$ in (711 mm)

$d = 25.0$ in (635 mm)

$A_s = 3$ no. 9 bars (3 in^2 (1935 mm^2)]

Determine the required shear reinforcement.

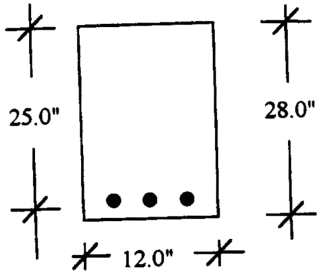


FIGURE X.7

Solution Determine the shear resistance V_c provided by concrete:

1. Using the simplified expression in Eq. (4A.40),

$$V_c = 2 f_c' b_w d = 2\sqrt{4000} \times 12 \times 25 = 37{,}947 \text{ lb } (168.9 \text{ kN})$$

2. Using the detailed expression in Eq. (4A.41),

$$V_c = \left(1.9 f_c' + 2500 \rho_w \frac{V_d d}{M_u}\right) b_w d \leq 3.5 f_c' b_w d$$

where
$$\rho_w = \frac{A_s}{b_w d} = \frac{3}{12 \times 25} = 0.01$$

$$\frac{V_d d}{M_u} = \frac{70{,}000 \times 25}{10^6} = 1.75 > 1.0 \qquad \therefore \text{ Use } 1.0.$$

$$V_c = (1.9\sqrt{4000} + 2500 \times 0.01 \times 1)12 \times 25 \leq 3.5\sqrt{4000} \times 12 \times 25$$

$$= 43{,}550 \text{ lb } (193.8 \text{ kN}) \leq 66{,}407 \text{ lb } (295.5 \text{ kN})$$

Thus V_c = 43,550 lb (193.8 kN), compared to 37,947 lb (168.9 kN) obtained using the simplified expression. Use V_c = 43,550 lb (193.8 kN) for the rest of the example.

$$V_s = V_n - V_c = \frac{V_u}{\phi} - V_c = \frac{70{,}000}{0.85} - 43550 = 38{,}803 \text{ 1b (172.7kN)}$$

$$V_s = 38{,}803 \text{ lb (172.7 kN)} \le 8\sqrt{f'_c}b_w d = 151{,}788 \text{ lb (675.5 kN)} \qquad \therefore \text{No need to enlarge section.}$$

$$V_s = 38{,}803 \text{ lb (172.7 kN)} \le 4\sqrt{f'_c}b_w d = 75{,}894 \text{ lb (337.7 kN)}$$

The maximum stirrup spacing is

$$s = \frac{d}{2} = \frac{25}{2} = 125 \text{ in (318 mm)} \le 24 \text{ in (610 mm)}$$

Hence a maximum spacing of 12.S in (318 mm) should be used.
Use a U-shape no. 3 bars for stirrups. Hence $A_v = 0.11 \times 2 = 0.22$ in^2 (142 mm^2). The required spacing is

$$s = \frac{A_v f_y d}{V_s} = \frac{0.22 \times 60{,}000 \times 25}{38{,}803} = 8.5 \text{ in (216 mm)} < 12.\text{S in (318 mm)} \qquad \text{O.K.}$$

Check the minimum reinforcement:

$$(A_v)_{\min} = \frac{50 b_w s}{f_y} = \frac{50 \times 12 \times 8.5}{60{,}000} = 0.085 \text{ in}^2 \text{ (55 mm}^2) < 0.22 \text{ in}^2 \text{ (142 mm}^2) \qquad \text{O.K.}$$

Therefore use U-shape no. 3 bars with spacing s = 8.0 in (200 mm).

Example 4A.8 Provide shear reinforcement for the beam shown in Fig. X.8.

Given:

$b_w = b$ = 14 in (3S6 mm)

d = 23 in (584 mm)

h = 26 in (660 mm)

f'_c = 5000 psi (34.45 MPa)

f_y = 60,000 psi (413.4 MPa)

w_D = 3000 lb/ft (437.S N/in)

w_L = 5000 lb/ft (729.2 N/in)

Solution Critical section:

$$W_u = 1.4 \times 3000 + 1.7 \times 5000 = 12{,}700 \text{ lb/ft (4710 N/in)}$$

$$V_u \text{ at face of support} = 12{,}700\left(\frac{15}{2}\right) = 95{,}250 \text{ lb (423.9 kN)}$$

$$V_u \text{ at distance d} = 23 \text{ in from face of support } 95{,}250\left(\frac{90-23}{90}\right) = 70{,}908 \text{ lb (315.5 kN)}$$

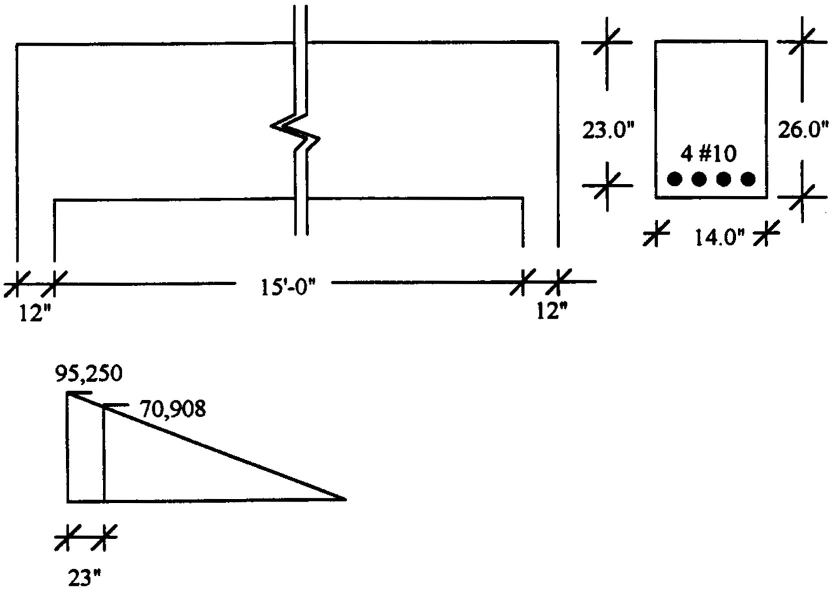


FIGURE X.8

$$V_c \text{ using simplified expression} = 2\sqrt{f'_c}b_w d = 2\sqrt{5000} \times 14 \times 23 = 45{,}S37 \text{ lb (202.6 kN)}$$

$$V_s \text{ required} = \frac{V_u}{\phi} - V_c = \frac{70{,}908}{0.85} - 45{,}537 = 37{,}884 \text{ lb (168.6 kN)}$$

$$V_s = 37{,}884 \leq 8\sqrt{f'_c}b_w d = 182{,}148 \text{ lb (810.6 kN)} \qquad \therefore \text{No need to enlarge section.}$$

$$V_s = 37{,}884 \leq 4\sqrt{f'_c}b_w d = 91{,}075 \text{ lb (405.3 kN)}$$

The maximum stirrup spacing is

$$s = \frac{d}{2} = \frac{23}{2} = 11.5 \text{ in (292 mm)} \leq 24 \text{ in (610 mm)}$$

Hence a maximum spacing of 11.5 in (292 mm) should be used.

Use U-shape no. 3 bars for stirrups, and we get

$$s = \frac{A_s f_y d}{V_s} = \frac{0.22 \times 60{,}000 \times 23}{37{,}884} = 8.01 \text{ in (203 mm)}$$

Therefore uses $s = 8$ in (200 mm) $< s_{max} = 11.5$ in (292 mm). O.K.

Check the minimum reinforcement:

$$(A_v)_{min} = \frac{50 b_w s}{f_y} = \frac{50 \times 14 \times 8}{60{,}000} = 0.093 \text{ in}^2 \text{ (60 mm}^2) < 0.22 \text{ in}^2 \text{ (142 mm}^2) \qquad \text{O.K.}$$

Section at which s_{max} can be used:

$$V_s \text{ with } s = 11.5 \text{ in} = \frac{A_v f_y d}{s} = \frac{0.22 \times 60{,}000 \times 23}{11.5} = 26{,}400 \text{ lb (117.5 kN)}$$

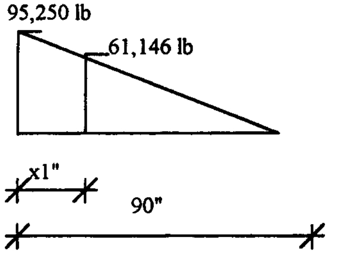
$$V_n = \frac{V_u}{\phi} = V_s + V_c = 26{,}400 + 45{,}537 = 71{,}937 \text{ lb (320.1 kN)}$$

$$V_u = 71{,}937 \times 0.85 = 61{,}146 \text{ lb (272.1 kN)}$$

This value is obtained at distance x_1 from the face of the support, such that

$$90 - x_1 = 90\left(\frac{61{,}146}{95{,}250}\right)$$

$$x_1 = 32 \text{ in (813 mm)}$$



Section at which stirrups can be omitted: In order not to use stirrups, the ACI code requires that

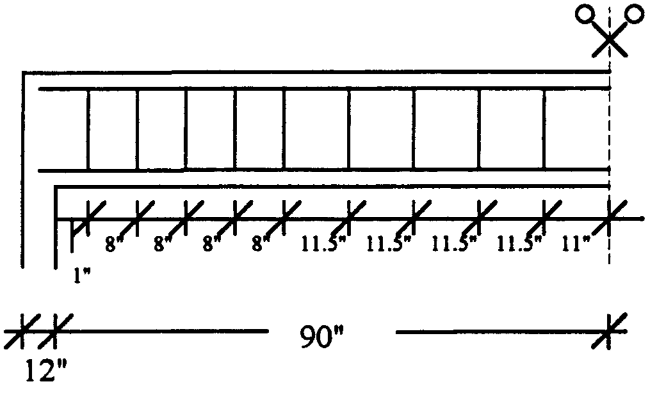
$$V_u \leq \frac{\phi V_c}{2} = \frac{0.85 \times 45{,}537}{2} = 19{,}353 \text{ lb (86.1 kN)}$$

This value is obtained at distance x_2 from the face of the support,

$$90 - x_2 = 90\left(\frac{19{,}353}{95{,}250}\right)$$

$$X_2 = 72 \text{ in (1829 mm)}$$

Shear reinforcement summary: This is illustrated by the following figure.



4A.4 TORSION BEHAVIOR

4A.4.1 Torsion Strength of Reinforced Concrete Beams

Torsion occurs in many structures, such as main girders of bridges, edge beams in buildings, spiral stairways, and balcony girders. Torsion, however, usually occurs in combination with shear and bending.

If a plain concrete member is subjected to pure torsion, it will first crack, then fail along 45° spiral lines. This failure pattern is due to the diagonal tension corresponding to the torsional stresses, which have opposite signs on both sides of the member (Fig. 4A.8).

The maximum torsional stresses v_{max} caused in a rectangular cross section can be calculated theoretically using the following expression:

$$v_{\max} = \alpha_1 \frac{T_{cr}}{x^2 y} \tag{4A.49}$$

where α is a constant varying from 3 to 5, according to the y/x ratio.

Using the lowest value of a (namely, 3 and equating the maximum torsional stress that concrete can resist without cracking to $6\sqrt{f'_c}$,the torsional moment at which diagonal tension cracking occurs can be estimated using the following expression:

$$T_{cr} = 2\sqrt{f'_c}\,x^2 y \tag{4A.50}$$

If the member cross section is T- or L-shaped, the following expression can be used satisfactorily:

$$T_{cr} = 2\sqrt{f'_c}\,\Sigma x^2 y \tag{4A.51}$$

In this case the section is divided into a set of rectangles, each resisting part of the twisting moment in proportion to its torsional rigidity. The ACT code limits the length of the overhanging flange to be considered effective in torsional rigidity computations to three times its thickness.

Due to the combined effect of bending and torsion, a portion of the diagonal cracks will fail in the compression zone on one side of the beam. As a result, even though diagonal torsion cracks have developed on part of the beam, the other part continues to resist some torsion. The ACI conservatively limits the torsional resistance of the cracked section to 40% of the uncracked section. Hence,

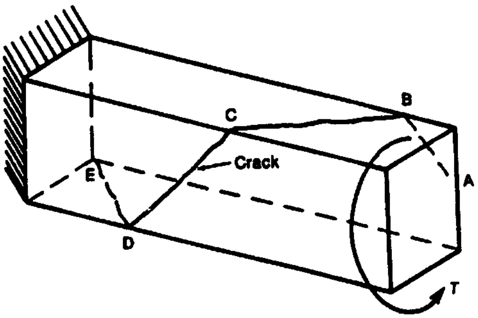


FIGURE 4A.8 Cracking pattern due to torsion.

$$T_c = 2 \times 0.4\sqrt{f'_c}\Sigma x^2 y = 0.8\sqrt{f'_c}\Sigma x^2 y \tag{4A.52}$$

If the factored torsional moment T_u exceeds the resistance of the reinforced concrete section without web reinforcement, additional torsional reinforcement needs to be provided such that

$$T_u \leq \phi(T_c + T_s) \tag{4A.53}$$

The torsion reinforcement is provided using closed vertical stirrups to form a continuous loop and additional longitudinal reinforcing bars, as shown in Fig. 4A.9. The longitudinal bars should be no smaller than no. 3, and they should be 12 in (0.3 m) apart.

According to the ACI code, the total twisting moment T_s resisted by the vertical closed stirrups and the longitudinal reinforcing bars can be estimated using the following expression:

$$T_s = \frac{A_t \alpha_1 x_1 y_1 f_y}{s} \tag{4A.54}$$

where A_t = area of one leg of a closed stirrup spaced at a distance s
x_1 = shorter dimension of stirrup
y_1 = longer dimension of stirrup
α_1 = empirical coefficient; $= 0.66 + 0.33y_1/x_1 \leq 1.5$.

The area of the longitudinal bars required for torsional resistance is obtained as the larger of the following two equations:

$$A_l = 2A_t\left(\frac{x_1 + y_1}{s}\right) \tag{4A.55}$$

or

$$A_l = \left[\frac{400 b_w s}{f_y}\left(\frac{T_u}{T_u + V_u/3C_t}\right) - 2A_t\right]\left(\frac{x_1 + y_1}{s}\right) \tag{4A.56}$$

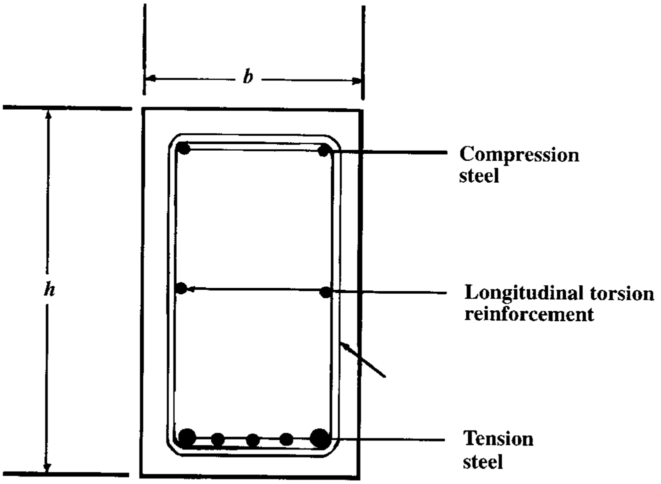


FIGURE 4A.9 Reinforcement for torsion resistance.

where $$C_t = \frac{b_w d}{\Sigma x^2 y}$$

In Eq. (4A.56) the value of A_l need not exceed the value obtained if $50b_w s/f_y$ is substituted in place of $2A_t$.

The spacing of the closed stirrups selected to provide torsion resistance should not be greater than $(x_1 + y_1)/4$ or 12 in (0.3 in). Also the code limits the amount of torsion reinforcement in order to ensure ductility. Hence the torsion resistance obtained for steel reinforcement is limited to

$$T_s \leq 4T_c \tag{4A.57}$$

Furthermore, the steel reinforcement yield stress is limited to 60,000 psi (413.4 MPa).

4A.4.2 Combined Shear, Bending, and Torsion

Due to the combined action of shear, bending, and torsion the following modified expressions are provided in the ACI code to obtain V_c and T_c, respectively:

$$V_c = \frac{2\sqrt{f'_c}\,b_w d}{\sqrt{1 + (2.5C_t T_u/V_u)^2}} \tag{4A.58}$$

$$T_c = \frac{0.8\sqrt{f'_c}\,\Sigma x^2 y}{\sqrt{1 + (0.4V_u/C_t T_u)^2}} \tag{4A.59}$$

Equations (4A.44) and (4A.54) could be used to provide shear and torsion reinforcement, respectively.

For reasons of ductility, the code limits the reinforcement contribution as follows:

$$V_t \leq 8\sqrt{f'_c}\,b_w d \tag{4A.60}$$

$$T_s \leq 4T_c \tag{4A.61}$$

The minimum area of steel reinforcement may not be less than $50b_w s/f_y$. Hence

$$A_v + 2A_t \leq \frac{50b_w s}{f_y} \tag{4A.62}$$

The minimum reinforcement needs to be provided if T_u exceeds $0.5\sqrt{f'_c}\,\Sigma x^2 y$. Otherwise torsional effects can be neglected.

Example 4A.9 A 6-in (152-mm) slab cantilevers 6 ft (1829 mm) from the face of a 14 × 24 in (356 × 610 mm) simple beam, as shown in Fig. X.9. The beam spans 30 ft (9144 mm). It carries a uniform service live load of 25 psf (1.02 kPa) on the cantilever.

Given:

$f'_c = 4000$ psi (27.56 MPa)

$f_y = 60{,}000$ psi (413.4 MPa)

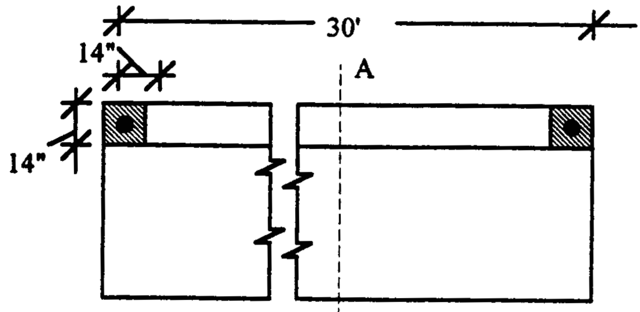
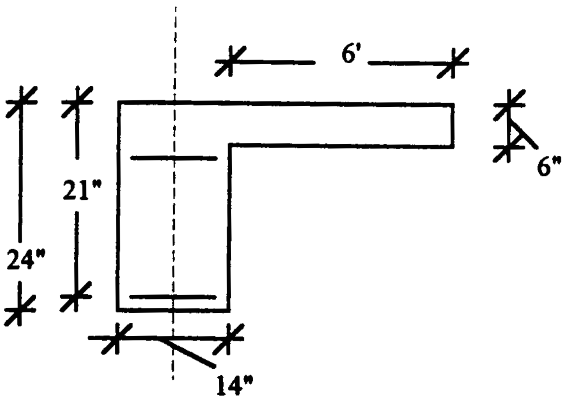


Figure X.9



SECTION A-A

$A_s = 3\ \text{in}^2\ (1935\ \text{mm}^2)$

Column dimensions = 14 × 14 in (356 × 356 mm)

Determine the required shear and torsion reinforcement at the critical section.

Solution

$$w_D = \frac{6}{12} \times 150 = 75\ \text{psf}\ (3.59\ \text{kPa})$$

$$w_u = 1.4 \times 75 + 1.7 \times 25 = 148\ \text{psf}\ (7.08\ \text{kPa})$$

At the centerline of the column,

$$V_u = \frac{148 \times 7.17 \times 30}{2} + 1.4\left[\frac{14(24-6)}{144} \times 150\right]\frac{30}{2} = 21{,}430\ \text{lb}\ (95.36\ \text{kN})$$

$$T_u = \frac{148 \times 30 \times 6}{2}\left(3 + \frac{7}{12}\right) = 47{,}730\ \text{ft} \cdot \text{lb}\ (64\ 72\ \text{kN} \cdot \text{m})$$

At a distance $d = 21$ in (533 mm) from the face of the support (critical section),

$$V_u = 21{,}430\left[\frac{15 - \left(\frac{7+21}{12}\right)}{15}\right] = 18{,}096 \text{ lb } (80.53 \text{ kN})$$

$$T_u = 47{,}730\left[\frac{15 - \left(\frac{7+21}{12}\right)}{15}\right] = 40{,}305 \text{ ft} \cdot \text{lb } (54.65 \text{ kN} \cdot \text{m})$$

$$\Sigma x^2 y = 14^2 \times 24 + 6^2 \times 24 = 5352 \text{ in}^3 \; (8.8 \times 10^7 \text{ mm}^3)$$

$$\phi(0.5\sqrt{f'_c}\Sigma x^2 y) = 0.85(0.5\sqrt{4000} \times 5352) = 143{,}858 \text{ in} \cdot \text{lb } (16.26 \text{ kN} \cdot \text{m})$$

$$< T_u = 483{,}660 \text{ in} \cdot \text{lb } (54.65 \text{ kN} \cdot \text{m}) \qquad \therefore \text{ Torsional effects need to be considered.}$$

$$C_t = \frac{b_w d}{\Sigma x^2 y} = \frac{14 \times 21}{5332} = 0.055$$

$$V_c = \frac{2\sqrt{f'_c}b_w d}{\sqrt{1 + \left(\frac{2.5C_t T_u}{V_u}\right)^2}} = \frac{2\sqrt{4000} \times 14 \times 21}{\sqrt{1 + \left(\frac{2.5 \times 0.055 \times 483{,}660}{18{,}096}\right)^2}} = 9763 \text{ lb } (43.4 \text{ kN})$$

$$T_c = \frac{0.8\sqrt{f'_c}\Sigma x^2 y}{\sqrt{1 + \left(\frac{0.4V_u}{C_t T_u}\right)^2}} = \frac{0.8\sqrt{4000} \times 5352}{\sqrt{1 + \left(\frac{0.4 \times 18{,}096}{0.055 \times 483{,}660}\right)^2}} = 261{,}291 \text{ in} \cdot \text{lb } (29.53 \text{ kN} \cdot \text{m})$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{18{,}906}{0.85} - 9763 = 11{,}526 \text{ lb } (51.29 \text{ kN}) < 8 f'_c b_w d = 148{,}753 \; (662 \text{ kN})$$

$\therefore$ No need to enlarge section.

$$T_s = \frac{T_u}{\phi} - T_c = \frac{483{,}660}{0.85} - 261{,}291 = 307{,}721 \text{ in} \cdot \text{lb } (34.77 \text{ kN} \cdot \text{m}) < 4T_c = 1{,}045 \text{ in} \cdot \text{lb}$$

$(118.1 \text{ kN} \cdot \text{m})$ $\therefore$ No need to enlarge section.

$$\frac{A_s}{s} = \frac{V_s}{f_y d} = \frac{11{,}526}{60{,}000 \times 21} = 0.0091 \text{ in}^2/\text{in spacing for two legs}$$

Assume 1.5 in (38 mm) clear cover and no. 4 closed stirrup,

$$x_1 = 14 - 2(1.5 + 0.25) = 10.5 \text{ in } (267 \text{ mm})$$

$$y_1 = 24 - 2(1.5 + 0.25) = 20.5 \text{ in } (521 \text{ mm})$$

$$\alpha_1 = 0.66 + 0.3\left(\frac{y_1}{x_1}\right) = 0.66 + 0.33\left(\frac{20.5}{10.5}\right) = 1.3 \leq 1.5$$

Hence

$$\frac{A_t}{s} = \frac{T_s}{f_y \alpha_1 x_1 y_1} = \frac{307{,}721}{60{,}000 \times 1.3 \times 10.5 \times 20.5} = 0.0183 \text{ in}^2\text{/in spacing for two legs}$$

Stirrup for combined shear and torsion:

$$\frac{A_v}{s} = \frac{2A_t}{s} = 0.0091 + 2 \times 0.0183 = 0.046 \text{ in}^2\text{/in spacing for two legs}$$

$$\frac{50b_w}{f_y} = \frac{50 \times 14}{60{,}000} = 0.0117 < 0.046 \qquad \text{O.K.}$$

Using no. 4 and area = 2 × 0.2 = 0.4 in^2 (258 mm^2), we get

$$s = \frac{\text{area}}{A_v/s + 2A_t/s} = \frac{0.4}{0.046} = 8.7 \text{ in (221mm)}$$

The maximum allowable spacing is

$$s_{\max} = \frac{x_1 + y_1}{4} = \frac{10.5 + 20.5}{4} = 7.75 \text{ in (197 mm)}$$

Therefore use no. 4 closed stirrups at spacing s = 7.75 in (197 mm).
Longitudinal torsional steel:

$$A_t = 2A_t\left(\frac{x_1 + y_1}{s}\right) = 2 \times 0.0183(10.5 + 20.5) = 1.13 \text{ in}^2 \text{ (728.9mm}^2\text{)}$$

or

$$A_t = \frac{400xs}{f_y}\left[\left(\frac{T_u}{T_u + V_u/3C_t}\right) - 2A_t\right]\left(\frac{x_1 + y_1}{s}\right)$$

Substituting $50b_w d/f_y$ for $2A_s$ if it is larger than $2A_t$, we get

$$\frac{50b_w d}{f_y} = \frac{50 \times 14 \times 7.5}{60{,}000} = 0.875 < 2A_t = 2 \times 0.0183 \times 7.5 = 2.745$$

Hence we use $2A_t$ = 0.2745,

$$A_t = \frac{400 \times 14 \times 7.5}{60{,}000}\left[\left(\frac{40{,}305}{40{,}305 + 18{,}096/(3 \times 0.055)}\right) - 0.2745\right]\left(\frac{10.5 + 20.5}{7.5}\right)$$

$$= -0.017 \text{ in}^2 \text{ } (-10.96 \text{ mm}^2)$$

Hence we use A_t = 1.13 in^2 (728.9 mm^2), and add $A_t/4$ = 0.3 in^2 (193.5 mm^2) on each face of the cross section. Thus we use 2 no. 4 bars on each vertical side of the cross section and on the top face [= 0.4 in^2 (258 mm^2)]. The reinforcement on the bottom face becomes A_s = 3 in^2 + 0.3 in^2 = 3.3 in^2, and we use 2 no. 9 + 2 no. 8 bars [= 3.56 in^2 (2296 mm^2)]. The final design is shown in Fig. X.10.

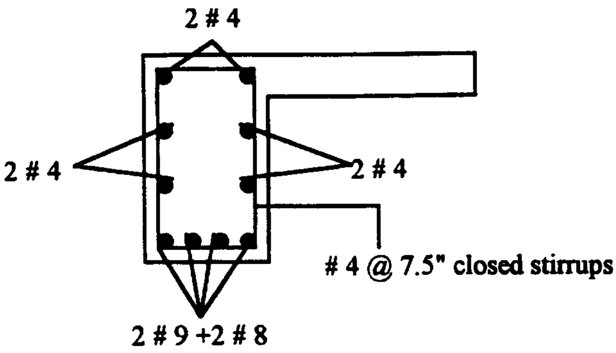


FIGURE X.10

4A.5 COLUMNS

4A.5.1 General

Columns are vertical members subjected to axial loads or a combination of axial loads and bending moments. They can be divided into three categories, depending on their structural behavior. Short compression blocks or pedestals are members with a height less than three times the least lateral dimensions. They may be designed with plain concrete with a maximum stress of $0.85\phi f'_c$, where ϕ = 0.7. For higher stresses the pedestal should be designed with reinforced concrete. Short reinforced concrete columns have low slenderness ratios, resulting in transverse deformations that will not affect the ultimate strength. Slender reinforced concrete columns have slenderness ratios that exceed the limits given for short columns. For this case the secondary moments due to transverse deformations reduce the ultimate strength of the member.

Sections of reinforced concrete columns are usually square or rectangular shapes for lower construction costs. Longitudinal steel bars are added to increase the load-carrying capacity. A substantial strength increase is obtained by providing lateral bracing for the longitudinal bars. If bracing is provided with separate closed ties, the column is referred to as a tied column. If a continuous helical spiral is used to contain the longitudinal bars, the columns are referred to as spiral columns. Commonly, a circular shaped cross section is used for spiral columns. Composite columns consist of structural steel shapes encased in concrete. The concrete may or may not be reinforced with longitudinal steel bars.

4A.5.2 Axially Loaded Columns

The theoretical nominal strength of an axially loaded short column can be determined by the following expression:

$$P_n = 0.85f'_c(A_g - A_{st}) + f_y A_{st} \tag{4A.63}$$

where A_g = gross concrete area
A_{st} = total cross-sectional area of longitudinal reinforcement

In actual construction situations there are no perfect axially loaded columns. Minimum moments occur even though no calculated moments are present. To account for these minimum moments, the ACT code requires that the theoretical nominal strength obtained from Eq. (4A.63) be multiplied by a reduction factor α. This reduction factor is equal to 0.85 for spiral columns and 0.80 for tied columns. The use of Eq. (4A.63) and of the reduction factor α is applicable for small moments where the eccentricity e is less than $0.1h$ for tied columns and less than $0.05h$ for spiral columns. For higher eccentricity values the procedures presented in the next section should be used.

4A.5.3 Uniaxial Bending and Compression

The ultimate-strength behavior of sections under combined bending and axial compression is presented in Fig. 4A.10. Depending on the magnitude of the strain in the tension steel, the section would fail either in tension or in compression. Tension failure is characterized by initial yielding of steel preceding crushing of concrete. Compression failure is due to concrete crushing before yielding of steel. If the tension steel yields at the same time that the concrete crushes, this condition is termed balanced condition and is defined by the following expressions:

$$\varepsilon_s = \varepsilon_s' = \varepsilon_y = \frac{f_y}{E_s} \tag{4A.64}$$

$$c_b = d\left(\frac{0.003}{0.003 + \varepsilon_y}\right) \tag{4A.65}$$

$$a_b = B_1 c_b \tag{4A.66}$$

$$P_{nb} = 0.85 f_c' b a_b + A_s' f_y - A_s f_y \tag{4A.67}$$

$$M_{nb} = 0.85 f_c' b a_b\left(\bar{y} - \frac{a}{2}\right) + A_s' f_y(\bar{y} - d) + A_s f_y(d - \bar{y}) \tag{4A.68}$$

$$e_b = \frac{M_{nb}}{P_{nb}} \tag{4A.69}$$

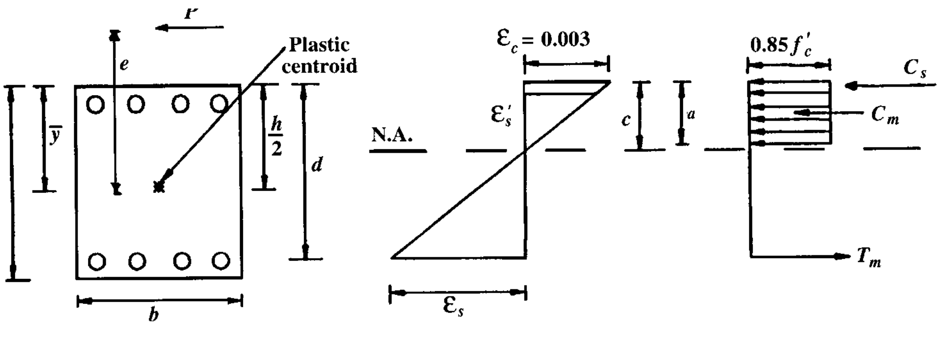


FIGURE 4A.10 Behavior of reinforced concrete columns.

Tension failure is obtained if $P_n < P_b$ or $e > e_b$, whereas compression failure is obtained if $P_n > P_b$ or $e < e_b$. For both types of failure the strain compatibility and equilibrium relationships have to be maintained.

The following procedure is used to obtain P_n and M for a given section and a known eccentricity e:

1. Assume a depth for neutral axis c. Then obtain $a = c\beta_1$.
2. Compute the compression strain of steel ε'_s and the tension strain of steel ε_s,

$$\varepsilon'_s = 0.003\left(\frac{c - d'}{c}\right) \tag{4A.70}$$

$$\varepsilon_s = 0.003\left(\frac{d - c}{d}\right) \tag{4A.71}$$

3. Compute the compression stress of steel f'_s and the tension stress of steel f_s,

$$f'_s = \varepsilon'_s E_s \leq f_y \tag{4A.72}$$

$$f_s = \varepsilon_s E_s \leq f_y \tag{4A.73}$$

4. Compute the value of P_n and M_n,

$$P_n = 0.85 f'_c ba + A'_s f'_s - A_s f_s \tag{4A.74}$$

$$M_n = 0.85 f'_c ba\left(\bar{y} - \frac{a}{2}\right) + A'_s f'_s(\bar{y} - a) + A_s f_s(d - \bar{y}) \tag{4A.75}$$

5. Compute $e^* = M_n/P_n$.
6. Compare e^* with the known eccentricity e. If equal, the values of P_n and M_n represent the nominal strength of the cross section. If different, repeat steps 1 to 6 with a different c value.

The procedure just presented, which ensures strain compatibility and equilibrium, converges rapidly, particularly if a computer program is used. It could be applied to a circular cross section with minor changes in Eqs. (4A.70) to (4A.75).

Example 4A.10 A 12 × 20 in (305 × 508 mm) tied column is carrying a vertical load with an eccentricity $e = 8$ in (203 mm), as illustrated in Fig. X.11.

Given:

$f'_c = 4000$ psi (27.56 MPa)

$f_y = 60{,}000$ psi (413.4 MPa)

$A_s = A'_s = 4$ no. 7 bars = 2.4 in² (1548 mm²)

Find the nominal load P_n

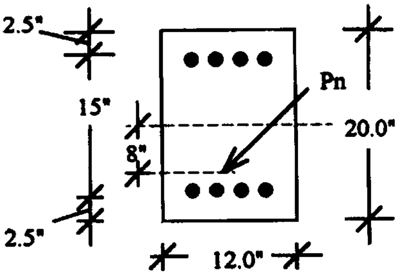


FIGURE X.11

Solution To obtain the balanced condition,

$$\varepsilon_s = \varepsilon'_s = \frac{60{,}000}{29 \times 10^6} = 0.0021$$

$$c_b = d\left(\frac{0.003}{0.003 + \varepsilon_y}\right) = 17.5\left(\frac{0.003}{0.003 + 0.0021}\right) = 10.29 \text{ in } 261.4 \text{ mm)}$$

$$a_b = \beta_1 c_b = 085 \times 10.29 = 8.75 \text{ in } (222.3 \text{ mm})$$

$$P_{nb} = 0.85 f'_c b a_b + A'_s f_y - A_s f_y = 0.85 \times 4000 \times 12 \times 8.75 = 357{,}000 \text{ lb } (1588.7 \text{ kN})$$

$$M_{nb} = 0.85 f'_c b a_b\left(\bar{y} - \frac{a}{2}\right) + A'_s f_y(\bar{y} - d) + A_s f_y(d - \bar{y})$$

$$= 0.85 \times 4000 \times 12 \times 8.75\left(10 - \frac{8.75}{2}\right) + 2.4 \times 60{,}000(10 - 2.5) + 2.4 \times 60{,}000(17.5 - 10)$$

$$= 4.17 \times 10^6 \text{ in} \cdot \text{lb } (471.21 \text{ kN} \cdot \text{m})$$

$$e_b = \frac{M_{nb}}{P_{nb}} = \frac{4.17 \times 10^6}{357{,}000} = 11.7 \text{ in } (297.2 \text{ mm})$$

$e = 8$ *in* (203 mm) $< e_b = 11.7$in (297.2 mm) $\quad \therefore$Column will fail in compression.

First trial

1. Assume $c = 15$ in (381 mm).

$$a = 15 \times 0.85 = 12.75 \text{ in } (323.9 \text{ mm})$$

2. $\varepsilon'_s = 0.003\left(\frac{c - d'}{c}\right) = 0.003\left(\frac{15 - 2.5}{15}\right) = 0.0025$

$$\varepsilon_s = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{17.5 - 15}{15}\right) = 0.0005$$

3. $f'_s = 0.0025 \times 29 \times 10^6 = 72{,}500$; take $f'_s = 60{,}000$ psi (413.4 MPa)

$$f_s = 0.0005 \times 29 \times 10^6 = 14{,}500 \text{ psi } (99.9 \text{ MPa})$$

4. $P_n = 0.85 f'_c ba + A' f_s - A_s f_s$

$$= 0.85 \times 4000 \times 12 \times 12.75 + 2.4 \times 60{,}000 - 2.4 \times 14{,}500 = 629{,}400 \text{ lb } (2800.8 \text{ kN})$$

$$M_n = 0.85 f'_c ba\left(\bar{y} - \frac{a}{2}\right) + A'_s s_y(\bar{y} - d) + A_s s_y(d - \bar{y})$$

$$= 0.85 \times 4000 \times 12 \times 12.75\left(10 - \frac{12.75}{2}\right) + 2.4 \times 60{,}000(10 - 2.5) + 2.4 \times 14{,}500(17.5 - 10)$$

$$= 3.2 \times 10^6 \text{ in} \cdot \text{lb}(361.6 \text{ kN} \cdot \text{m})$$

5. $e^* = \dfrac{M_n}{P_n} = \dfrac{3.2 \times 10^6}{629{,}400} = 5.12$ in (130.05 mm)

Second trial
Assume $c = 11$ in (279.4 mm), and by repeating the same procedure we get

$$P_n = 465{,}000 \text{ lb } (2069.3 \text{ kN})$$

$$M_n = 3.8 \times 10^6 \text{ in} \cdot \text{lb } (429.4 \text{ kN} \cdot \text{m})$$

$$e^* = \frac{3.8 \times 10^6}{465{,}000} = 8.1 \text{ in} \cong 8 \text{ in given } 465{,}000$$

Hence the nominal force P_n that can be applied on this section with an eccentricity $e = 8$ in (203 mm) is 465,000 lb (2069.3 kN).

4A.5.4 Load-Moment Interaction Diagrams

The strength of a concrete section subjected to combined axial and bending loads can be conveniently determined with the help of interaction diagrams. An interaction diagram gives a relationship between the nominal axial load P_n and the nominal moment capacity M_n of a given section, as shown in Fig. 4A.11. Each point on the diagram represents one possible combination of nominal axial load and nominal axial moment. The interaction diagram is divided into the tension control region and the compression control region by the balanced condition point (P_{nb}, M_{nb}). These diagrams are available as design aids for different column sections with different reinforcement percentages and arrangements. Typical diagrams are presented in Fig. 4A.12.

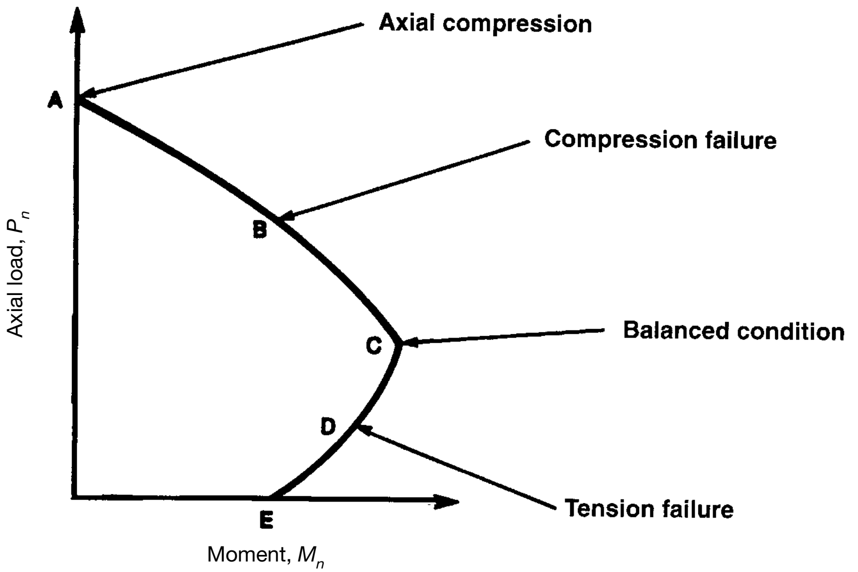
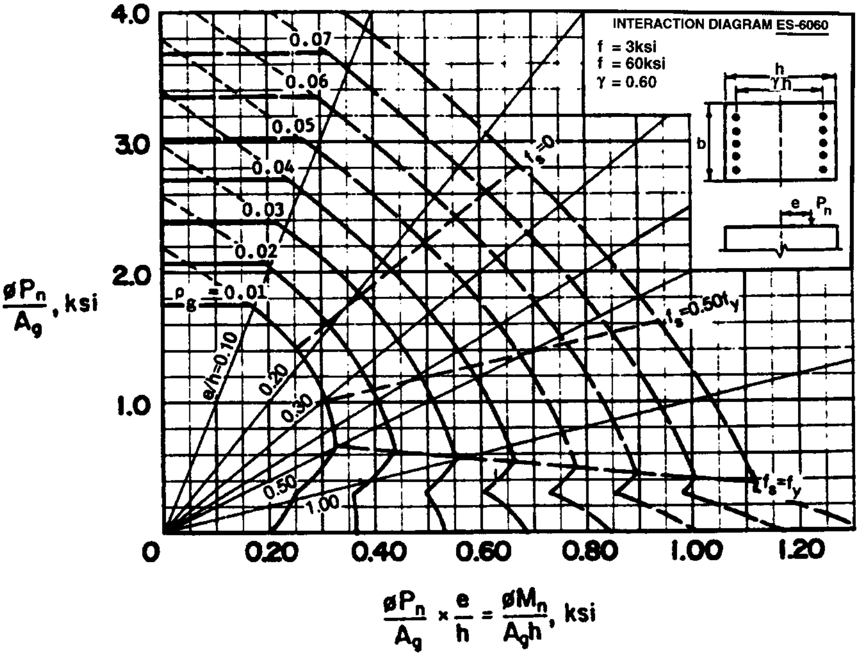


FIGURE 4A.11 Load-moment interaction diagrams for columns.



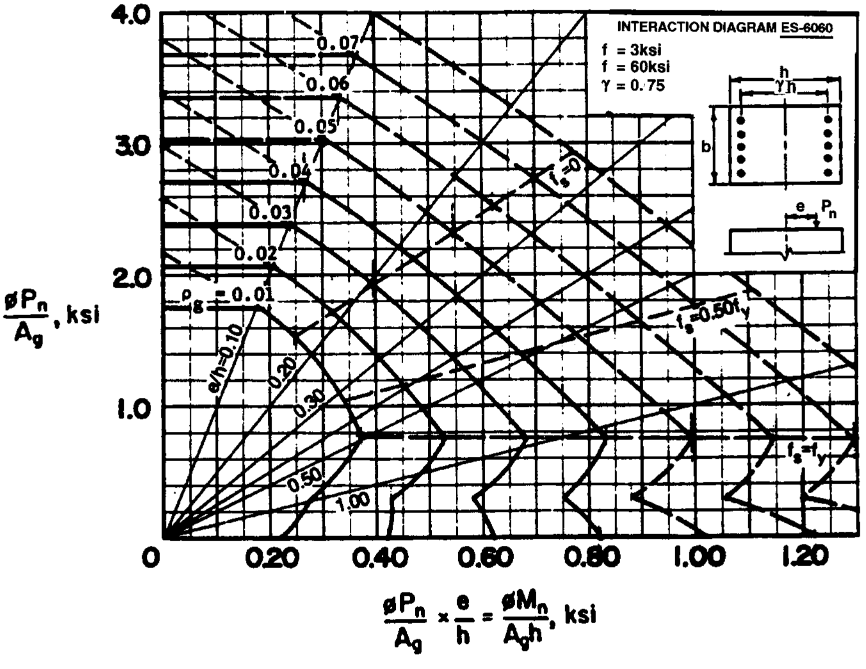
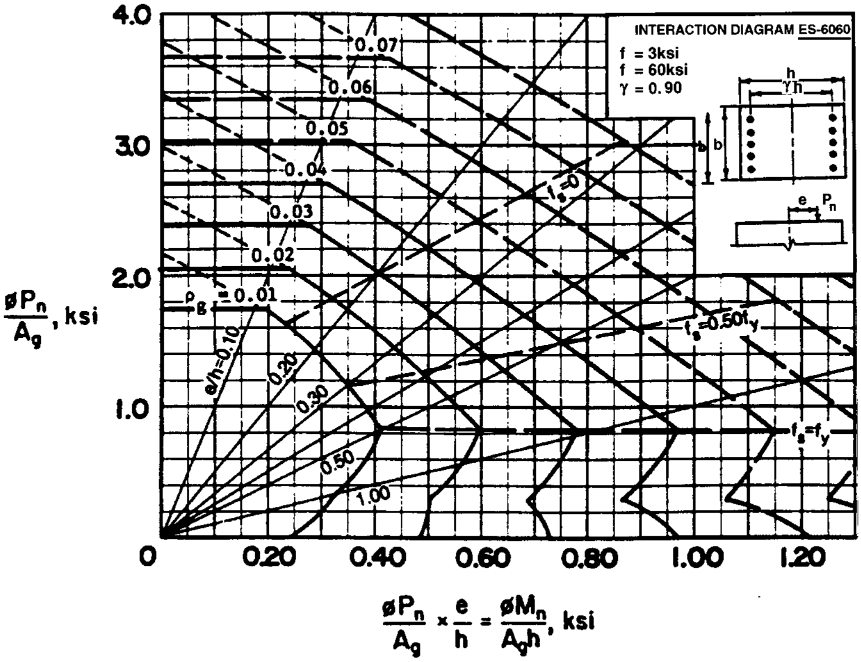


FIGURE 4A.12 Typical load-moment interaction diagrams.



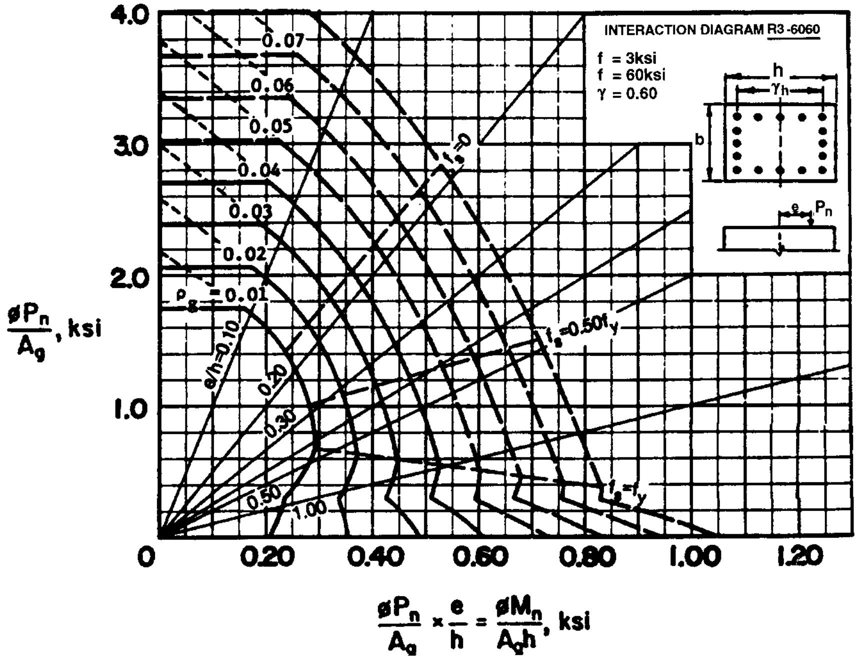
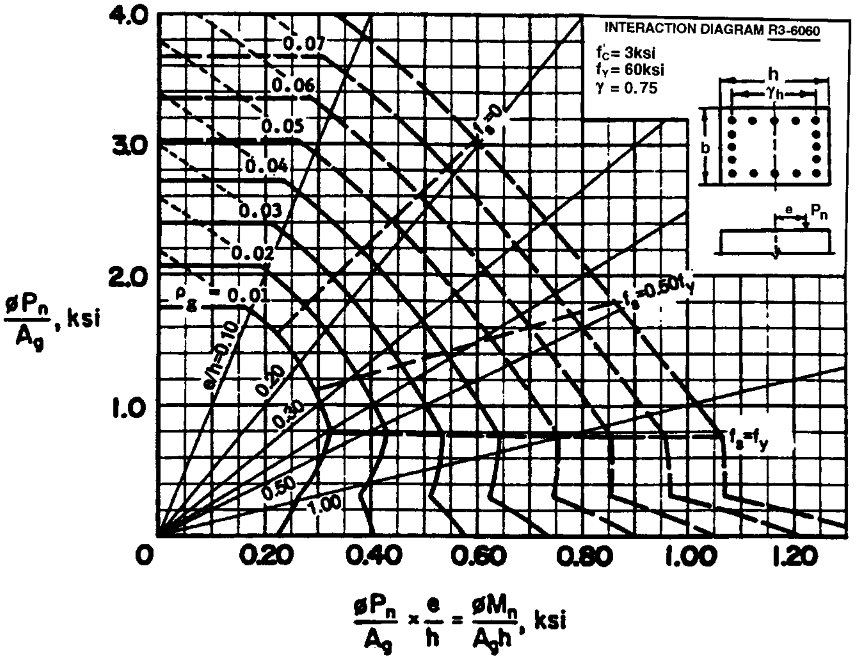


FIGURE 4A.12 (*Continued*)



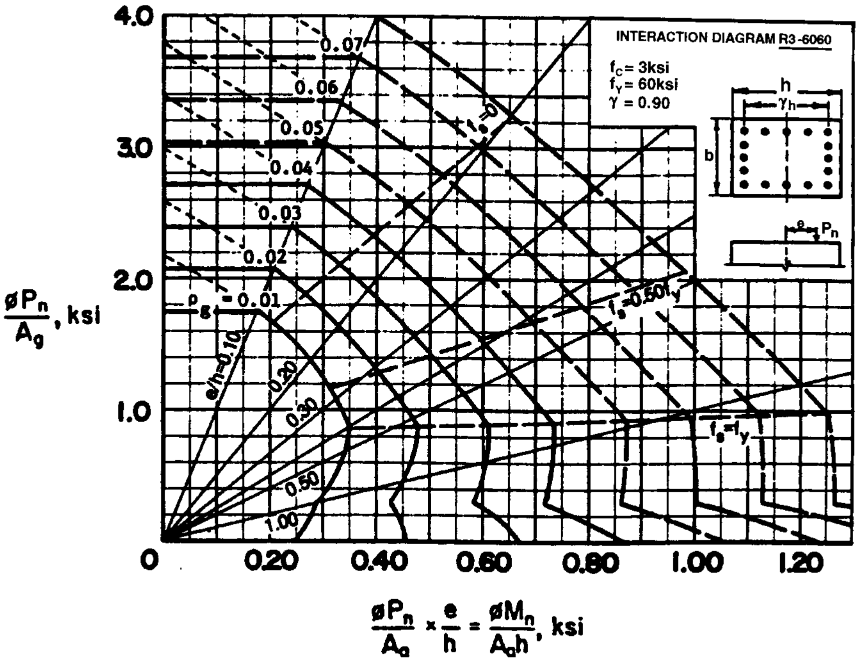
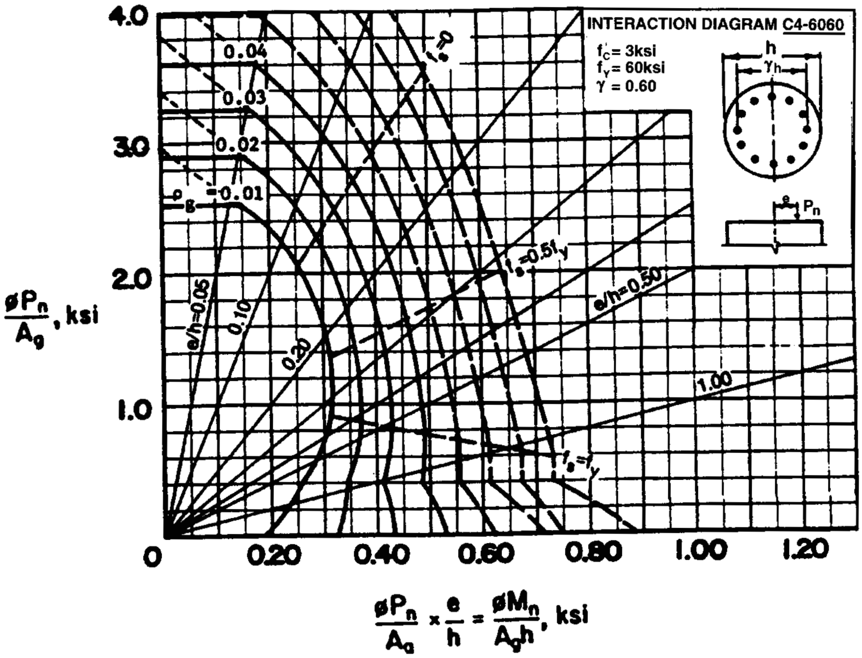


FIGURE 4A.12 (*Continued*)



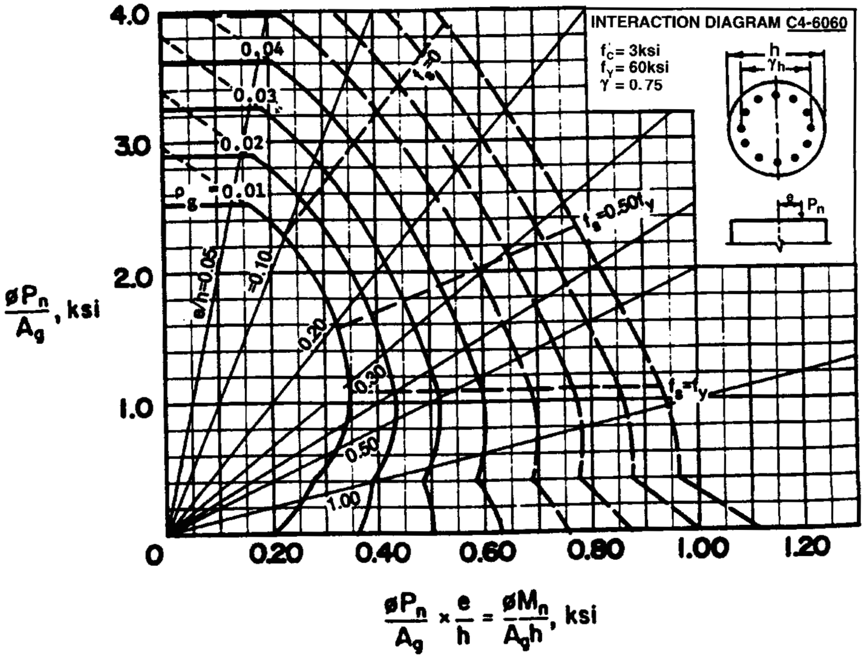


FIGURE 4A.12 (*Continued*)

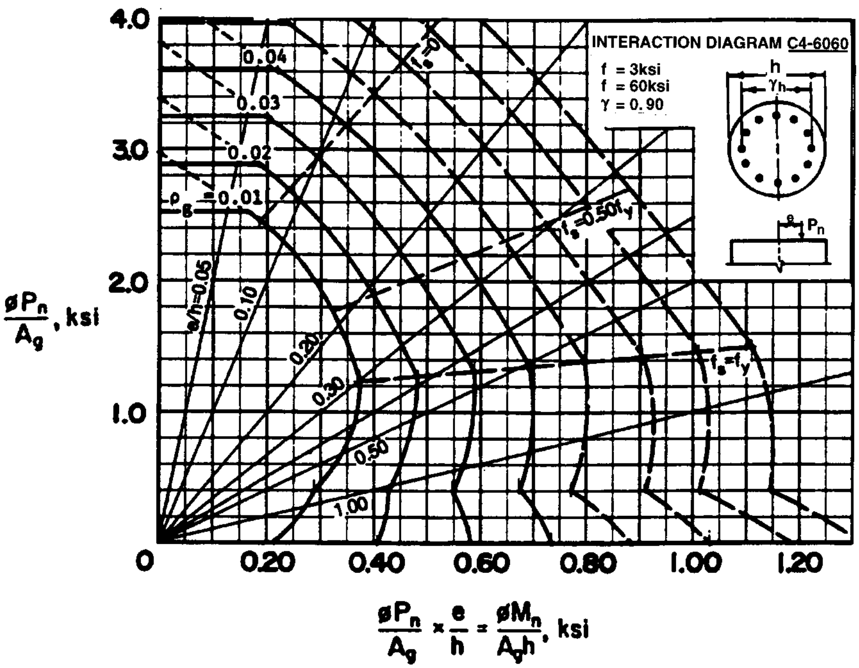


FIGURE 4A.12 (*Continued*)

Example 4A.11 Solve Example 4A. 10 using the moment interaction diagrams.

Solution

$$\gamma_h = 20 - 2.5 = 15 \text{ in } (381 \text{ mm})$$

$$\gamma = \frac{\gamma_h}{h} = \frac{15}{20} = 0.75$$

$$\rho_{\text{total}} = \frac{2 \times 2.4}{20 \times 12} = 0.02$$

$$\frac{e}{h} = \frac{8}{20} = 0.4$$

From load-moment interaction diagrams,

$$\frac{\phi P_n}{A_g} = 1.35$$

Hence

$$P_n = \frac{1.35 \times 20 \times 12}{0.7} = 463{,}000 \text{ lb } (2060.3 \text{ kN})$$

4A.5.5 Slender Columns (Buckling Effect)

The strength of slender columns is affected by the secondary stresses due to the slenderness effect. The slenderness is usually expressed in terms of the slenderness ratio KL_u/r, where K depends on the column's end conditions, L_u is the column length, and r is the radius of gyration (= I/A). The lower limits for neglecting the slenderness effect are

$$\text{Braced frames} \qquad \frac{KL_u}{r} < 34 - 12\frac{M_1}{M_2} \tag{4A.76}$$

$$\text{Unbraced frames} \qquad \frac{KL_u}{r} < 22 \tag{4A.77}$$

where M_1 and M_2 are the smaller and larger moments at the opposite ends of the compression member, respectively. This ratio is positive if the column is bent in single curvature, negative if it is bent in double curvature.

Two methods can be used to analyze slender columns.

1. The moment magnification method, where the member is designed for a magnified moment δM, with $\delta \leq 1.0$ and M the nominal moment based on an analysis neglecting the slenderness effect. This method is presented in the ACI code and is applicable to compression members with slenderness ratios lower than 100.
2. The second-order analysis takes into consideration the effect of deflection, change in stiffness, sustained load effects, and stability. This type of analysis is usually done with the aid of computers and is only required by the code for slenderness ratios greater than 100. It should he noted that the majority of reinforced concrete columns does not require such an analysis because in most cases the slenderness ratio is below 100.

In the moment magnification method, the magnified moment M_c can be determined using the following equation:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \tag{4A.78}$$

where the subscripts b and s refer to the moments due to gravity loads and lateral loads, respectively. Also,

$$\delta_b = \frac{C_m}{1 - P_u/\phi P_c} \geq 1.0 \tag{4A.79}$$

$$\delta_s = \frac{1}{1 - \Sigma P_u/\phi \Sigma P_c} \geq 1.0 \tag{4A.80}$$

$$P_c = \frac{\pi^2 EI}{(KL_u)^2} \tag{4A.81}$$

and $C_m = 0.6 + 0.4(M_1/M_2) \geq 0.4$ for columns braced against side sway and not exposed to transverse loads between supports. For all other cases $C_m = 1.0$. EI in Eq. (4A.81) must account for the effects of cracking, creep, and nonlinearity of concrete. The ACI code provides the following two equations. For a heavily reinforced member,

$$EI = \frac{E_c I_g/5 + E_s I_s}{1 + \beta_d} \tag{4A.82}$$

and for a lightly reinforced member,

$$EI = \frac{E_c I_g/2.5}{1 + \beta_d} \tag{4A.83}$$

where $$\beta_d = \frac{\text{design dead-load moment}}{\text{design total moment}}$$

In practical situations, columns have end conditions that are partially restricted by adjoining members. Therefore the factor K will vary with the ratio of column stiffness to flexure member stiffness, which provides restraint at the column ends. The value of K can be determined from charts given in Fig. 4A.13. The Ad code recommends the use of $0.5I_g$ for flexural members and I_g for compression members to compute the stiffness parameter ψ in 14g. 4A.13.

4A.5.6 Biaxial Bending

Many reinforced concrete columns are subjected to biaxial bending, or bending about both axes. Corner columns in buildings where beams and girders frame into the columns from both directions

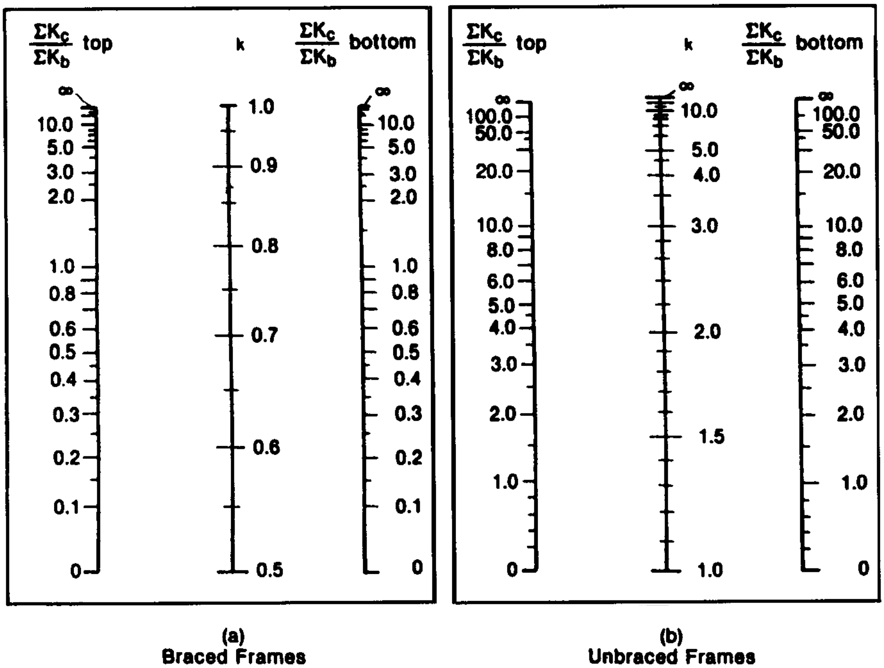


FIGURE 4A.13 Column stiffness factors iii. (a) Braced frames. (b) Unbraced frames.

are typical examples of biaxial bending. An approximate procedure developed by Bresler[5] has been found to provide satisfactory results. For a given column section, the nominal axial capacity under biaxial bending P_n can be computed using the following equation:

$$\frac{1}{P_n} = \frac{1}{P_{n_x}} + \frac{1}{P_{ny}} - \frac{1}{P_{n0}} \tag{4A.84}$$

where P_{n_x} = axial load capacity when load is placed at eccentricity e_x with $e_y = 0$
P_{ny} = axial load capacity when load is placed at eccentricity e_y with $e_x = 0$
P_{n0} = capacity for axially loaded case

Bresler' s equation produces reliable results if the axial load P_n is larger than 0.1 P_{n0}. For lower P_n values it is satisfactory to neglect the axial force and design the section as a member subjected to biaxial bending.

4A.5.7 ACI Code Requirements

The ACT code requires the following limitations on dimensions, reinforcing steel, and lateral restraint:

1. The percentage of longitudinal reinforcement should not be less than 1% nor greater than 8% of the gross cross-sectional area of the column. Usually for practical considerations the percentage of reinforcement does not exceed 4%.
2. Ties provided shall not be less than no. 3 for longitudinal bars no. 10 or smaller. They shall not be less than no. 4 for longitudinal bars larger than no. 10 and for bar bundles.
3. Tie spacing is restricted to the least of the following three values:
   a. Least lateral column dimension
   b. 16 times the diameter of the longitudinal bars
   c. 48 times the diameter of the tie
4. The spiral reinforcement size is determined from

$$\rho_s = \frac{\text{volume of spiral in one loop}}{\text{volume of concrete core for pitch}} = \frac{a_s \pi (D_c - d_b)}{\frac{1}{4}\pi \Delta_s^2 s}$$

where a_s = cross section of spiral bar
D_c = diameter of concrete core
d_b = diameter of spiral

$$(\rho_s)_{\min} = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_y}$$

where A_g = cross-sectional area
A_c = core cross section

4A.6 DEVELOPMENT OF REINFORCING BARS

4A.6.1 General

Reinforced concrete is a composite material where the compressive stresses are resisted by concrete and the tensile stresses are resisted by steel reinforcement. For this mechanism to work properly, a bond or a force transfer must exist between the two materials. The bond strength is controlled by several factors: (1) adhesion between concrete and steel reinforcement, (2) frictional resistance between steel and surrounding concrete, (3) shear interlock between bar deformations and concrete, (4) concrete strength in tension and compression, and (5) the geometrical characteristics of the steel bar—diameter, deformation spacing, and deformation height. The actual bond stress along the length of a steel bar embedded in concrete and subjected to tension varies with its location and the crack pattern. For this reason the ACI code uses the concept of development length rather than bond stress to ensure an adequate anchorage. Development length is defined as the minimum length of a bar in which the bar stress can increase from zero to the yield stress f_y. Hence a shorter embedded length will result in the bar pulling out of the concrete. The ACI code specifies different development length for steel bars in tension and in compression.

4A.6.2 Tension Development Length

A basic development length I_{db} is determined according to the following ACI code equations. Its value, however, should not be less than 12 in (0.3 in).

- No. 11 bars and smaller and deformed wire,

$$I_{db} = \frac{0.04A_b f_y}{\sqrt{f'_c}} \tag{4A.85}$$

- No. 14 bars,

$$I_{db} = \frac{0.085f_y}{\sqrt{f'_c}} \tag{4A.86}$$

- No. 18 bars,

$$I_{db} = \frac{0.125f_y}{\sqrt{f'_c}} \tag{4A.87}$$

where A_b is the cross-sectional area of the bar, and $\sqrt{f'_c}$ shall not be taken greater than 100. Values of I_{db} are given in Table 4A.3.[6]

The basic development length is multiplied by a series of multipliers given in the ACI code, secs. 12.2.3, 12.2.4, and 12.2.5, to obtain the necessary development length I_d. These multipliers account for bar spacing, amount of cover, transverse reinforcement, reinforcement location, concrete unit weight, coating of steel reinforcement, and amount of reinforcement.

For development length multipliers

1. Compute $I_d = \lambda_d I_{db}$, where λ_d is from *a, b,* or *c:*
   a. $\lambda_d = 1$ if (1) clear spacing $s = 3d_b$ or more and stirrups are used with minimum ACI code cover requirements, or (2) bars in inner layer of slab or wall with clear spacing $s \geq 3d_b$ or (3)bars with cover $\geq 2d_b$ and clear spacing $\geq 3d_b$.
   b. λ_d 2 for bars with cover $<d_b$ or clear spacing $<2d_b$.
   c. $\lambda_d = 1.4$ for bars not covered by *a* or *b*.
2. Reduce the multiplier λ_d by multiplying it by spacing
   a. 0.8 for no. 11 bars and smaller if clear spacing $\geq 5d_b$ and cover $\geq 2.5d_b$ and
   b. 0.75 for reinforcement enclosed within spiral reinforcement of diameter ≥¼ in and pitch ≤4 in, or ties of no. 4 or more with spacing ≤4 in.

TABLE 4A.3 Basic Tension Development Length*

Bar no.	f'_c = 3000 psi (20.7 MPa) Bottom bar	Top bar	Lower limit†	f'_c = 3750 psi (25.9 MPa) Bottom bar	Top bar	Lower limit†	f'_c = 4000 psi (27.6 MPa) Bottom bar	Top bar	Lower limit†	f'_c = 5000 psi (34.5 MPa) Bottom bar	Top bar	Lower limit†	f'_c = 6000 psi (41.4 MPa) Bottom bar	Top bar	Lower limit†
f'_y = 60,000 psi, normal = weight concrete															
3	4.8	6.3	12.3	4.3	5.6	11	4.2	5.4	11	3.7	4.9	9.5	3.4	4.4	8.7
4	8.8	11.4	16.4	7.8	10.2	15	7.6	9.9	14	6.8	8.8	13	6.2	8.1	12
5	14	18	21	12	16	18	12	15.5	18	10.5	14	16	9.6	12.5	15
6	19	25	25	17	22	22	18	23	21	15	20	19	14	18	18
7	26	34	29	24	31	26	23	30	25	20	27	22	19	24	20
8	35	45	33	31	40	29	30	39	29	27	35	25	25	32	23
9	44	57	37	39	51	33	38	49	32	34	44	29	31	40	26
10	S6	72	42	50	65	37	48	62	36	43	56	32	39	51	30
11	68	89	46	61	80	46	59	77	44	53	69	36	48	63	33
14	93	121	S6	83	108	50	81	105	48	72	94	43	66	86	39
18	137	178	74	122	159	66	119	154	64	106	138	S7	97	126	52
f_y = 40,000 psi, normal-weight concrete															
3	3.2	4.2	8.2	2.9	3.7	7.4	2.8	3.6	7.1	2.S	3.2	6.4	2.3	3	5.8
4	5.8	7.6	11	5.2	6.8	9.8	5.1	6.6	9.5	4.5	55.9	8.5	4.1	5.4	7.7
5	9	12	14	8.1	10.5	12	7.8	10.2	12	7.0	9.1	10.6	6.4	8.3	9.7
6	13	17	17	11.5	15	15	11.1	14.S	14.5	10	13	13	9.1	11.8	12

*$I_d = I_{db}$ × factors in ACI Sec. 12.2.3 but not less than lower limit × factors in Secs. 12.2.4 and 12.2.5 but not less than 12 in (0.3 m).
Source: From MacGregor.[6]

3. The resulting development length as modified in 1 and 2 should not be taken less than $0.03d_b f_y/\sqrt{f'_c}$, with the value of $\sqrt{f'_c}$ not to exceed 100.
4. The following additional multipliers λ_{dd} are applied for special conditions to obtain the development length $I_d = \lambda_d \lambda_{dd} I_{db}$:
   a. $\lambda_{dd} = 1.3$ for top reinforcement, that is, horizontal reinforcement with more than 12 in of concrete cast below the bars.
   b. $\lambda_{dd} = 1.3$ for lightweight concrete. When f_{ct} is specified, use $6.7\sqrt{f'_c}/f_{ct}$, where f_{ct} is the splitting tensile strength of concrete.
   c. For epoxy-coated reinforcement (1) when cover $<3d_b$ or clear spacing between bars $<6d_b$, use $\lambda_{dd} = 1.5$; (2) for other conditions use $\lambda_{dd} = 1.2$.
   d. Multiply by the excess reinforcement ratio,

$$\lambda_{dd} = \frac{A_s(\text{required})}{A_s(\text{provided})}$$

4A.6.3 Compression Development Length

The basic development length I_{db} is computed according to the Ad code from

$$I_{db} = \frac{0.02 d_b f_y}{\sqrt{f'_c}} \geq 0.0003 d_b f_y \tag{4A.85}$$

Then the following multipliers are applied to obtain $I_d = \lambda_d I_{db}$:

1. For excess reinforcement, $\lambda_d = A_s(\text{required})/A_s(\text{provided})$.
2. For spirally enclosed reinforcement, $\lambda_d = 0.75$.

The minimum total development length should be greater than 8 in (200 in).

4A.7 REFERENCES

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3. E. G. Nawy, *Reinforced Concrete: a Fundamental Approach*, 2d ed., Prentice-Hall, Englewood Cliffs N.J., 1990.
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5. B. Bresler, "Design Criteria for Reinforced Concrete Columns under Axial Load and Biaxial Bending," *Journal of the Americon Concrete Institute*, vol. 57, pp 481–490, November 1960.
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SECTION 4B
FOUNDATIONS DESIGN

A. SAMER EZELDIN

4B.1 INTRODUCTION

Footings are structural elements that transfer the loads from a structure above ground surface (superstructure) to the underlying soil. The soil-carrying capacity is in general much lower than the high stress intensities carried by the columns and walls in the superstructure. Hence the footings (substructure or foundations) can be considered as interface elements that spread the high-intensity stresses in the supporting elements to much lower stress levels along the weaker soil. This section will be limited to the design of foundations at a shallow depth. Design considerations for foundations in cold regions and in earthquake regions will be also presented.

4B.2 FOOTING TYPES

The most common types of footings are illustrated in Fig. 4B.1.

- Isolated spread footings are used beneath individual columns. They can be square or rectangular in shape. They spread the load of the column to the soil in two perpendicular directions.
- Strip footings or wall footings support bearing walls essentially in a one-dimensional action by cantilevering out on both sides of the wall.
- Combined footings are used to support two or more columns. Usually they have a rectangular or trapezoidal plan. Such footings are often used when a column is close to a property line.
- Pile caps are used to transmit the loads of columns or bearing walls to a series of piles. These piles transfer the loads from the upper poor soil layers to deeper and stronger soil layers.
- A mat or raft foundation is one large footing carrying the loads of all the columns of the structure. This type of foundation is used when weak soil layers are present but piles are not used.

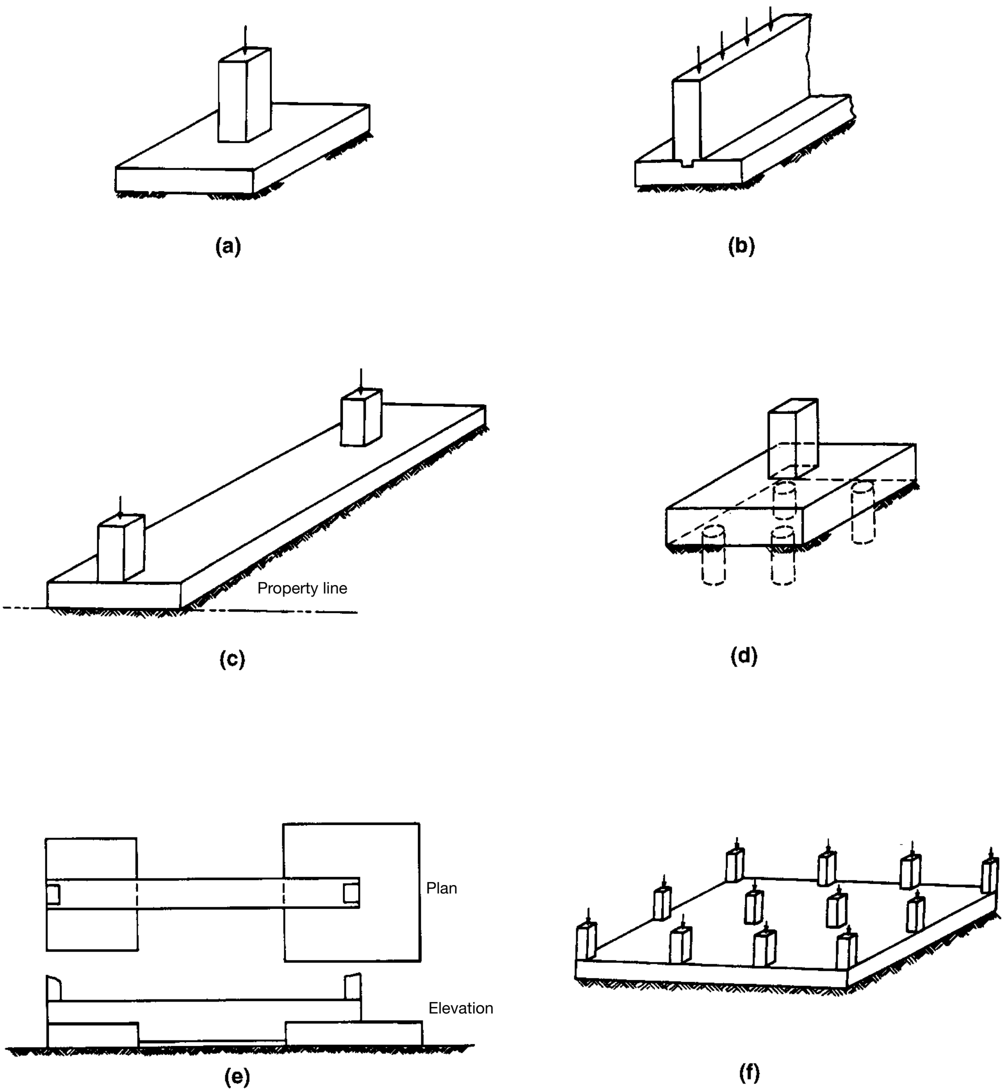


FIGURE 4B.l Types of footings. (*a*) Spread footing. (*b*) Strip or wall footing. (*c*) Combined footing. (*d*) Pile cap. (*e*) Strap footing. (*f*) Mat or raft footing.

4B.3 BEARING CAPACITY OF SOILS UNDER SHALLOW FOUNDATIONS

In order to avoid a bearing failure of the footing, in which the soil beneath the footing moves downward and outward from under the footing, the service load stress under the footing must be limited. This limitation is provided by ensuring that the service load stress q_s is less than or equal to an allowable bearing capacity q_a,

$$q_s \leq q_a = \frac{q_{\text{ult}}}{FS} \tag{4B.1}$$

where q_{ult} is the ultimate bearing capacity corresponding to failure of the footing, and FS is a factor of safety, usually taken to be 2.0 to 3.0. Soil mechanics principles are relied upon to establish the ultimate bearing capacity, which depends on the shape of the footing, its depth, the surcharge on top of the footing, the position of the underground water table, and the soil type. The allowable bearing capacity may vary from 15,000 psf for rock to 2000 psf for clay.

The soil beneath a footing is assumed to be subjected to a linearly elastic compression action. The service pressure distribution, shown in Fig. 4B.2, is obtained by the equation

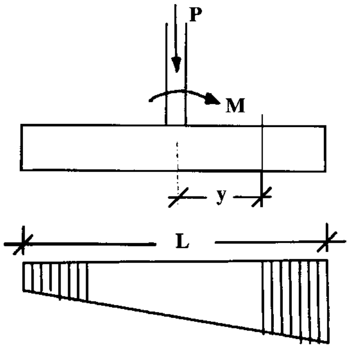


FIGURE 4B.2 Pressure distribution under footing.

$$q_s = \frac{P}{A} \pm \frac{My}{I} \tag{4B.2}$$

where P = vertical load, positive in compression
A = area of contact surface between soil and footing (length of footing L × width of footing B)
I = moment of inertia of area A
M = moment about centroidal axis of area
y = distance from centroidal axis to point where stress is being calculated

In general tensile stresses are not acceptable underneath concrete footing.

The gross soil pressure is considered the pressure caused by the total load applied on a footing, including dead loads (structure, footing, and surcharge) and live loads. In Fig. 4B.3 the gross soil pressure is

$$q_{\text{gross}} = (h_f - h_c)\gamma_s + h_c\gamma_c + \frac{P}{A} \tag{4B.3}$$

The gross soil pressure must not exceed the allowable bearing capacity q_a in order to avoid failure of the footing.

The net soil pressure is taken as the pressure that will cause internal forces in the footing. Considering Fig. 4B.3, the net soil pressure is

$$q_{\text{net}} = h_c(\gamma_c - \gamma_s) + \frac{P}{A} \tag{4B.4}$$

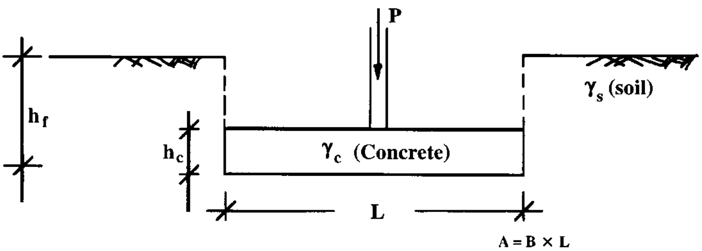


FIGURE 4B.3 Gross and net soil pressures

The net soil pressure is used to calculate the flexural reinforcement and the shear strength of the concrete footing.

4B.4 TYPES OF FAILURE OF FOOTINGS

Three different types of failure may occur in a concrete footing subjected to a concentrated load (Fintel, 1985; Winterkorn and Fang, 1975).

4B.4.1 Diagonal Tension Failure

This type of failure is also referred to as punching shear failure (Fig. 4B.4). The footing fails due to the formation of inclined cracks around the perimeter of the column. Test results have indicated that the critical section can be taken at $d/2$ from the face of the column. To avoid such a failure, the upward ultimate shearing force V_u increased by applying the strength reduction factor ϕ must be lower than the nominal punching shear strength V_c,

$$\frac{V_u}{\phi} \leq V_c \tag{4B.5}$$

V_u, acting on the tributary area shown in Fig. 4B.4, is computed with the load factors applied (see Table 3B.l) and ϕ taken as 0.85. V_c is taken as the smallest of

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}b_0 d \tag{4B.6}$$

$$V_c = \left(\frac{\alpha_s}{b_0/d} + 2\right)\sqrt{f_c'}b_0 d \tag{4B.7}$$

$$V_c = 4\sqrt{f_c'}b_0 d \tag{4B.8}$$

where b_0 = perimeter of critical section taken at $d/2$ from face of column
d = depth at which tension steel reinforcement is placed
β_c = ratio of long side to short side of column section
α_s = 40 for interior columns, 30 for edge columns, and 20 for corner columns

4B.4.2 One-Way Shear Failure

The footing fails due to the formation of inclined cracks that intercept the bottom of the slab at a distance d from the face of the column (Fig. 4B.5). For footings carrying columns with steel base plates, the distance d is measured from a line halfway between the face of the column and the edge of the base plate.

In order to avoid such a failure, Eq. (4B.5) must be satisfied. V_u is the upward ultimate shearing force acting on the tributary area shown in Fig. 4B.5 and ϕ is taken to be 0.85. V_c is taken in accordance with the Ad code as

$$V_c = 2\sqrt{f_c'}Bd \tag{4B.9}$$

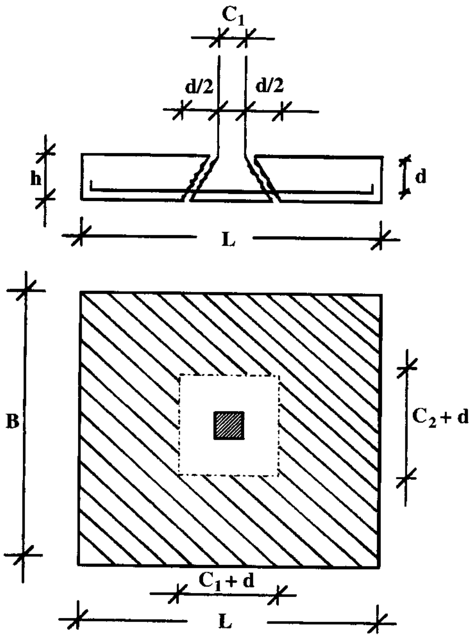


FIGURE 4B.4 Diagonal tension failure.

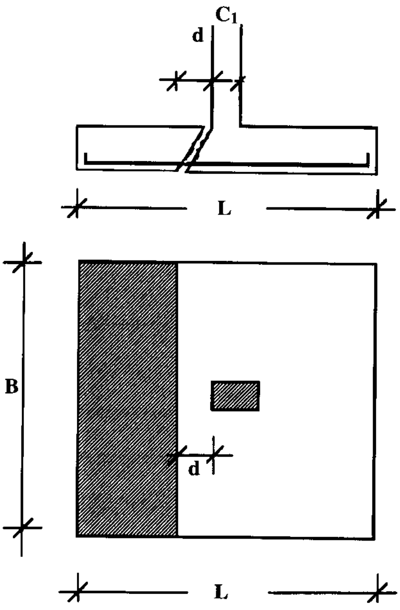


FIGURE 4B.5 One-way shear failure.

4B.4.3 Flexure Failure

A moment M_u/ϕ is acting at the face of column, as shown in Fig. 4B.6, where

$$M_u = (q_{n_u} BX)\frac{X}{2} \tag{4B.10}$$

and $\phi = 0.9$. The factored net soil pressure q_{nu} is obtained by dividing the factored applied loads on the footing by its area.

The moment M_u/ϕ must be lower than or equal to the nominal strength of the concrete section having an effective depth d, a width b, and reinforced with tension steel A_s. Thus

$$\frac{M_u}{\phi} \le M_n \tag{4B.11}$$

where

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) \tag{4B.12}$$

$$a = \frac{A_s f_y}{0.85 f_c' B} \tag{4B.13}$$

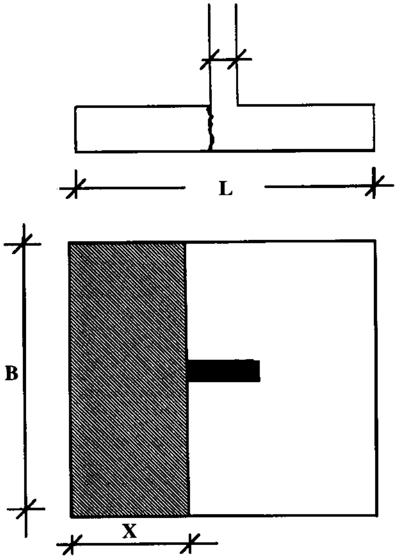


FIGURE 4B.6 Flexure failure.

In a similar manner, the moment M_u/ϕ acting at the perpendicular face of the column must be resisted by the tension reinforcement layer placed orthogonally, resulting in two layers of steel, one in each direction. ACI requires the minimum steel reinforcement placed in structural slabs of uniform thickness to be

$$(A_s)_{min} = 0.002bh, \quad \text{for } f_y = 40 \text{ or } 50 \text{ ksi (276 or 345 MPa)} \tag{4B.14}$$

$$(A_s)_{min} = 0.0018bh, \quad \text{for } f_y = 60 \text{ ksi (414 MPa)} \tag{4B.15}$$

4B.4.4 Additional Design Aspects

4B.4.4.1 Development of Reinforcement

The flexural reinforcement is provided in the footing with the assumption that the reinforcement stress reaches the yield stress f_y at the face of the column. In order to ensure that, the reinforcement must be extended beyond the critical section to develop this stress. This implies that a development length l_d must be provided from the critical section. The ACI code development length requirements for different bar diameters were presented in Sec. 3B.6.

4B.4.4.2 Load Transfer from Column to Footing

The ACI code requires that the forces acting on the column be safely transmitted to the footing. Dowels in steel connection are used to transfer any tension forces whereas the compression forces are transferred by bearing.

The bearing capacity of the column is checked by

$$\frac{P_u}{\phi} \leq 0.85 f'_c A_1 \tag{4B.16}$$

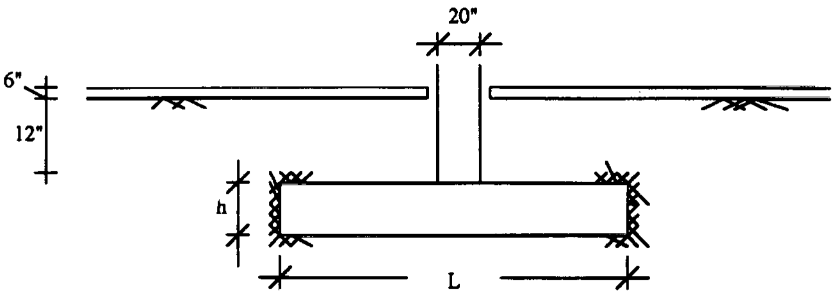
where $\phi = 0.7$
P_u = ultimate load applied on column
A_1 = the column area

The bearing capacity of the concrete footing is checked by

$$\frac{P_u}{\phi} \leq 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} \tag{4B.17}$$

where A_2 is the maximum area of the supporting surface that is geometrically similar and concentric with A_1. The value of $\sqrt{A_2/A_1}$ should not be greater than 2.

Example 4B.1: Design of Square Spread Footing Design an interior spread footing to carry a service load of 500 kips (2225 kN) and a service live load of 350 kips (1558 kN) from a 20-in (508-mm)-square tied column containing no. 11 bars [1.56 in^2 (960 mm^2)] as the principal column steel. The top of the footing will be covered with l2in (305 mm) of fill having a density of 110 lb/ft^3 (1337 kg/m^3) and a 6-in (152-mm) basement floor. The basement floor loading is 100 psf (4.78 kPa). The allowable bearing pressure on the soil q_a is 7000 psf (335 kPa). Use f_c' = 5000 psi (34.45 MPa) and f_y = 60,000 psi (413.4 MPa).



Solution

1. Estimate the thickness of the footing as between one and two times the width of the column, say h = 36 in (914 mm). The allowable net soil pressure is

$$q_{\text{net}} = 7 \text{ ksf} - (\text{weight of footing} + \text{soil} + \text{floor} + \text{floor load})$$

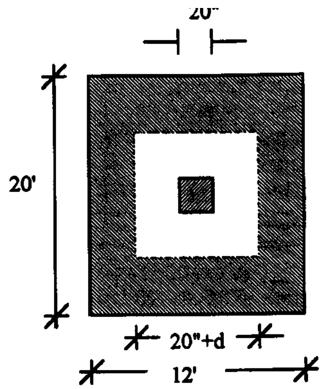
$$= 7 - \left(\frac{36}{12} \times 0.15 + 1 \times 0.11 + 0.5 \times 0.15 + 0.1\right) = 6.265 \text{ ksf (299.8 kPa)}$$

2. Required area $= \dfrac{P_D + P_L}{q_{\text{net}}} = \dfrac{500 + 350}{6.265} = 135.7 \text{ ft}^2 \text{ (12.9 m}^2\text{)}$

   Try a 12-fl (3.6-in) square by 36-in (914-mm)-thick footing.

3. The factored net soil pressure is obtained from

$$q_{n_u} = \frac{1.4 \times 500 + 1.7 \times 350}{12^2} = 9 \text{ ksf (430.6 kPa)}$$



4. The two-way shear check is performed on the critical section at the distance $d/2$ from the face of the column,

$$d = h - \text{concrete cover} - \text{bar diameter}$$

$$= 36\text{ in} - 3\text{ in} - 1\text{ in} = 32\text{ in (813 mm)}$$

$$V_u = q_{n_u}\,(\text{tributary area}) = 9\left[12^2 - \left(\frac{20+32}{12}\right)^2\right]$$

$$= 1127 = \text{kips (5015 kN)}$$

$$\frac{V_u}{\phi} = \frac{1127}{0.85} = 1326\text{ kips (5900.7 kN)}$$

V_c is the smallest of

$$\left(\frac{2+4}{\beta_c}\right)\sqrt{f'_c}\,b_0 d = \frac{(2+4/1)\sqrt{5000}\times 52\times 4\times 32}{1000} = 2824\text{ kips (12,567 kN)}$$

$$\left(\frac{\alpha_s}{b_0 d}+2\right)\sqrt{f'_c}\,b_{0d} = \frac{[40/(52\times 4/32)+2]\sqrt{5000}\times 52\times 4\times 32}{1000} = 3836\text{ kips (17,070 kN)}$$

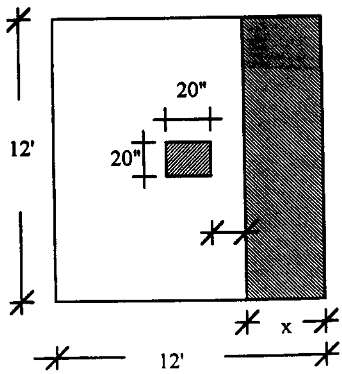
$$4\sqrt{f'_c}\,b_0 d = \frac{4\sqrt{5000}\times 52\times 4\times 32}{1000} = 1883\text{ kips (8379 kN)} \qquad \therefore \text{Controls design.}$$

Hence

$$V_c = 1883\text{ kips (8379 kN)} > \frac{V_u}{\phi} = 1326\text{ kips (5900.7 kN)}$$

The thickness of the footing is adequate to prevent two-way shear failure.

5. The one-way shear is performed on a critical section at the distance d from the face of the column. The width of the tributary area is



$$x = \frac{144-20}{2} - 32 = 30\text{ in (762 mm)}$$

$$V_u = q_{n_u}\,(\text{tributary area}) = 9\left(12\times\frac{30}{12}\right) = 270\text{ kips (1201 kN)}$$

$$\frac{V_u}{\phi} = \frac{270}{0.85} = 318\text{ kips (1415 kN)}$$

$$V_c = 2\sqrt{f'_c}\,bd = \frac{2\sqrt{5000}\times 12\times 12\times 32}{1000}$$

$$= 652\text{ kips (2901 kN)}$$

Hence

$$V_c = 652 \text{ kips (2901 kN)} > \frac{V_u}{\phi} = 318 \text{ kips (1415 kN)}$$

The thickness of the footing is capable of preventing one-way shear failure.

6. Design for flexure reinforcement. The width of the tributary area is

$$y = \left(\frac{144 - 20}{2}\right) = 62 \text{ in (1575 mm)}$$

$$M_u = 9\left(12 \times \frac{62}{12} \times \frac{62}{2 \times 12}\right) = 1442 \text{ ft} \cdot \text{kip 1955 kN} \cdot \text{m)}$$

$$\frac{M_u}{\phi} = \frac{1442}{0.9} = 1602 \text{ ft} \cdot \text{kips (2172.6 kN} \cdot \text{m)}$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right)$$

Assuming $(d - a/2) = 0.9d$,

$$(A_s)_{\text{req}} = \frac{M_u/\phi}{f_y(0.9d)} = \frac{1602 \times 12{,}000}{60{,}000(0.9 \times 32)} = 11 \text{ in}^2 \text{ (7095 mm}^2\text{)}$$

$$(A_s)_{\min} = 0.0018bh = 0.0018(144 \times 36) = 9.3 \text{ in}^2 \text{ (5998 mm}^2\text{)}$$

Choose 11 no. 9 each way; then $A_s = 11 \text{ in}^2 > (A_s)_{\min}$.
Check the chosen area of the reinforcement:

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{11 \times 60{,}000}{0.85 \times 5000 \times 144} = 1.08 \text{ in (27.4 mm)}$$

Then

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) = 11 \times 60{,}000\left(32 - \frac{1.08}{2}\right) = 20.8 \times 10^6 \text{ in} \cdot \text{lb (2350 kN} \cdot \text{m)}$$

$$= 1730 \text{ ft kips (2350kN} \cdot \text{m)} > \frac{M_u}{\phi} = 1602 \text{ ft} \cdot \text{kips (2172.6 kN} \cdot \text{m)}$$

7. Check the development length:

$$I_{db} = 0.04\frac{A_b f_y}{\sqrt{f_c'}} = 0.04\frac{1.0 \times 60{,}000}{\sqrt{5000}} = 33.94 \text{ in} \cong 34 \text{ in (864 mm)}$$

No increase in the basic development length I_d is needed to account for the effects of bar spacing, cover, stirrup confinement, and reinforcement location.

$$(I_d)_{\min} = 0.03d_b \frac{f_y}{\sqrt{f_c'}} = 0.03 \times 1.125 \frac{60{,}000}{\sqrt{5000}} = 28.7 \text{ in (729 mm)}$$

Hence choose I_d = 34 in (864 mm).

The bar length available from the location of the maximum moment on each side is

$$y - \text{concrete cover} = 62 - 3 = 59 \text{ in (1499 mm)} > 34 \text{ in (864 mm)}$$

Therefore the development length is provided.

8. Check the bearing at the column-footing interface:

$$P_u = 1.4 \times 500 + 1.7 \times 350 = 1295 \text{ kips (5763 kN)}$$

$$\frac{P_u}{\phi} = \frac{1295}{0.7} = 1850 \text{ kips (8233 kN)}$$

Footing capacity:

$$P_n = 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}$$

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{144 \times 144}{20 \times 20}} = 7.2 \qquad \therefore \text{Use} \sqrt{\frac{A_2}{A_1}} = 2.$$

Then

$$P_n = \frac{0.85 \times 5000 \times 20 \times 20 \times 2}{1000} = 3400 \text{ kips (15,130 kN)}$$

$$P_n = 3400 \text{ kips (15,130 kN)} > \frac{P_u}{\phi} = 1850 \text{ kips (8233 kN)} \qquad \text{O.K.}$$

Column capacity:

$$P_n = 0.85 f_c' A_1 = \frac{0.85 \times 5000 \times 20 \times 20}{1000} = 1700 \text{ kips (7565 kN)}$$

$$P_n < \frac{P_u}{\phi}$$

Hence dowels are needed to transfer the excess load.

$$\text{Area of dowel required} = \frac{1850 - 1700}{f_y} = \frac{150}{60} = 2.5 \text{ in}^2 \text{ (1613 mm}^2\text{)}$$

The area of the dowel must be higher than the minimum specified by the ACI code,

$$(\text{Area of dowel})_{\min} = 0.005A_g = 0.005 \times 20 \times 20 = 2 \text{ in}^2 \ (1290 \text{ mm}^2) < 2.5 \text{ in}^2 \ (1613 \text{ mm}^2)$$

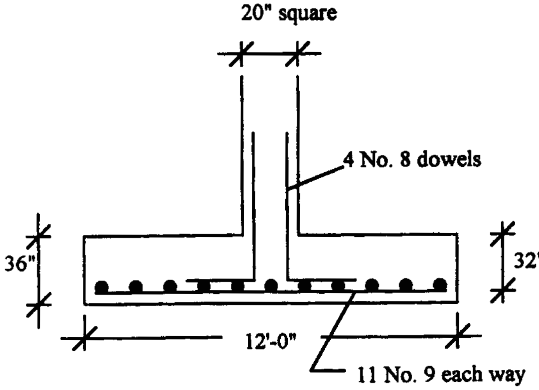
Hence the value of 2.5 in^2 controls. Choose 4 no. 8 bars [3.16 in^2 (2038 mm^2)]. The dowels must extend at least the compression development length of the 8 bar into the footing,

$$I_{db} = 0.02d_b \frac{f_y}{\sqrt{f_c'}} \geq 0.0003d_b f_y$$

$$I_{db} = 15 \text{ in } (381 \text{ mm}) < 16 \text{ in } (406 \text{ mm})$$

Hence extend 4 no. 8 dowels at least 16 in (406 mm) into the footing.

The complete design is detailed here.



4B.5 RECTANGULAR FOOTINGS

Rectangular footings are usually employed as spread footings when the space is inadequate for a square footing. The design procedures for these footings are basically similar to those of square footings, except that the one-way shear and bending moments have to be checked in both principal directions. Also in such footings the flexural reinforcement in the short direction has to be distributed in three regions with more concentration in the region beneath the column (Fig. 4B.7). The total required reinforcement A_s is obtained such that the bending moment at the column face (section A-A) is resisted. The reinforcement in the central region under the column shall be $A_s[2/(\beta + 1)]$, where β is the ratio of the long side of the footing to the short side. The remaining reinforcement is distributed equally between the two outer regions of the footing.

Example 4B.2: Design of Rectangular Footing Redesign the footing of Example 4B.1, given that the maximum width of the footing cannot exceed 10 ft (3 in).

Solution

1. From the solution of Example 4B.1, take $h = 36$ in (914 mm) and $d = 32$ in (813 mm). Then

$$q_{\text{net}} = 6.265 \text{ ksf } (299.8 \text{ kPa})$$

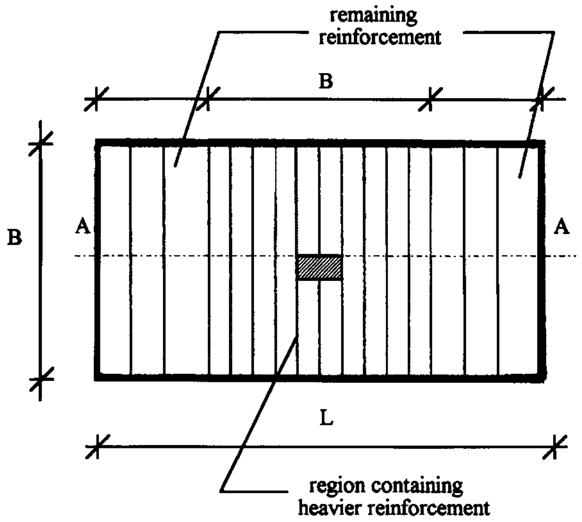


FIGURE 4B.7 Reinforcement in short direction of a rectangular footing.

2.
$$\text{Required area} = \frac{P_D + P_L}{q_{\text{net}}} = \frac{500 + 350}{6.265} = 135.7\ \text{ft}^2\ (12.9\ \text{m}^2)$$

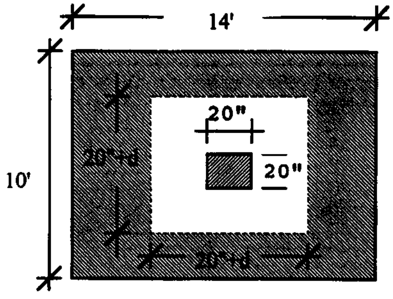
$$\text{Required length} = \frac{135.7}{10} = 13.57\ \text{ft}\ (4.2\ \text{m}) \qquad \text{Take } L = 14\ \text{ft}\ (4.25\ \text{m}).$$

Try a footing 10 ft (3 m) wide by 14 ft (4.25 m) long by 36 in (914 mm) thick.

3. The factored net soil pressure is

$$q_{n_u} = \frac{1.4 \times 500 + 1.7 \times 350}{10 \times 14} = 9.25\ \text{ksf}\ (442.6\ \text{kPa})$$

4. Two-way shear analysis:



$$V_u = q_{n_u}(\text{tributary area}) = 9.25\left[140 - \left(\frac{20+32}{12}\right)^2\right] = 1121 \text{ kips (4988 kN)}$$

$$\frac{V_u}{\phi} = \frac{1121}{0.85} = 1319 \text{ kips (5869 kN)}$$

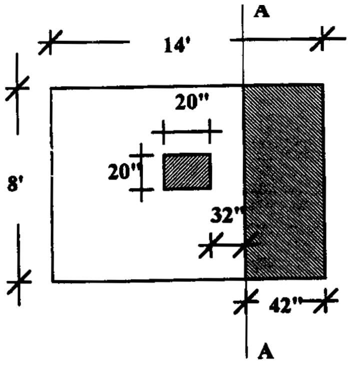
$$V_c = 4\sqrt{f'_c}b_0 d = \frac{4\sqrt{5000} \times 52 \times 4 \times 32}{1000} = 1883 \text{ kips (8379 kN)}$$

Hence

$$V_c = 1883 \text{ kips (8379 kN)} > \frac{V_u}{\phi} = 1319 \text{ kips (5869 kN)}$$

The thickness of the footing is adequate to prevent two-way shear failure.

5. The one-way shear is performed along two sections.
   a. Section A-A:



$$V_u = 9.25\left(8 \times \frac{42}{12}\right) = 259 \text{ kips (1153 kN)}$$

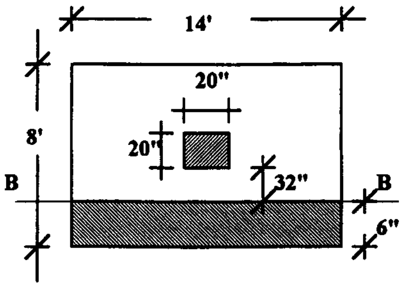
$$\frac{V_u}{\phi} = \frac{259}{0.85} = 305 \text{ kips (1356 kN)}$$

$$V_c = 2\sqrt{f'_c}bd \; \frac{2\sqrt{5000} \times 8 \times 12 \times 32}{1000} = 434 \text{ kips (1931 kN)}$$

Hence

$$V_c = 434 \text{ kips (1931 kN)} > \frac{V_u}{\phi} \; 305 \text{ kips (1356 kN)}$$

b. Section B-B:



$$V_u = 9.25\left(14 \times \frac{6}{12}\right) = 64.75 \text{ kips (289 kN)}$$

$$\frac{V_u}{\phi} = \frac{64.75}{0.85} = 76 \text{ kips (340 kN)}$$

$$V_{uc} = \frac{2\sqrt{5000} \times 14 \times 12 \times 32}{1000} = 760 \text{ kips (3382 kN)}$$

The thickness of the footings is capable of preventing two-way shear failure in both directions.

6. Design for flexure reinforcement
   a. Section A-A (Long direction):

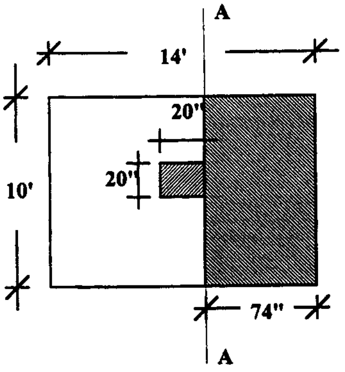
$$M_u = 9.25\left(10 \times \frac{74}{12} \times \frac{74}{2 \times 12}\right) = 1759 \text{ ft} \cdot \text{kips (2385 kN} \cdot \text{m)}$$

$$\frac{M_u}{\phi} = \frac{1759}{0.9} = 1954 \text{ ft} \cdot \text{kips (2650.2 kN} \cdot \text{m)}$$

$$(A_s)_{\text{req}} = 13.6 \text{ in}^2 \text{ (8772 mm}^2\text{)}$$

$$(A_s)_{\min} = 0.0018bh = 0.0018 \times 10 \times 12 \times 36 = 7.8 \text{ in}^2 \text{ (5031 mm}^2\text{)} < (A_s)_{\text{req}}$$

Choose 14 no. 9 [(14 in^2)(9030 mm^2)] in the long direction.



b. Section B-B (short direction):

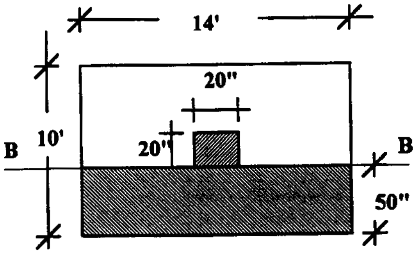
$$M_u = 9.25\left(14 \times \frac{50}{12} \times \frac{50}{2 \times 12}\right) = 1124 \text{ ft} \cdot \text{kip } (1524 \text{ kN} \cdot \text{m})$$

$$\frac{M_u}{\phi} = \frac{1124}{0.9} = 1249 \text{ ft} \cdot \text{kips } (1693.5 \text{ kN} \cdot \text{m})$$

$$(A_s)_{\text{req}} = 8.8 \text{ in}^2 \ (5676 \text{ mm}^2)$$

$$(A_s)_{\min} = 0.0018bh = 0.0018 \times 14 \times 12 \times 36 = 10.9 \text{ in}^2 \ (5031 \text{ mm}^2) \qquad \therefore \text{Controls.}$$

Choose $A_s = 10.9$ in^2 (7031 mm^2) in the short direction.



In the 10-ft inner region provide

$$10.9\left(\frac{2}{\beta + 1}\right) = 10.9\left(\frac{2}{14/10 + 1}\right) = 9.08 \text{ in}^2 \ (5857 \text{ mm}^2) \qquad (12 \text{ no. } 8)$$

In the 2-ft outer regions provide

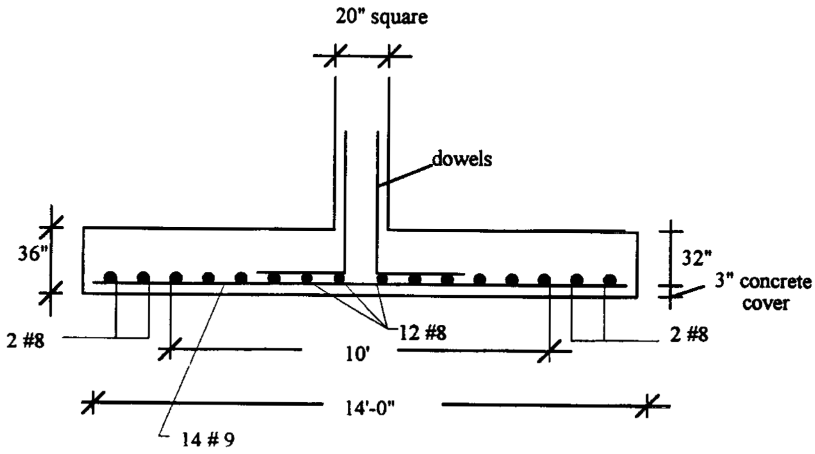
$$\frac{10.9 - 9.08}{2} = 0.91 \text{ in}^2 \ (587 \text{ mm}^2) \text{ on each side} \qquad (2 \text{ no. } 8)$$

The checks for the development length and the bearing at the column-footing interface are similar to those in Example 4B.1. The details of the final design are shown at the top of the next page

4B.6 ECCENTRICALLY LOADED SPREAD FOOTINGS

In some cases, due to a moment at the column base or an eccentrically applied load, the bearing pressure beneath the footing will deviate from the uniform distribution shown in Fig. 4B.2. The design of such a footing can be performed in a manner similar to that of a square or rectangular footing with the following conditions satisfied:

1. Tensile stresses are not generated beneath the footing under extreme loading conditions.
2. The difference in compressive stresses between the two edges of the footing is not extremely high in order to avoid tilting settlement of the footing.



3. The designs for one-way shear, two-way shear, and bending moment are performed using the actual pressures under the footing resulting from critical loading conditions that might occur.

4B.7 COMBINED FOOTINGS

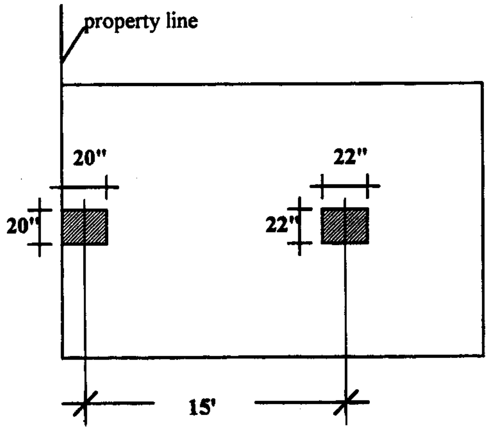
A combined footing 15 usually used when an exterior column is close to a property line, preventing the use of an isolated spread footing (see Fig. 4B.1). Thus a combined footing is used to support the exterior column along with an interior column. The shape of a combined footing is usually rectangular or trapezoidal. That shape is carefully designed in order to have the centroid of the footing coincide with the resultant of the column loads applied to the footing. For cases where the load is lower on the exterior column P_{ext} than on the interior column P_{int} a rectangular combined footing is considered an economical solution. In cases when $0.5 < P_{int}/P_{ext} < 1$ a trapezoidal footing is preferred. However, when $P_{int}/P_{ext} < 0.5$, a strip or cantilever footing should be considered. In a strip or cantilever footing, the overturning of the exterior footing is prevented by connecting it with an adjacent interior footing using a strip beam. The exterior footing is designed for one-way bending whereas the interior footing is designed for two-way bending, as in isolated footings. The strip beam is subjected to a constant shear force and a linearly decreasing negative moment. This behavior is similar to a cantilever beam. It is preferable that all three elements, namely, the exterior footing, the interior footing, and the strip beam, have the same thickness. This thickness is chosen such that the shear requirement for the footings and the shear and flexure requirements for the strip beam are satisfied.

Example 4B.3: Design of a Combined Footing Design a combined rectangular footing to support two columns. The exterior column is 20 in (508 mm) square, carrying service loads of 150 kips (667.5 kN) dead load and 120 kips (534 kN) live load. The interior column is 22 in (559 mm) square carrying service loads of 200 kips (890 kN) dead load and 180 kips (801 kN) live load. The distance between the columns is 15 ft (4.6 in) centerline to centerline. The top of the footing is 3 ft (914 mm) below grade and the fill above the footing is 120 lb/ft^3 (1459 kg/in^3). Use $f'_c = 4000$ psi (27.56 MPa) and $f_y = 60{,}000$ psi (413.4 MPa).

Solution

1. Estimate the depth of the footing to be one to two times the column dimension. Take $h = 36$ in (914 mm). Hence

$$d = h - \text{cover} - \text{bar diameter} = 36 - 3 - 1 = 32 \text{ in (813 mm)}$$



The allowable net soil pressure is

$$q_{\text{net}} = 5 - (\text{weight of footing + soil}) = 5 - (3 \times 0.15) - (3 \times 0.12) = 4.19 \text{ ksf (200.5 kPa)}$$

2. $$\text{Required area} = \frac{P_D + P_L}{q_{\text{net}}} = \frac{(150 + 200) + (120 + 180)}{4.19} = 155 \text{ ft}^2 \text{ (14.73 m}^2\text{)}$$

The distance of the center of gravity of loads from the exterior column is

$$\frac{(150 + 120)0 + (200 + 180)15}{(150 + 120) + (200 + 180)} = 8.77 \text{ ft (2.67 m)}$$

The distance from the property line to the center of gravity is

$$\frac{10 \text{ in}}{12} + 8.77 \text{ ft} = 9.6 \text{ ft (2.93 m)}$$

$$\text{Length of footing} = 2 \times 9.6 \text{ ft} = 19.2 \text{ ft (5.85 m)} \qquad \therefore \text{ Say 19.5 ft (5.9 m).}$$

$$\text{Width of footing} = \frac{155}{19.5} = 7.95 \text{ ft (24 m)} \qquad \therefore \text{ Say 8 ft (2.5 m).}$$

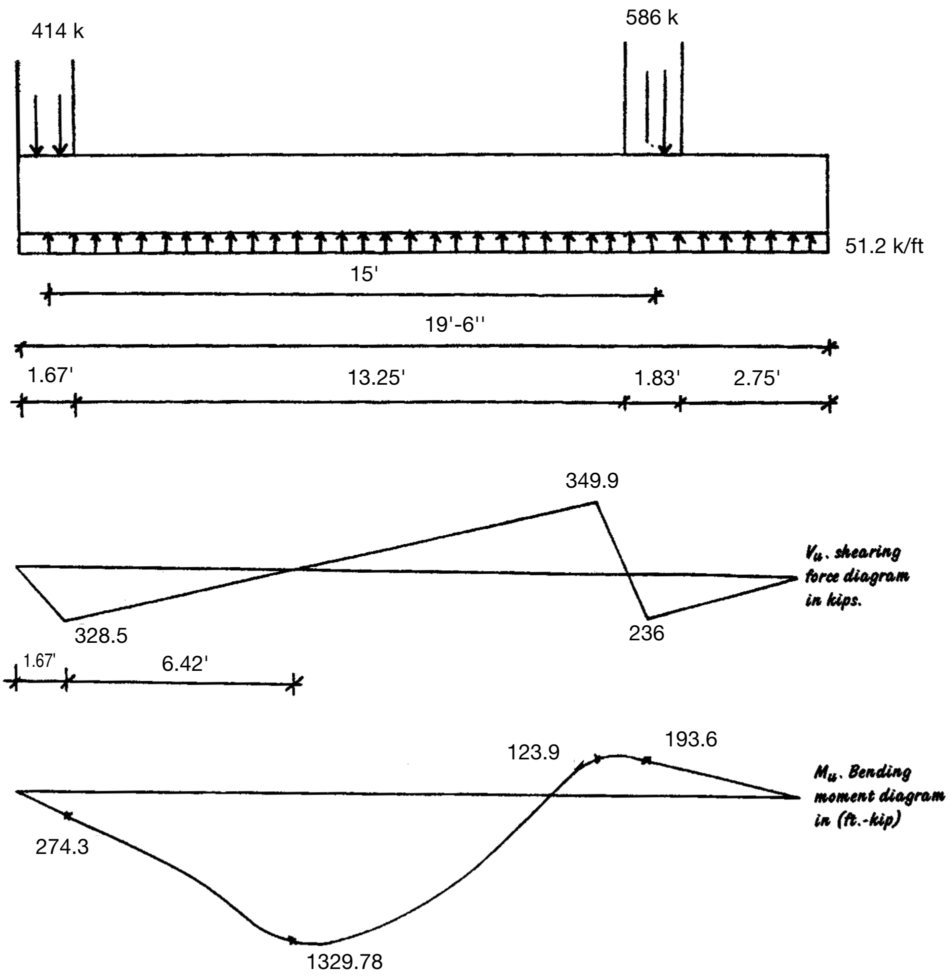
Try a 19.5 × 8 ft (5.9 × 2.5 m) rectangular footing with 36-in (914-mm) thickness.

3. The factored net soil pressure is

$$q_{n_u} = \frac{1.4(150 + 200) + 1.7(120 + 180)}{19.5 \times 8} = 6.4 \text{ ksf (306.25 kPa)}$$

$$= 6.4 \times 8 = 51.2 \text{ kips/ft (7.47 kN/m)}$$

4. Using q_{na}/ft, determine the factored bending moment and shearing force diagrams for the footing. These diagrams are plotted here for the full 8-ft (2.5-in) width of the footing.



5. Check one-way shear: V_u at a distance d from the interior face of the inner column is

$$V_u = 349.9 - \frac{32}{12} 51.2 = 213.3 \text{ kips } (949.5 \text{ kN})$$

$$\frac{V_u}{\phi} = \frac{213.37}{0.85} = 251 \text{ kips } (1117 \text{ kN})$$

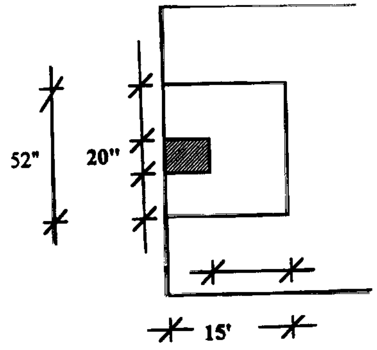
$$V_c = 2\sqrt{f_c'}bd = 2\sqrt{4000} \times 8 \times 12 \times 32 = 388.6 \text{ kips } (1729.3 \text{ kN})$$

Hence,

$$V_c = 388.6 \text{ kips } (1729.3 \text{ kN}) > \frac{V_u}{\phi} = 251 \text{ kips } (1117 \text{ kN})$$

The thickness of the footing is adequate to prevent one-way shear failure.

6. Check of two-way shear
   a. Exterior column:



$$V_u = 414 - q_{na}\left(\frac{52}{12}\right)\left(\frac{36}{12}\right) = 414 - 6.4 \times 4.33 \times 3 = 330.9 \text{ kips } (1472.5 \text{ kN})$$

$$\frac{V_u}{\phi} = \frac{330.9}{0.85} = 389.3 \text{ kips } (1732 \text{ kN})$$

$$b_0 = 2 \times 36 + 52 = 124 \text{ in } (3150 \text{ mm})$$

V_c is the smallest of

$$\frac{(2+4)\sqrt{4000} \times 124 \times 32}{1000} = 1506 \text{ kips } (6702 \text{ kN})$$

$$\left(\frac{30}{124/32} + 2\right)\sqrt{4000} \times 124 \times 32 = 2444 \text{ kips } (10876 \text{ kN})$$

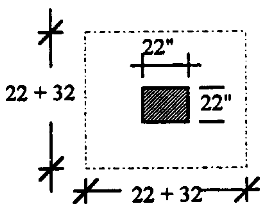
$$4\sqrt{4000} \times 124 \times 32 = 1003 \text{ kips } (4463 \text{ kN}) \qquad \therefore \text{ Controls design.}$$

Hence

$$\frac{V_u}{\phi} = 389.3 \text{ kips } (1732 \text{ kN}) < 1003 \text{ kips } (4463 \text{ kN})$$

The thickness is adequate for the exterior column.

b. Interior column:



$$V_u = 586 - 6.4\left(\frac{54}{12}\right)\left(\frac{54}{12}\right) = 456.4 \text{ kips } (2031 \text{ kN})$$

$$\frac{V_u}{\phi} = \frac{456.4}{0.85} = 537 \text{ kips } (2389 \text{ kN})$$

$$b_0 = 4 \times 54 = 216 \text{ in } (5486 \text{ mm})$$

$$V_c = 4\sqrt{4000} \times 216 \times 32 = 1749 \text{ kips } (7783 \text{ kN})$$

Hence

$$\frac{V_u}{\phi} = 537 \text{ kips } (2389 \text{ kN}) < 1749 \text{ kips } (7783 \text{ kN})$$

The thickness is adequate for the interior column.

7. Design for flexure reinforcement
   a. Midspan negative moment:

$$\frac{M_u}{\phi} = \frac{1329.78}{0.9} = 1477.5 \text{ ft} \cdot \text{kips } (2003.5 \text{ kN} \cdot \text{m})$$

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

Assuming $(d - a/2) = 0.9d$,

$$(A_s)_{\text{req}} = \frac{M_u/\phi}{f_y(0.9d)} = \frac{1477.5 \times 12{,}000}{60{,}000 \times 0.9 \times 32} = 10.26 \text{ in}^2 \text{ } (6618 \text{ mm}^2)$$

$$(A_s)_{\text{min}} = 0.0018bh = 0.0018 \times 8 \times 12 \times 36 = 6.22 \text{ in}^2 \text{ } (4012 \text{ mm}^2)$$

Choose 11 no.9 $\quad \therefore A_s = 11 \text{ in}^2 \ (7095 \text{ mm}^2) > (A_s)_{\text{min}}$
Checking the area of the reinforcement,

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{11 \times 60{,}000}{0.85 \times 4000 \times 8 \times 12} = 2.0 \text{ in } (50.8 \text{ mm})$$

$$M_n = 11 \times 60{,}000 \left(32 - \frac{2}{2} \right) = 20.45 \times 10^6 \text{ in} \cdot \text{lb}$$

$$= 1704 \text{ ft} \cdot \text{kips } (2311 \text{ kN} \cdot \text{m}) > 1477.5 \text{ ft} \cdot \text{kips } (2003.49 \text{ kN} \cdot \text{m})$$

Use 11 no. 9 top bars at midspan.

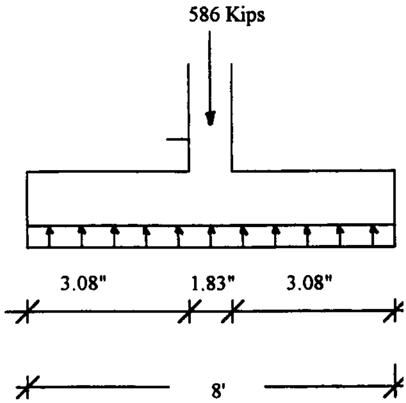
   b. Interior column positive moment:

$$\frac{M_u}{\phi} = \frac{193.6}{0.9} = 215 \text{ ft} \cdot \text{kips } (291.54 \text{ kN} \cdot \text{m})$$

This would require $A_s = 2 \text{ in}^2$ (1290 mm²), which is less than $(A_s)_{\text{min}} = 6.22 \text{ in}^2$ (4012 mm²). Use 7 no. 9 bottom bars for the interior column.

8. Design for transverse beams under columns. It is assumed that transverse beams under each column transmit the load from the longitudinal direction into the columns. The width of the transverse beam is taken to be the width of the column plus an extension d/2 on each side of the column.
   a. Transverse steel under interior column:

$$\text{Beam width} = 22 + 2\left(\frac{32}{2} \right) = 54 \text{ in } (1372 \text{ mm})$$



$$q_{n_u} = \frac{586}{8} = 73.25 \text{ kips/ft (10.68 kN/m)}$$

$$M_u = 73.25\frac{3.08^2}{2} = 347.44 \text{ ft} \cdot \text{kips (471.1 kN} \cdot \text{m)}$$

$$\frac{M_u}{\phi} = \frac{347.44}{0.9} = 386.04 \text{ ft} \cdot \text{kips (523.47 kN} \cdot \text{m)}$$

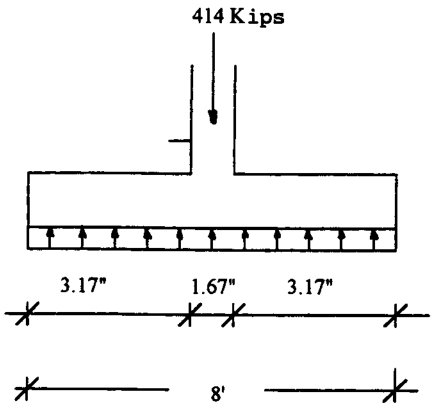
This would require $A_s = 2.5$ in^2 (1613 mm^2).

$$(A_s)_{\min} = 0.0018bh = 0.0018 \times 54 \times 36 = 3.5 \text{ in}^2 \text{ (2258 mm}^2) \qquad \therefore \text{Controls.}$$

Select 6 no. 7 [3.6 in^2 (2322 mm^2)]

b. Transverse steel under exterior column:

$$\text{Beam width} = 20 + \frac{32}{2} = 36 \text{ in (914.4 mm)}$$



$$q_{n_u} = \frac{414}{8} = 51.75 \text{ kips/ft } (7.55 \text{ kN/m})$$

$$M_u = 51.75 \frac{3.17^2}{2} = 260 \text{ ft} \cdot \text{kips } (352.6 \text{ kN} \cdot \text{m})$$

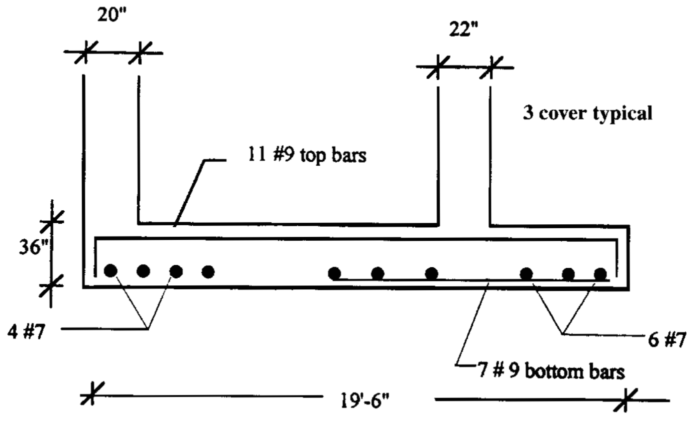
$$\frac{M_u}{\phi} = 289 \text{ ft} \cdot \text{kips } (392 \text{ kN} \cdot \text{m})$$

This would require $A_s = 1.9$ in^2 (1226 mm^2).

$$(A_s)_{\min} = 0.0018bh = 0.0018 \times 36 \times 36 = 2.33 \text{ in}^2 \ (1503 \text{ mm}^2)$$

Select 4 no. 7 [2.4 in^2 (1548 mm^2)].

The checks for the development length and the bearing at the column-footing interface are similar to those in Example 4B.1 and will not be repeated here. The final details of the design are shown here.



4B.8 MAT FOUNDATIONS

4B.8.1 Introduction

A mat foundation consists of a large concrete slab that supports the column of the entire structure (see Fig. 4B.1). It is generally used when the underlying soil has a low bearing capacity. The advantages of using mat foundations are (1) the applied pressure on the supporting soil is reduced because a larger area is used and (2) the bearing capacity of the supporting soil is increased because of the larger foundation depth. Mat foundations can also be used on rock exhibiting irregular compositions, creating weak regions. To overcome the differential settlements that could result from such nonhomogeneous behavior, the mat foundation presents a practical solution. Mat foundations also present an attractive solution to support structures and machinery sensitive to differential settlements.

4B.8.2 Types of Mat Foundations

The most common type of mat foundation is a flat concrete slab of uniform thickness (see Fig. 4B.1). This type provides an economical solution for structures with moderate column loads and uniform and small column spacings. For large column loads the slab thickness is increased beneath the columns to resist resulting shear stresses. If the column spacing becomes large, thickened beams may be used along the column lines. For structures requiring foundations with large flexural rigidity, box structures made of rigid frames or cellular construction are used.

4B.8.3 Design Methods

Different methods for designing mat foundations can be used, depending on the assumptions pertaining to the structure.

4B.8.3.1 Rigid Method

If the mat is rigid enough compared to the subsoil, flexural deflections of the mat will not vary the contact pressure. Hence the contact pressure can be assumed to vary linearly. The line of action of the resultant for the column loads coincides with the centroid of the contact pressure. This assumption is justifiable when the following conditions apply:

1. The column load does not vary by more than 20% compared to adjacent columns.
2. The column spacing is less than 1.75/λ. The coefficient × is defined as

$$\lambda = \sqrt[4]{\frac{K_b b}{4E_c I}} \tag{4B.18}$$

where K_b = coefficient of subgrade reaction
b = width of a strip of mat between centers of adjacent bays
E_c = modulus of elasticity of concrete
I = moment of inertia of strip of width b

On soft soils the actual contact pressure distribution is close to being linear. Hence it is commonly acceptable to design a mat on soft clay or organic soils using the rigid method.

The resultant force of all column loads and its location are first determined. Then the contact pressure q can be calculated using the principles of the strength of materials [Fig. 4B.8(*a*)],

$$q = \frac{\Sigma Q}{A} \pm \frac{(\Sigma Q \cdot e_y)x}{I_y} \pm \frac{(\Sigma Q \cdot e_x)y}{I_x} \tag{4B.19}$$

where ΣQ = resultant force of all column loads
A = total area of mat
e_x, e_y = coordinates determining location of resultant force
x, y = coordinates for a given point under mat
I_x, I_y = moments of inertia of mat with regard to × andy axes

The mat could then be analyzed in each of the two perpendicular directions. As an example, the total shear force acting on section *a-a* is equal to the algebraic sum of the column loads P_1, P_2, and P_3 and the contact pressure reaction on the tributary area $R_{a\text{-}a}$ [Fig. 4B.8(*b*)],

$$V = P_1 + P_2 + P_3 - R_{a\text{-}a} \tag{4B.20}$$

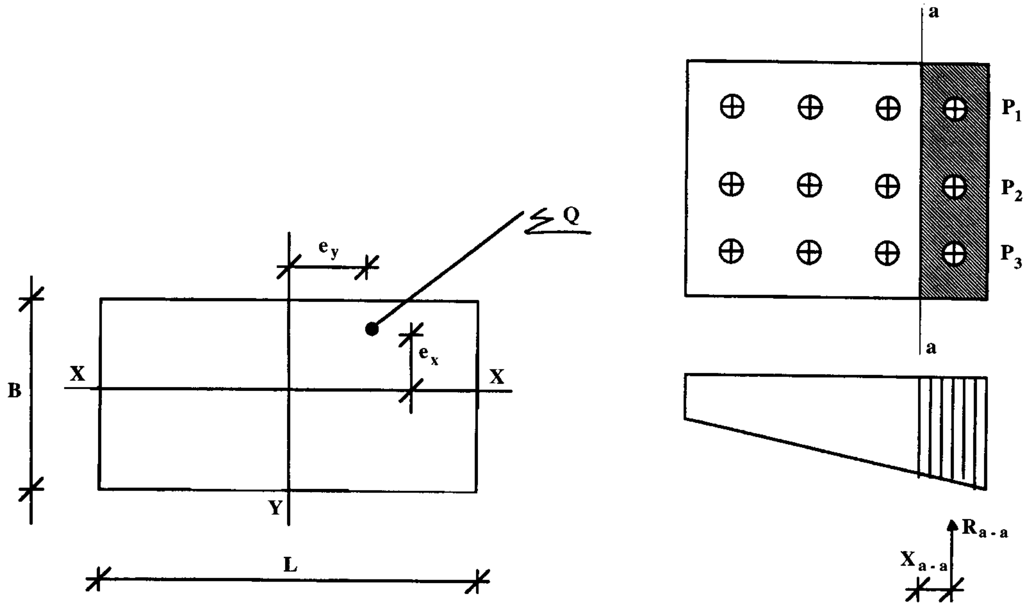


FIGURE 4B.8 Analysis of mat foundations using the rigid method

Similarly the total bending moment acting on section a-a is equal to

$$M = \sum_{i=1}^{n} P_i x_i - (R_{a\text{-}a} x_{a\text{-}a}) \tag{4B.21}$$

The rigid mat should be checked for shear and bending failures. The designer must calculate the shear force at each section and the punching shear under each column and provide an adequate mat thickness. Flexural reinforcement is provided on the top and bottom of the raft foundation in order to guarantee adequate resistance to applied moments.

4B.8.3.2 Elastic Method

This method is based on the theory of plates on elastic foundations (Hetenyi, 1946). For a typical mat on stiff or compact soils it has been found that the effect of a concentrated load damped out quickly. By determining the effect of a column load on the surrounding area, and by superimposing the effects of all the column loads within the influence area, the total effect at any point can be determined. The influence area is usually considered no more than two bays in all directions. The use of polar coordinates is necessary when applying this method since the effect of loads is transferred through the mat to the soil in a radial direction. The application of this method involves extensive mathematical manipulations. Tables have been developed to speed up the solutions. However, for cases involving variable moments of inertia of the mat and possibly variable coefficients of subgrade reaction, the work to be performed remains tedious.

ACI Committee 436 (1966), based on the theory of plates on elastic foundations, recommends the following procedure to design mat foundations of constant moment of inertia and constant coefficient of subgrade reaction.

1. The mat thickness h is chosen such that shear at critical sections is adequately resisted.
2. The coefficient K of subgrade reaction is determined.
3. The flexural rigidity of the mat foundation is calculated using

$$D = \frac{Eh^3}{12(1 - \mu^2)} \tag{4B.22}$$

where E = modulus of elasticity of concrete
μ = Poisson's ratio of concrete

4. The radius of effective stiffness l is determined using

$$l = \sqrt[4]{\frac{D}{K_b}} \tag{4B.23}$$

where K_b is the coefficient of subgrade reaction adjusted for mat size.

5. Radial moment M_r, tangential moment M_t, and deflection Δ at any point are calculated:

$$M_r = -\frac{P}{4}\left[Z_4\left(\frac{r}{l}\right) - (1 - \mu)\frac{Z_3'(r/l)}{(r/l)}\right] \tag{4B.24}$$

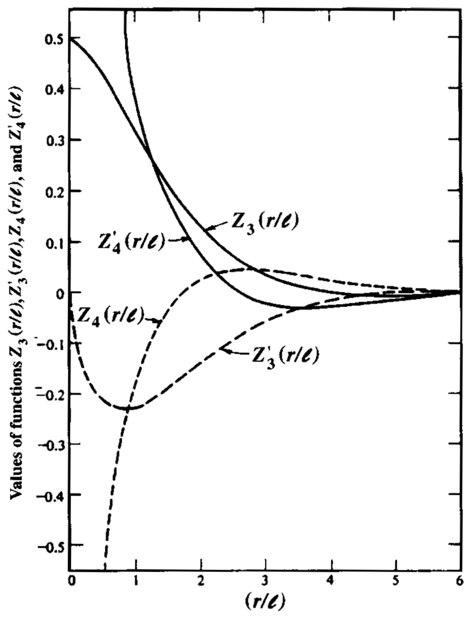


FIGURE 4B.9 Functions for mat foundation design using the elastic method. (*From Hetenyi, 1946.*)

$$M_t = -\frac{P}{4}\left[\mu Z_4\left(\frac{r}{l}\right) - (1-\mu)\frac{Z_3'(r/l)}{(r/l)}\right] \tag{4B.25}$$

$$\Delta = -\frac{Pl^2}{4D} Z_3\left(\frac{r}{l}\right) \tag{4B.26}$$

where P = column load

r = distance of point of interest from column load along radius l

$Z_3\left(\frac{r}{l}\right), Z_3'\left(\frac{r}{l}\right), Z_4\left(\frac{r}{l}\right)$ = functions for moments and deflections (Fig. 4B.9)

6. The radial and tangential moments are transformed to rectangular coordinates,

$$M_x = M_r \cos^2\phi + M_t \sin^2\phi \tag{4B.27}$$

$$M_y = M_r \sin^2\phi + M_t \cos^2\phi \tag{4B.28}$$

where $\tan \phi = y/x$.

7. The shear force Q for a unit width of the mat foundation is determined by

$$Q = -\frac{P}{4l} Z_4'\left(\frac{l}{r}\right) \tag{4B.29}$$

where $Z'_4(l/r)$ is the function for shear (Fig. 4B.10).

8. The moments and shear forces computed for each column are superimposed to obtain the total moment on shear design values.

4B.8.3.3 Numerical Methods

With the increasing use of computers in design applications, numerical methods capable of handling cases of variable moments of inertia and variable coefficients of subgrade reaction are becoming more and more attractive. The methods of finite difference and of finite elements are among the mostly used numerical techniques.

The finite-difference method is based on the assumption that the effect of the underlying soil can be represented by uniformly distributed elastic springs. These springs have an elastic constant K equal to the subgrade reaction. The differential equation of such a mat foundation is

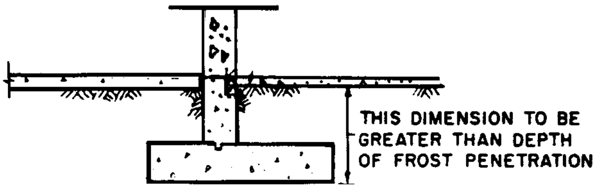


FIGURE 4B.10 Depth of frost penetration.

$$\frac{\delta^4\Delta}{\delta^4 x} + \frac{2\delta^4\Delta}{\delta^4 x\delta^4 y} + \frac{\delta^4\Delta}{\delta^4 y} = \frac{q - Kw}{D} \tag{4B.30}$$

where q = subgrade reaction per unit area of mat
K = coefficient of subgrade reaction
w = deflection
D = rigidity of mat defined in Eq. (4B.22)

The deflection of any point can be related to the deflection at the adjacent points to the right, left, top, and bottom using a numerical difference equation. The mat foundation is divided into a network of points. The difference equations for these points are formulated and rapidly solved for the deflections with a programmed computer. With the knowledge of deflections, the bending moments and shear forces can be determined from the theory of elasticity. The accuracy of the results obtained with the finite-difference method largely depends on the size and number of networks used.

The finite-element method uses the concept of matrix structural analysis to address the problem of plates on elastic foundations. The mat foundation is idealized as a mesh of plates (finite elements) interconnected only at the nodes, where isolated springs are used to model the soil reactions. A more detailed discussion of this method and its applications for foundation design can be found in Weaver and Johnston (1984).

4B.9 FOUNDATIONS IN COLD REGIONS

A locality, city, or state that spends a large amount of its financial resources to maintain a program for continuous social and economical operations under cold weather conditions and snow storms is considered located in a cold region. Seasonal and permanently frozen grounds are characteristics of cold regions and require special attention from the foundations designer.

In areas of seasonal frost during winter months, the foundation depth is carefully taken below the frost line (Fig. 4B.10). This is a necessary measure to prevent heaving of the structure due to freezing of the underlying soil. Heave is a phenomenon caused by the formation and growth of ice particles in the soil. If a foundation is placed at or above the frost line, it will move upward as the underlying soil freezes and expands. Later it will suddenly settle when thawing occurs. An additional problem is encountered in the case of fine-grained soils, namely, the decrease in the soil shear strength when it thaws after being frozen. This loss of strength is due to thawing, liberating moisture that had been soaked up by the soil particles during freezing. Thus the moisture content of the soil is increased compared to conditions prior to freezing. Such a loss of shear strength could result in a foundation failure.

An estimate of the depth of the frost line in different regions can be obtained from data supplied by the U.S. Weather Bureau (see Fig. 4B.11). The depth values obtained from such charts are only approximate. They should be corrected to account for several factors, such as susceptibility of the soil type to frost, location of the footing (interior versus exterior), and local experience (local regulations and adjacent buildings).

For frost action to occur, the following conditions must apply:

1. *Presence of frost-susceptible soil*. These are soils with enough fine pores to initiate and enhance the mechanism of ice formation and growth. Several criteria have been proposed, based on the particle-size distribution of the soil. One of the most widely known of these criteria was proposed by Casagrande (1932):

> Under natural conditions and with sufficient water supply, one should expect considerable ice segregation in uniform soils containing more than 3% of grains smaller than 0.2 mm and in very uniform soils containing more than 10% smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1% of grains smaller than 0.02 mm, even if the groundwater level was as high as the frost line.

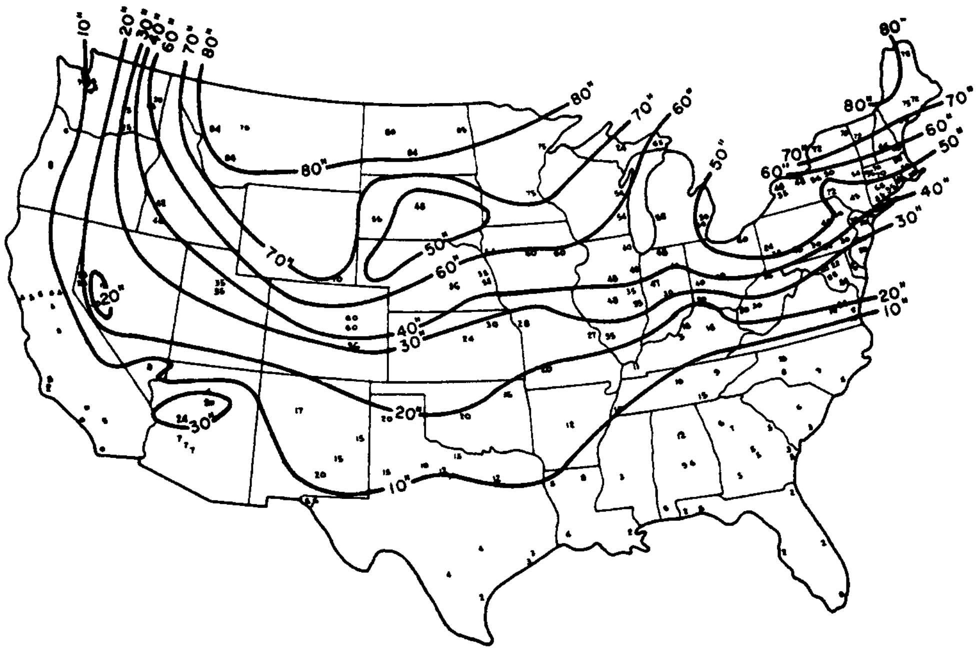


FIGURE 4B.11 Frost penetration map.

A definite distinction between soils that are frost-susceptible and those that are not is not available. Thus soils that are borderline should be used with caution.

2. *Availability of water*. For the ice particles to grow, water in the liquid phase must move in the soil to the frost line. This movement is carried by the capillary action and by suction due to supercooling at the frost front.
3. *Freezing conditions*. These conditions are determined by air temperature, solar radiation, snow cover, and exposure to wind.

In permanently frozen regions (permafrost areas), the loads of the structure are transmitted to the frozen soil with utmost attention to maintaining the frozen state. This is usually performed by insulating and ventilating between the building and the frozen ground such that the presence of the building will not alter the temperature of the ground. Another possible solution is to excavate the soil down to foundation depth and then replace it with soil that is not susceptible to frost action. Thus the foundations will not be affected by the freezing and thawing cycles. In some cases foundations are allowed to bear on frozen ground with a source of artificial refrigeration provided to keep the soil under the footings permanently frozen. This approach is, however, used rarely because of its high cost.

In general the same foundation types used in moderate regions can be used in cold regions, such as spread footings, mat foundations, piles, and caissons. The selection of a specific type of foundation will depend on the particular site conditions, particularly soil type, temperature characteristics, and structural loads. Detailed discussions of the mechanical properties of frozen soil and its bearing capacity are presented in the *Canadian Foundation Engineering Manual*, Andersland and Anderson (1978), and Sodhi (1991). The design of foundations in cold regions rarely requires higher-strength materials to resist the stresses induced from the frost-susceptible soils. What is necessary instead are techniques to avoid problems of frost heaving.

4B.10 FOUNDATIONS IN EARTHQUAKE REGIONS

4B.10.1 General

Earthquakes can produce extensive damage to foundations and structures supported on them. This damage could be related to a gross instability of the soil or to ground movement developing high-intensity stress on the structural systems. Instability of the soil can occur in loose dry sand deposits which are compacted by the ground vibrations of earthquakes, leading to large settlements and differential settlements of the ground surface. The settlements are larger for sands with smaller relative density. In cases where the soil consists of saturated loose sand, the compaction by ground vibrations could increase the hydrostatic pressure to a sufficient magnitude to cause "liquefaction" of the soil. Liquefaction is a phenomenon whereby saturated loose granular soil loses its shear strength due to the earthquake motion. Reports on many earthquakes refer to such liquefaction causing large settlements, tilting, and overturning of structures. Sudden increases in pore water pressures due to ground vibration in deposits of soft clay and sands have been the cause for major landslides in earthquake regions.

Liquefaction is likely to occur under the following soil conditions (Oshaki, 1970):

1. The sand layer is within 45 to 60 ft (15 to 20 m) of the ground level and is not subjected to high overburden pressure.
2. The sand deposits consist of uniform medium-size particles and are below the groundwater level (saturated).
3. The standard penetration test is below a certain value.

To reduce the possibility of liquefaction, several measures can be taken:

1. Increasing the sand relative density by compaction
2. Replacing the sand with another soil having better characteristics to withstand liquefaction
3. Lowering the ground water level or installing drainage equipment

4B.10.2 Dynamic Properties of Soils

In order to perform a seismic design for foundations in an earthquake region, the dynamic soil characteristics must be determined. The following is a brief description of tests used to obtain such data (Wakabayashi, 1986).

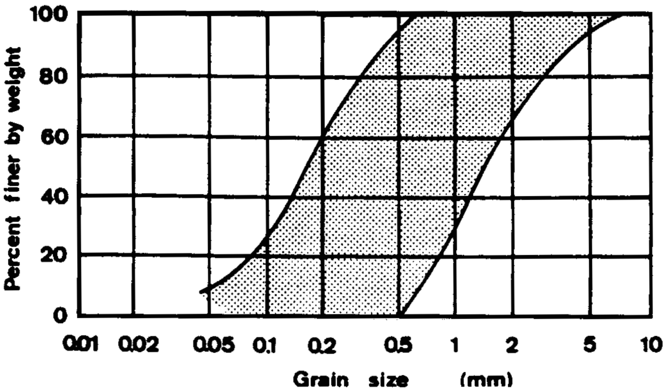


FIGURE 4B.12 Zone of liquefaction potential for cohesionless soils. (*From Ohsaki, 1970.*)

4B.10.2.1 Particle Size Distribution
The soil particle size distribution is related to the liquefaction of saturated cohesionless soils. Figure 4B.12 indicates a liquefaction potential zone based on the performance of cohesionless soils in previous earthquakes.

4B.10.2.2 Relative Density Test
This test indicates the degree of soil compaction. It gives helpful information in determining the possibility of excessive settlement for dry sands and the potential of liquefaction for saturated sands in earthquake regions. The relative density is obtained from one of the equations

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$$D_r = \frac{\rho_{max}(\rho - \rho_{min})}{\rho(\rho_{max} - \rho_{min})} \qquad (4B.31)$$

where e_{max}, e_{min} = maximum and minimum void ratios
ρ_{max}, ρ_{min} = maximum and minimum unit mass
e = in situ void ratio
ρ = in situ unit mass

4B.10.2.3 Cyclic Triaxial Test
This test is performed to determine the shear modulus and damping of cohesive and cohesionless soils. The shear modulus can be obtained from the compressive modulus of elasticity E using

$$G = \frac{E}{2(1 + v)} \qquad (4B.32)$$

where v is Poisson's ratio.

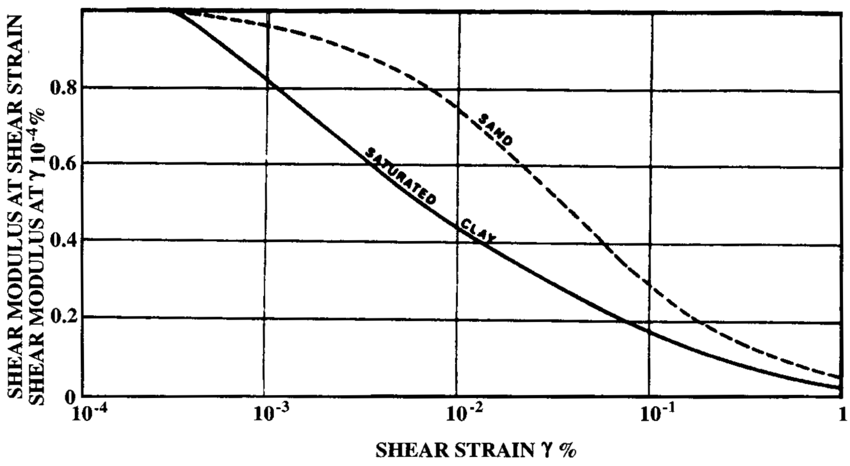


FIGURE 4B.13 Average relationships of shear modulus to strain. (*From Seed and Idrisa, 1970.*)

It can also be obtained directly by performing cyclic shear tests to obtain stress-strain relationships. The shear modulus is strain-dependent. Hence the level at which G is determined must be defined. Average relationships of shear modulus to strain for clay and sand are shown in Fig. 4B.13. During earthquakes, developed shear strains may range from 10^{-3} to 10^{-1}%, with a different maximum strain at each cycle. For this reason it has been suggested to use a value of two-thirds the shear modulus measured at the maximum strain developed for earthquake design purposes. In the field, the shear modulus of soil can be estimated from a shear wave velocity test. An explosive charge or a vibration source is used to initiate waves in the soil. The velocity of these waves is measured and the following relationship is used to determine the shear modulus of elasticity:

$$G = \rho v_s^2 \tag{4B.33}$$

where ρ = mass density of soil

v_s = shear wave velocity

The second chief dynamic parameter for soils is damping. Two different damping phenomena are related to soils—material damping and radiation damping. Material damping takes place when any vibration wave travels through the soil. It is related to the loss of vibration energy resulting from hysteresis in the soil. Damping is generally expressed as a fraction of critical damping and thus referred to as damping ratio. The damping ratio is expressed as (Fig. 4B.14)

$$\varepsilon = \frac{W}{4\pi\Delta W} \tag{4B.34}$$

where W = energy loss per cycle (area of hysteresis loop)

ΔW = strain energy stored in equivalent elastic material (area OAB in Fig. 4B.14)

Typical material damping ratios, representing average values of laboratory test results on sands and saturated clays, are give in Fig. 4B.15.

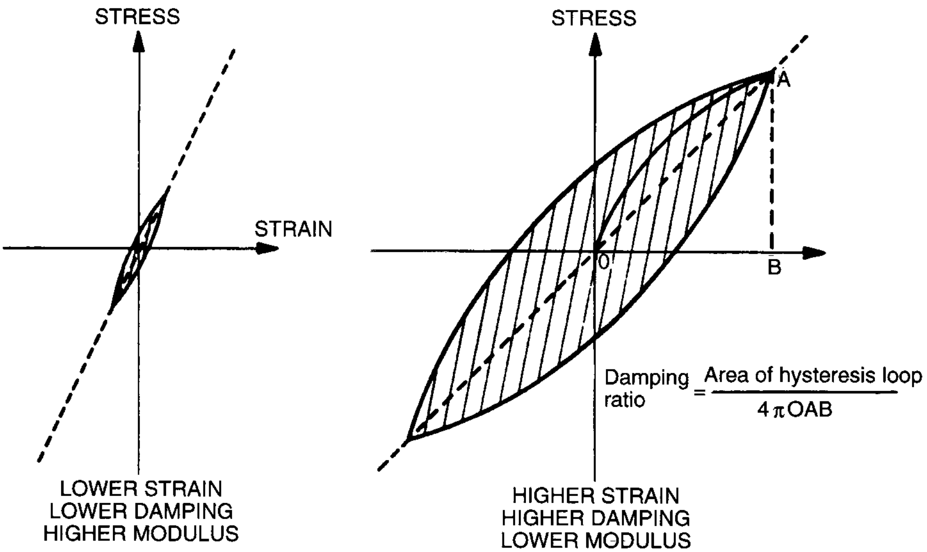


FIGURE 4B.14 Calculation of material damping ratio. (*From Seed and Idriss, 1970.*)

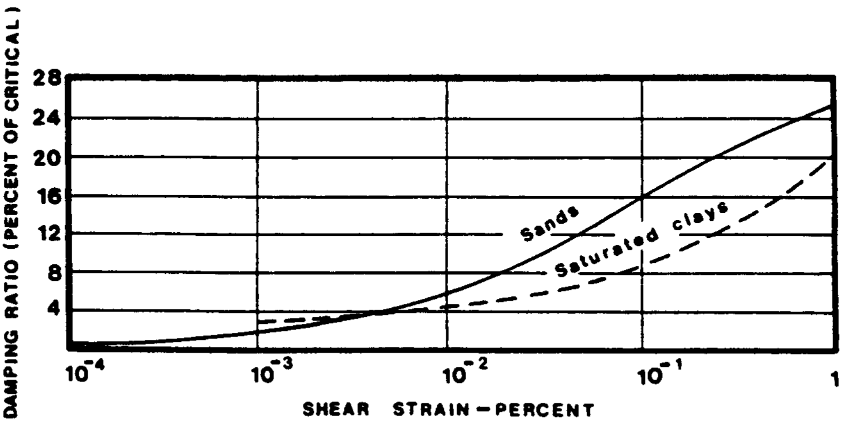


FIGURE 4B.15 Typical material damping ratios. (*From Seed and Idriss, 1970.*)

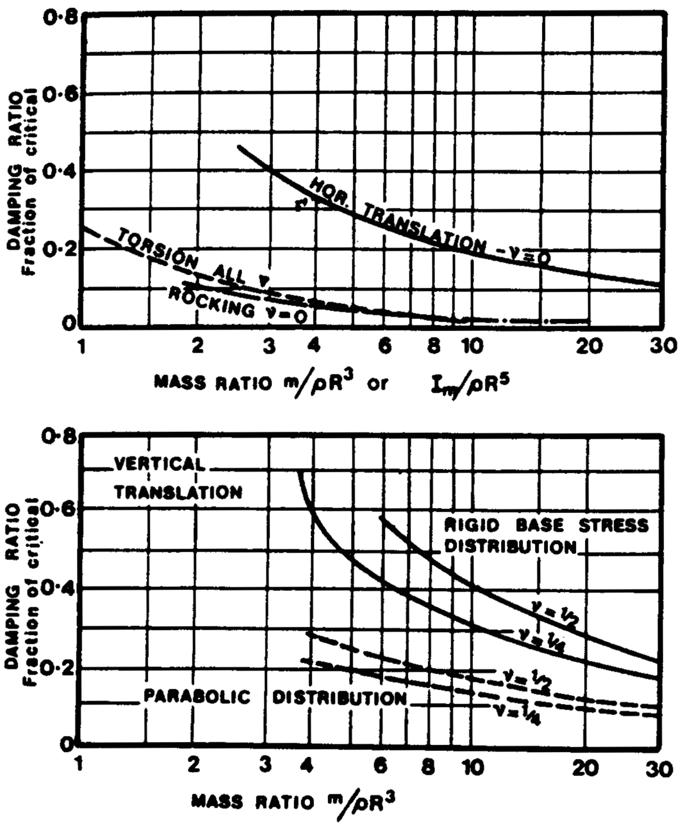


FIGURE 4B.16 Radiation damping values. (*From Whitman and Richart, 1967.*)

Radiation damping is a measure of the loss of energy through the radiation of waves from the structure. It is related to the geometrical properties of the foundation. The theory for the elastic half-space has been used to provide estimates for radiation damping. Figure 4B.16 shows values of radiation damping for circular footings of machinery obtained by Whitman and Richart (1967).

4B.10.3 Design Considerations

Additional design considerations for foundations in earthquake regions include (1) transmission of horizontal base shear forces from the structure to the soil, (2) resisting the earthquake-induced overturing moments, and (3) differential settlements and liquefaction of the subsoil (Wakabayashi, 1986; Dorwick, 1987).

Generally in earthquake design practice two separate stress systems are considered—seismic vertical stress resulting from overturning moments and seismic horizontal stresses caused by the base shear on the structure. Unless it is very slender, overturuing moments are not a design problem for the structure as a whole. However, they can drastically impact on individual footings. Hence the foundation should be proportioned so as to maintain the maximum bearing pressures caused by the overturning moments and gravity loads within the allowable seismic bearing capacity of the present soil. Safe seismic bearing pressures vary from one location to another, and local codes should be used for guidelines. In general most soils are capable of resisting higher short-term loads than long-term loads. Some sensitive clays that lose strength under dynamic loading are an exception.

With shallow foundations the base shear is assumed to be resisted by friction on the bottom surfaces of footings. The total resistance to horizontal displacement of a structure is taken to be equal to the product of the dead load of the structure and the coefficient of friction between soil and footings. Some codes recommend the use of 75% of the standard friction coefficients. Additional horizontal resistance can be obtained from the passive soil pressures developed against footing surfaces. However, if this resistance is to be relied upon, reducing the computed total resistance becomes necessary. This can be done by reducing either the frictional force or the passive resistance by about 50%. Also, careful compaction of the backfill against the sides of the footing must be performed in order to rely on the passive restraint of the soil.

To avoid or minimize damage to the foundation structure in earthquake regions due to differential settlements, it is recommended to provide ties or beams between column footings. These ties should be designed to withstand a prescribed differential movement between the connected footings.

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SECTION 4C
PILE FOUNDATIONS

A. SAMER EZELDIN

4C.1 INTRODUCTION

Piles are vertical or slightly inclined members used to transmit the loads of the superstructure to lower layers in the soil mass. The load transfer mechanism relies either on the skin resistance occurring along the surface contact of the pile with the soil or on the end bearing on a dense or firm layer. The design of some piles can also be based on the utilization of both the skin resistance and the end bearing to carry the applied load jointly. In general, pile foundations are relied upon to transfer the load acting on the superstructures in situations where the use of shallow foundations becomes inadequate or unreliable. Such situations include (1) the top soil layers have a weak bearing capacity, with the soil layer at greater depth possessing a high bearing capacity; (2) large values of concentrated loads are to be transmitted from the superstructure to the foundation; and (3) the structure to be designed is very sensitive to unequal settlements.

Materials usually used to make piles are concrete, steel, and timber. The upper part of the pile connected to the superstructure is referred to as the pile head. The middle part is called the shaft, the lower is the pile tip. The pile cross section can either be maintained throughout the length of the pile or it can be tapered to a rather pointed pile. The cross section can be circular, octagonal, hexagonal, square, triangular, or H-shaped. Figure 4C.1 illustrates typical pile shapes and various cross sections.

Piles can be classified into two types—displacement piles and nondisplacement piles. Displacement piles are those which displace the soil to allow for the pile penetration. These piles can be of solid cross section, driven into the ground, and left in position. Timber, steel, prestressed concrete piles, and precast concrete piles are of this type. Displacement piles are also obtained by driving shell (hollow) piles by means of an internal steel mandrel onto which the shell is threaded. After the mandrel is pulled out, the shell pile is filled with concrete internally. The Raymond pile is a mandrel-driven steel-shell pile; the Western pile is a mandrel-driven concrete-shell pile. Another method for obtaining displacement piles is driving a pilelike body into the ground and withdrawing it while filling the void with concrete (Franki pile). Nondisplacement piles are those in which the soil is removed to accommodate the pile. Typically a borehole is formed in the ground, then concrete is cast in place in the hole.

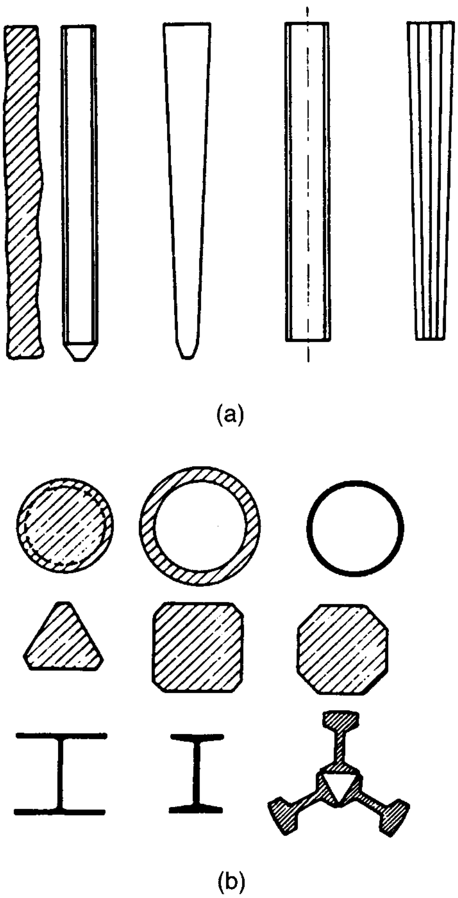


FIGURE 4C.1 Typical pile shapes and cross sections. (a) Piles shapes. (b) Various pile cross sections.

4C.2 ALLOWABLE STRESSES IN PILES

In pile foundation design it is necessary to determine the required number of piles, their cross section, and their length. This will require knowledge about the pile loading capacity as well as their allowable stresses. This section will present the allowable design stresses for service loads as adopted in the design guides of the U.S. Army Corps of Engineers and published by the American Society of Civil Engineers (ASCE, 1993). The pile loading capacity is discussed in Sec. 4C.3. The allowable stresses presented in this section may be increased by one-third to account for unusual loading such as maintenance, infrequent floods, barge impact, construction, or hurricanes.

4C.2.1 Concrete Piles

Concrete piles can be prestressed, precast-reinforced, cast in place, or mandrel-driven.

4C.2.1.1 Prestressed Concrete Piles

Prestressed concrete piles are formed by tensioning high-strength steel cable having an ultimate stress f_{pu} of 250 to 270 ksi with a prestress of about 0.5 to $0.7f_{pu}$ before casting the concrete. Af-

TABLE 4C.1 Allowable Concrete Stresses for Prestressed Concrete Piles

Uniform axial tension	0
Bending (extreme fiber)	
Compression	$0.40f'_c$
Tension	0

Source: ASCE, 1993.

ter casting the concrete, and only when it develops adequate strength, the prestress cables are cut. Due to the bond between steel and concrete, the cables will apply a compressive stress on the concrete pile as they attempt to return to their original length. When designing prestressed concrete piles, both strength and serviceability requirements must be satisfied. Strength design should be conducted in accordance with the American Concrete Institute code (ACI, 1989), except that a strength reduction factor ϕ of 0.7 is to be used for all failure modes and a load factor of 1.9 for both dead and live loads. The use of these factors will result in a factor of safety of 2.7 for all dead and live load combinations. The axial strength to be used in design is the least of: (1) 80% of the concentric axial strength or (2) the axial strength corresponding to an eccentricity equal to 10% of the pile diameter or width. Cracking control is achieved by limiting the actual concrete compressive and tensile stresses resulting from working conditions to the values presented in Table 4C.1. For the combined condition of axial force and bending, the concrete stresses should satisfy the following:

$$f_a + f_b + f_{pc} \leq 0.4f'_c \tag{4C.la}$$

$$f_a - f_b + f_{pc} \geq 0 \tag{4C.lb}$$

where f_a = computed axial stress (tensions negative)
f_b = computed bending stress (tensions negative)
f_{pc} = effective prestress
f'_c = concrete compressive strength

The allowable stresses for hydraulic structures are limited to 0.85% of the values recommended by ACI Committee 543 for improved serviceability (ACI, 1986). Permissible stresses in the prestressing steel cables should be in accordance with the ACI code requirements (ACI, 1989). In cases where the pile is free-standing or when the soil is too weak to provide a reliable lateral support, the pile capacity should be reduced due to slenderness effects. The moment magnification method of ACI as modified by PCI can be used to perform such design (PCI, 1988). Figure 4C.2 illustrates typical prestressed concrete piles.

4C.2.1.2 Precast-Reinforced Concrete Piles

Precast-reinforced concrete piles are designed in accordance with the ACI code (ACI, 1989). For hydraulic structures, the ultimate load is to be increased by a hydraulic load factor H_f. The hydraulic load factor is taken as (1) 1.30 for reinforcement calculations in flexure or compression, (2) 1.65 for reinforcement in direct tension, and (3) 1.30 for reinforcement in shear. When performing shear reinforcement design, the calculations should exclude the shear carried by the concrete prior to application of the hydraulic load factor. The axial strength limitations are taken as in the case for prestressed piles. The slenderness effects are accounted for according to the ACI code moment magnification method (ACI, 1989). Figure 4C.3 shows typical precast-reinforced concrete piles.

4C.2.1.3 Cast-in-Place and Mandrel-Driven Piles

Figure 4C.4 illustrates various types of cast-in-place piles. The depths indicated are for the usual ranges for the different piles. These piles are mostly used when continuous lateral support is present. Cast-in-place and mandrel-driven piles are designed such that working stresses are limited to the al-

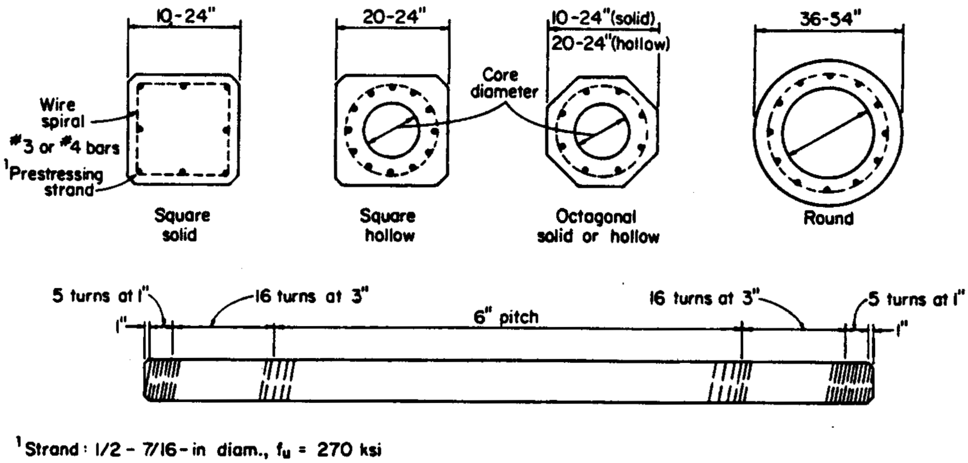
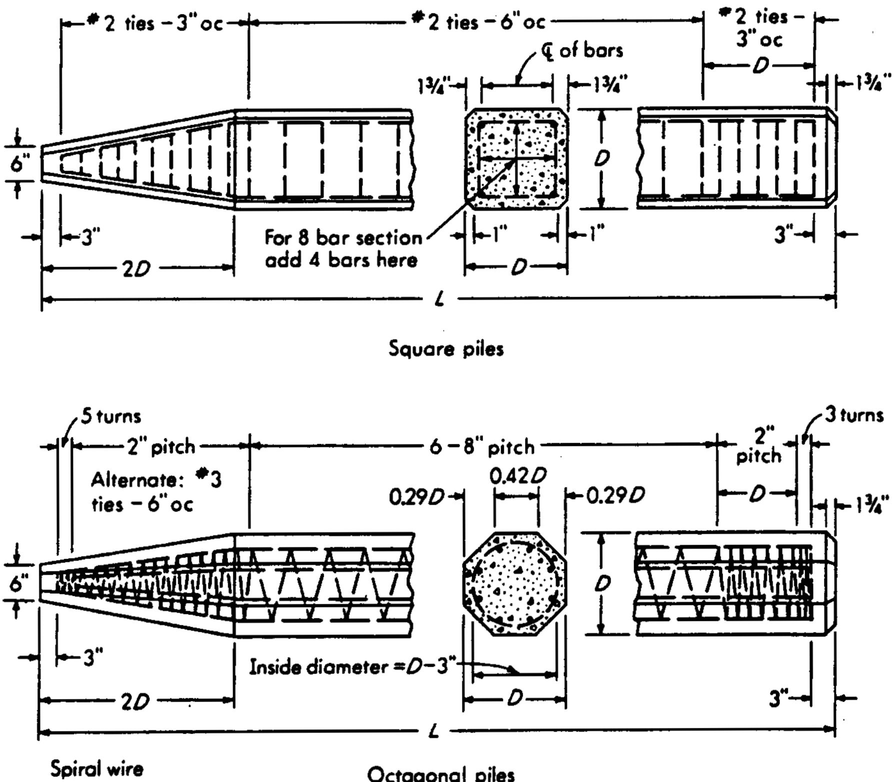


FIGURE 4C.2 Typical prestressed concrete pile. (*From Bowles, 1982.*)



D, in.	16	20	24
Size	#5	#4	#3

FIGURE 4C.3 Typical precast concrete pile. (*From Bowles, 1982.*)

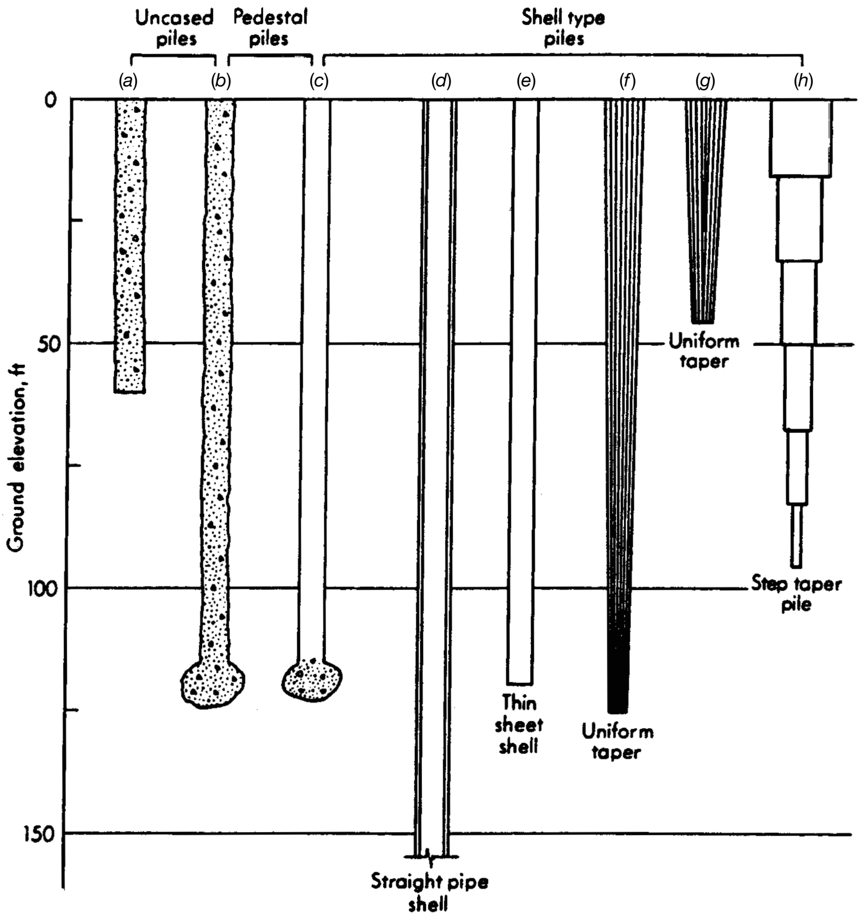


FIGURE 4C.4 Typical cast-in-place concrete pile. (*a*) Western uncased pile. (*b*) Franki uncased-pedestal pile. (*c*) Franki cased-pedestal pile. (*d*) Welded or seamless pile. (*e*) Western cased pile. (*f*) Union or Monotube pile. (*g*) Raymond standard. (*h*) Raymond step-taper pile. (*From Bowles, 1982.*)

lowable stresses shown in Table 4C.2. In case of axial load combined with bending, the concrete stresses are such that

$$\left| \frac{f_a}{F_a} + \frac{f_b}{F_b} \right| \leq 1.0 \qquad \text{(4C.2)}$$

where f_a = computed axial stress
F_a = allowable axial stress
f_b = computed bending stress
F_b = allowable bending stress

TABLE 4C.2 Allowable Concrete Stresses for Cast-in-Place and Mandrel-Driven Piles

Uniform axial compression	
Confined	$0.33f'_c$
Unconfined	$0.27f'_c$
Uniform axial tension	0
Bending (extreme fiber)	
Compression	$0.40f'_c$
Tension	0

Source: ASCE, 1993.

4C.2.2 Steel Piles

Steel piles are usually rolled, H-shaped, or pipe piles. The lower region of these piles could be subjected to damage during driving. This is why the U.S. Army Corps of Engineers uses allowable stresses with a high factor of safety for that region, as shown in Fig. 4C.5. Pile shoes are usually used when driving piles in the dense layer. Table 4C.3 shows the allowable stresses for fully supported piles when using pile shoes. The upper portion of the pile is designed as a beam-column where

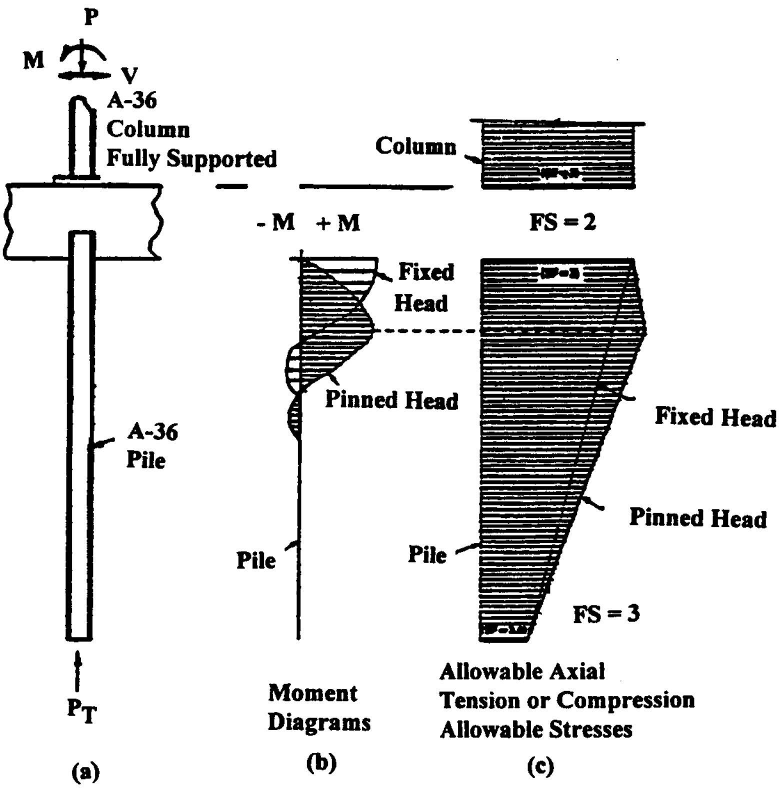


FIGURE 4C.5 Behavior of steel piles. (*From ASCE, 1993.*)

TABLE 4C.3 Allowable Stresses in Lower Pile Region for Steel Piles

Concentric axial tension or compression only 10 ksi ($\frac{1}{3} \times F_y \frac{5}{6}$)	10 ksi for A-36 material
Concentric axial tension or compression only with driving shoes ($\frac{1}{3} \times F_y$)	12 ksi for A-36 material
Concentric axial tension or compression only with driving shoes, at least one axial load test and use of a pile driving analyzer to verify pile capacity and integrity ($1/2.5 \times F_y$)	14.5 ksi for A-36 material

Source: ASCE, 1993.

the lateral support conditions are accounted for. Bending moments are, however, negligible in the lower portion of the pile. The moment diagram along the pile is shown in Fig. 4C.5. For laterally unsupported piles the allowable stress should be five-sixths of the values the American Institute of Steel Construction code gives for beam columns (AISC, 1989). For combined axial compression and bending conditions, the stress should be

$$\left| \frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \right| \leq 1.0 \qquad \text{(4C.3)}$$

where f_a = computed axial stress
F_a = allowable axial stress, = $0.5F_y$
f_{bx}, f_{by} = computed bending stress
F_b = allowable bending stress, 0.5 F_y for noncompact section and $\frac{5}{9}$ F_y for compact section

3.D.2.3 Timber Piles

Timber piles are cut from tree trunks and driven with the smaller cross section down. Representative allowable stress values for pressure-treated round timber piles are presented in Table 4C.4. These stresses have been adjusted to account for treatment. For untreated piles, or piles that were either air- or kiln-dried before pressure treatment, the allowable stress shown in Table 4C.4 should be increased by dividing each value by 0.9 for Pacific Coast Douglas fir and by 0.85 for southern pine. To account for combined axial load and bending moment effects, the stresses should satisfy

$$\left| \frac{f_a}{F_a} + \frac{f_b}{F_b} \right| \leq 1.0 \qquad \text{(4C.4)}$$

TABLE 4C.4 Allowable Stresses for Pressure-Treated Round Timber Piles

Species	Compression parallel to grain, psi F_a	Bending, psi F_b	Horizontal shear, psi	Compression perpendicular to grain, psi	Modulus of elasticity, psi
Pacific Coast Douglas fir	875	1700	95	190	1,500,000
Southern pine	825	1650	90	205	1,500,000

Source: ASCE, 1993.

where f_a = computed axial stress
F_a = allowable axial stress
f_b = computed bending stress
F_b = allowable bending stress

4C.3 PILE LOADING CAPACITY

4C.3.1 General

The mechanism of the load transfer from piles to the soil layer is illustrated in Fig. 4C.6. The horizontal earth pressures act on the shaft surface area, creating vertical frictional reactions that increase with depth. If enough displacement occurs, adhesion could also contribute to these reactions. The sum of these reactions is referred to as the mantle friction or skin resistance. In addition vertical reactions occur at the tip of the pile, mobilizing tip-bearing resistance. The ratio of the mantle friction to the tip-bearing resistance varies according to the physical properties and profile of the soil, the pile dimensions, and the method of installation.

4C.3.2 Axial Single-Pile Capacity

The pile loading capacity consists of the sum of the skin resistance and the tip-bearing resistance. Hence it may be represented by the equation

$$Q_{\text{ult}} = Q_s + Q_t = f_s A_s + qA_t \tag{4C.5}$$

where Q_{ult} = ultimate pile capacity
Q_s = shaft resistance of pile due to skin friction
Q_t = tip-bearing resistance of pile
f_s = average skin resistance stress
A_s = surface area of shaft in contact with soil
q = tip-bearing stress
A_t = effective area at tip of pile in contact with soil

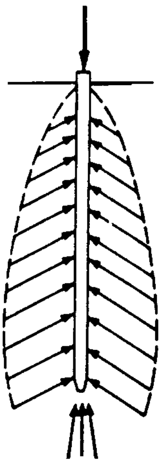


FIGURE 4C.6 Load transfer mechanism using skin friction resistance and tip-bearing resistance.

TABLE 4C.5 Typical K values* for Piles in Compression and in Tension

Soil type	K_c	K_t
Sand	1.00 to 2.00	0.50 to 0.70
Silt	1.00	0.50 to 0.70
Clay	1.00	0.70 to 1.00

*Values do not apply to piles that are prebored, jetted, or installed with a vibratory hammer. Picking K values at the upper end of these ranges should he based on local experience. K, δ, and N_q values hack-calculated from load tests may he used.

Source: ASCE, 1993.

4C.3.2.1 Piles in Cohesionless Soil

The skin resistance of piles in cohesionless soil is assumed to increase linearly up to a critical depth d_c. For design purposes, the critical depth is taken as 10B for loose sand, 15B for medium dense sand, and 20B for dense sand, where B is the pile diameter or width. Below the critical depth, the skin resistance is taken to be a constant value (equal to the critical-depth skin resistance stress). The average skin resistance stress at a particular depth may be calculated using the equation

$$f_s = K\sigma_v' \tan \delta \tag{4C.6}$$

where K = coefficient for lateral earth pressure (see Table 4C.5 for K values for piles in compression and tension)

σ_v' = effective overburden pressure at particular depth d

$= \gamma' d$ for $d < d_c$

$= \gamma' d_c$ for $d < d_c$ using γ' as the effective unit weight for soil

δ = friction angle between soil and pile material (see Table 4C.6 for typical δ values)

It must be emphasized that the K and δ values presented should be selected based on experience and site conditions and could be replaced with better representative values if such are available to the designer. When using steel H piles, the value of δ should be the average friction angle of steel against soil and soil against soil (ϕ value). Also, the value of A_s for steel H piles is to be taken as the block perimeter of the pile.

The tip-bearing stress, q can be determined from the expression

$$q = \sigma_v' N_q \tag{4C.7}$$

where σ_v' is as defined earlier, and the bearing capacity factor N_q is obtained from Fig. 4C.7. When using steel H piles, the area A_t is taken as the area included within the block perimeter. The pile tension capacity in cohesionless soil is obtained by solely calculating the shaft resistance of the pile due to skin friction Q_s using the corresponding K values in Table 4C.5.

TABLE 4C.6 Typical δ Angles in terms of ϕ

Pile material	δ
Steel	0.67 ϕ to 0.83 ϕ
Concrete	0.90 ϕ to 1.0 ϕ
Timber	0.80 ϕ to 1.0 ϕ

Source: ASCE, 1993.

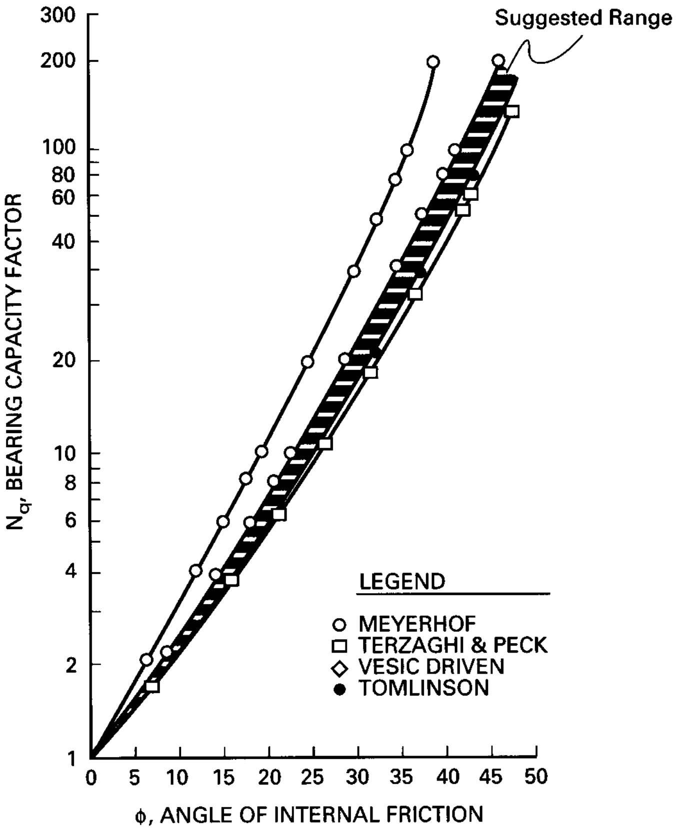


FIGURE 4C.7 Bearing capacity factor N_q versus angle of internal friction ϕ. (*From Terzaghi and Peck, 1967.*)

Example 4C.1 Find the allowable compression capacity of a 12-in-diameter (305 mm) reinforced concrete pile with a total length of 45 ft (13.7 mm) driven in medium dense sand. The K and δ values are found to be 1.5 and 0.9ϕ, respectively. The soil profile is shown in Fig. 4C.X. 1. The soil angle of friction ϕ is 30°. Use a factor of safety of 3.0.

Solution The critical depth is

$$d_c = 15B = 15\left(\frac{12}{12}\right) = 15 \text{ ft } (4.57 \text{ m})$$

The effective overburden pressure σ_v' at water table level is

$$\sigma_v' = 110 \times 10 = 1100 \text{ psf } (53.647 \text{ kPa})$$

The effective overburden pressure σ_v' at the critical depth d_c is

$$\sigma'_{v(-15)} = 1100 + 5(125 - 62.4) = 1413 \text{ psf } (68.91 \text{ kPa})$$

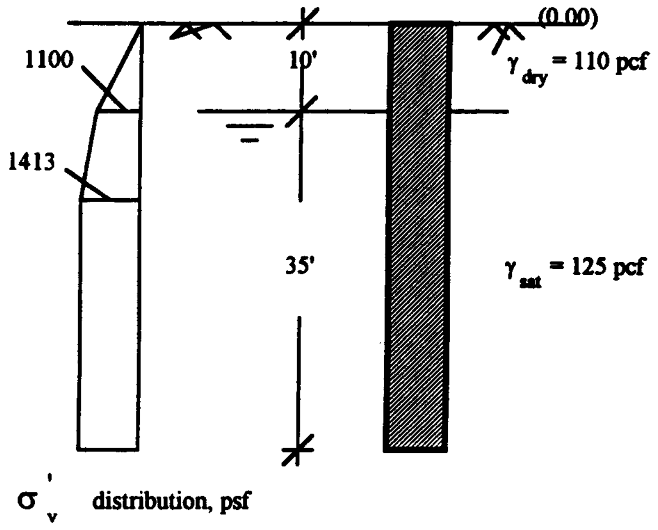


FIGURE 4C.X.1

The shaft resistance due to skin friction from level (0.00) to (–10.00) (3 m) is

$$\left[1.5\left(\frac{1100+0}{2}\right)(\tan 0.9\times 30°)\right]\left[\pi\left(\frac{12}{12}\right)10\right] = 13{,}185\text{ lb }(58.65\text{ kN})$$

The shaft resistance due to skin friction from level (–10.00) (3 m) to (–15.00) (4.6 m) is

$$\left[1.5\left(\frac{1100+1413}{2}\right)(\tan 0.9\times 30°)\right]\left[\pi\left(\frac{12}{12}\right)5\right] = 15{,}061\text{ lb }(66.99\text{ kN})$$

The shaft resistance due to skin friction from level (–15.00) (–4.6 m) to (–45.00) (13.7 m) is

$$[1.5\times 1413(\tan 0.9\times 30°)]\left[\pi\left(\frac{12}{12}\right)30\right] = 101{,}625\text{ lb }(452\text{ kN})$$

The total shaft resistance due to skin friction is

$$Q_s = 13{,}185 + 15{,}061 + 101{,}625 = 129{,}871\text{ lb }(582\text{ kN})$$

The tip-bearing resistance is, using $N_q = 18$,

$$Q_t = \sigma_v' N_q A_t$$

$$= 1413\times 18\times\frac{\pi\times 1^2}{4} = 19{,}965\text{ lb }(88.8\text{ kN})$$

The allowable compression capacity is

$$Q_{\text{all}} = \frac{Q_{\text{ult}}}{FS} = \frac{129{,}871 + 19{,}965}{3} = 49{,}945 \text{ lb} \approx 50 \text{ kips (222 kN)}$$

4C.3.2.2 Piles in Cohesive Soil

The skin resistance is due to adhesion of the cohesive soil to the pile shaft. The following expression can be used to estimate the average skin resistance stress:

$$f_s = \alpha c \tag{4C.8}$$

where c = undrained shear strength of soil from a Q test
α = adhesion factor (see Fig. 4C.8 for values of a in terms of undrained shear strength)

An alternate method proposed by Semple and Rigden (1984) is especially applicable for long piles. It consists of obtaining the adhesion factor α as the product of two factors α_1 and α_2. These two factors can be obtained using Fig. 4C.9.

The tip-bearing stress q is obtained from the expression

$$q = 9c \tag{4C.9}$$

To develop such tip-bearing stress the required pile movement may have to be larger than that necessary to mobilize skin resistance. The pile tension capacity in cohesive soil can be calculated using only the shaft resistance due to skin friction.

Example 4C.2 Find the allowable compression capacity of a 12-in-diameter (305 mm) reinforced concrete pile with a total length of 45 ft (13.7 m) driven in clay layers as shown in Fig. 4C.X.2. Use a factor of safety of 2.5.

Solution The shaft resistance due to skin friction from level (0.00) to (–10.00) [3 m] is

$$1 \times 400\left[\pi\left(\frac{12}{12}\right)10\right] = 12{,}560 \text{ lb (55.9 kN)}$$

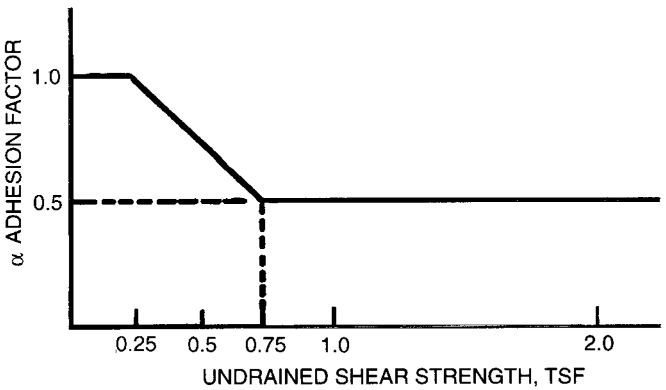


FIGURE 4C.8 Values of α versus undrained shear strength. (*From ASCE, 1993.*)

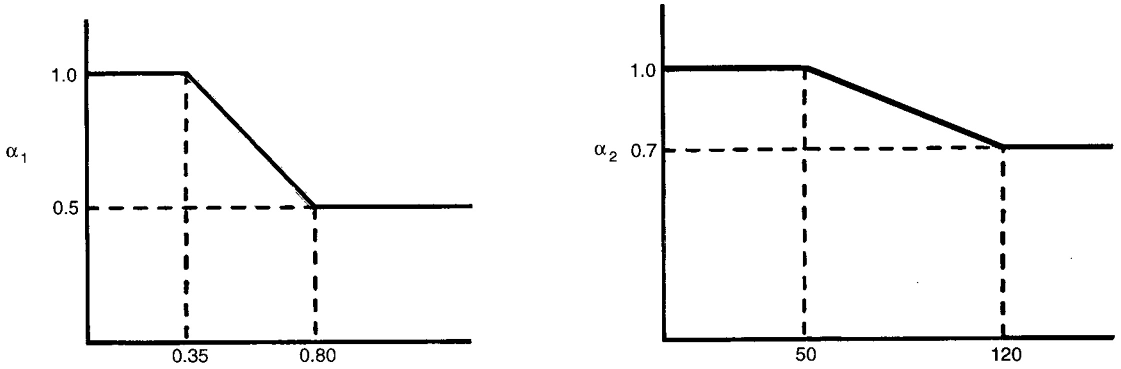


FIGURE 4C.9 Values of α_1 and α_2 applicable for long piles. (*From Semple and Rigden. 1984.*)

The shaft resistance due to skin friction from level (–10.00) [–3 mm] to (–15.00) [–4.5 m] is

$$0.95 \times 600\left[\pi\left(\frac{12}{12}\right)5\right] = 8949 \text{ lb } (39.8 \text{ kN})$$

The shaft resistance due to skin friction from level (–15.00) [–4.5 m] to (–30.00) [–9 m] is

$$0.9 \times 700\left[\pi\left(\frac{12}{12}\right)15\right] = 29{,}673 \text{ lb } (131.98 \text{ kN})$$

The shaft resistance due to skin friction from level (–30.00) [–9 m] to (–45.00) [–13.7 m is

$$0.85 \times 800\left[\pi\left(\frac{12}{12}\right)15\right] = 32{,}028 \text{ lb } (143.5 \text{ kN})$$

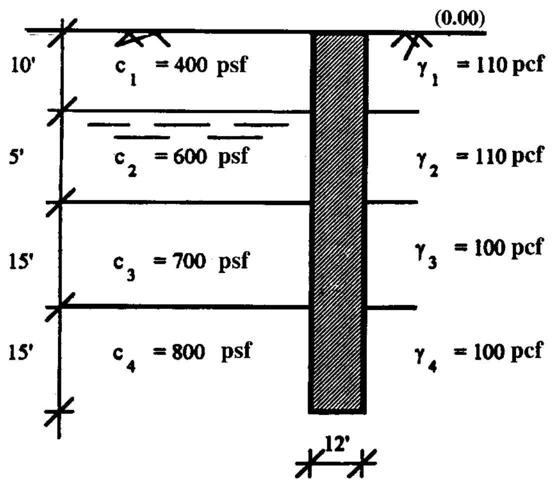


FIGURE 4C.X.2

The total shaft resistance due to skin friction is

$$Q_s = 12{,}560 + 8949 + 29{,}673 + 32{,}028 = 83{,}210 \text{ lb } (370.1 \text{ kN})$$

The tip-bearing resistance is

$$Q_r = 9c\frac{\pi(d)^2}{4} = 9 \times 800\frac{\pi(1)^2}{4} = 5652 \text{ lb } (25.14 \text{ kN})$$

The allowable compression capacity is

$$Q_{\text{all}} = \frac{Q_{\text{ult}}}{FS} = \frac{83{,}210 + 5652}{2.5} = 35{,}545 \text{ lb} \approx 36 \text{ kips } (158.1 \text{ kN})$$

4C.3.2.3 Piles in Silt

The skin resistance of piles in silt is generated from two sources—friction along the pile shaft and adhesion of soil to the pile shaft. The portion of the friction resistance increases linearly up to the critical depth d_c, below which the frictional resistance remains constant. In design, the critical depth is assumed as $10B$ for loose silts, $15B$ for medium silts, and $20B$ for dense silts, where B is the pile diameter or width. The portion of the adhesion is controlled by the undrained shear strength of the soil. The combined average skin resistance stress f_s, can be determined using the equation

$$f_s = K\sigma_v' \tan\delta + \alpha c \tag{4C.10}$$

where all variables are as defined in Secs. 4C.3.2.1 and 4C.3.2.2.

The tip-bearing stress q can be calculated from Eq. (4C.7). The pile tension capacity in silt soil is obtained by excluding the tip-bearing stress calculation and including only the effect of the skin resistance stress f_s along the pile shaft.

Example 4C.3 Determine the allowable compression capacity of a 12-in-diameter (304 mm) reinforced concrete pile with a total length of 50 ft (15 m) driven in medium dense silt. The K and δ values are found to be 1.0 and 1.0ϕ, respectively. The soil angle of friction ϕ is 20°, and its undrained shear strength c is 200 psf (9.5 kPa). The soil profile is shown in Fig. 4C.X.3. Use a factor of safety of 3.0.

Solution The critical depth is

$$dc = 15B = 15\left(\frac{12}{12}\right) = 15 \text{ ft } (4.57 \text{ m})$$

The effective overburden pressure σ_v' at water table level is

$$\sigma'_{v(-b)} = 110 \times 10 = 1100 \text{ psf } (52.67 \text{ kPa})$$

The effective overburden pressure σ_v' at the critical depth d_c is

$$\sigma'_{v(-15)} = 1100 + 5(110 - 62.4) = 1334 \text{ psf } (63.87 \text{ kPa})$$

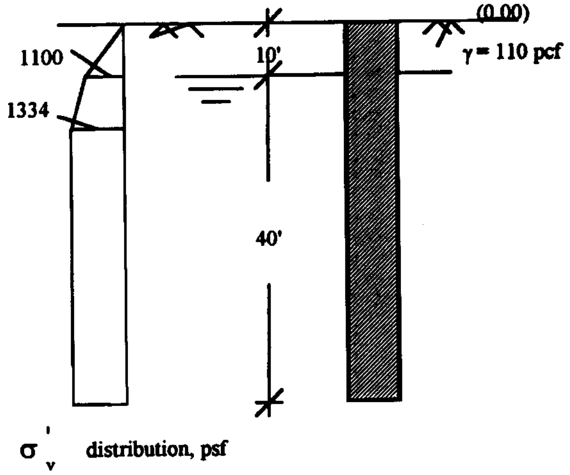


FIGURE 4C.X.3

The shaft resistance due to skin friction from level (0.00) to (–10.00) [3 m] is

$$\left[\left(\frac{1100+0}{2}\right)\tan 20° + 200\right]\left[\pi\left(\frac{12}{12}\right)10\right] = 12{,}566 \text{ lb } (55.89 \text{ kN})$$

The shaft resistance due to skin friction from level (–10.00) [–3 m] to (–15.00)[–4.57 m] is

$$\left[\left(\frac{1100+1334}{2}\right)\tan 20° + 200\right]\left[\pi\left(\frac{12}{12}\right)5\right] = 10{,}095 \text{ lb } (44.9 \text{ kN})$$

The shaft resistance due to skin friction from level (–15.00) [–4.57 m] to (–50.00) [–15 m] is

$$[1334(\tan 20°) + 200]\left[\pi\left(\frac{12}{12}\right)35\right] = 75{,}340 \text{ lb } (335.1 \text{ kN})$$

The total shaft resistance due to skin friction is

$$Q_t = 12{,}566 + 10{,}095 + 75{,}340 = 98{,}001 \text{ lb } (435.9 \text{ kN})$$

The tip-bearing resistance is, using $N_q = 8$,

$$Q_t = \sigma_v' N_q A_t$$

$$= 1334 \times 8 \frac{\pi(1)^2}{4} = 8378 \text{ lb } (37.26 \text{ kN})$$

The allowable compression capacity is

$$Q_{\text{all}} = \frac{Q_{\text{ult}}}{FS} = \frac{98{,}001 + 8378}{3} = 35{,}460 \text{ lb} \approx 35 \text{ kips } (157.7 \text{ kN})$$

4C.3.3 Pile Group Capacity

In actual construction applications it is rare to encounter a foundation consisting of a single pile. Rather, a group of piles is used to transmit the load of the superstructure to the soil mass. Figure 4C.10 presents some typical pile groupings. When several piles are placed with a distance s from each other, it is expected that both the skin resistance stress and the tip-bearing stresses developed in the soil will overlap. Due to this overlap it is reasonable to expect that the pile group capacity could be less than the sum of the individual pile capacities. The pile group efficiency η is the ratio of the pile group capacity to the sum of the individual pile capacities. Several equations have been proposed to determine the numerical value of η. Of these equations, the Converse-Labarre equation seems to be one of the most accepted. According to this equation, the ratio η is equal to

$$\eta = 1 - \frac{\theta}{90°} \frac{(n-1)m + (m-1)n}{mn} \tag{4C.11}$$

where m = number of rows
n = number of piles in a row
θ = arctan (d/s)
d = pile diameter
s = center-to-center spacing of piles

The drawback of using the η equation is neglecting the beneficial effects of the pile-driving operations on increasing the relative density and the friction angle of the sand. Another approach to determine the pile group capacity was presented by Terzaghi and Peck (1967). They assumed the group of piles with the enclosing soil to form a rigid pier which behaves as a unit (see Fig. 4C.11). The bearing capacity of the pier is calculated as the sum of the tip-bearing resistance of the area $a \times b$ and the skin resistance on the perimeter of the pier. Thus,

$$Q_{\text{ult(group)}} = f_s[2(a \times b)L] + q(a \times b) \tag{4C.12}$$

where f_s and q are obtained as presented in Sec. 4C.3.2 for different soil types.

For design purposes the pile group capacity of driven piles in sand not underlain by a weak layer is to be taken as the sum of the single-pile capacities. For other conditions the pile group capacity is the least of either the sum of the single-pile capacities or the group capacity as determined from Eq. (4C.12). The pile spacing is generally taken not less than three times the pile diameter on centers for bearing piles, and a minimum of three to five times the pile diameter on centers for friction piles, depending on the characteristics of the soil and the piles.

4C.4 PILE DYNAMICS

4C.4.1 Dynamics Equations for Pile Capacity

It is well known that the pile load capacity is affected by the method of installation. There are several installation methods based on dynamic processes such as vibration and ramming. In order to estimate the capacity of the pile while it is being driven at the site, many driving formulas have been proposed. These formulas are all based on the following relation:

$$\text{Energy input} = \text{energy used} + \text{energy lost} \tag{4C.13}$$

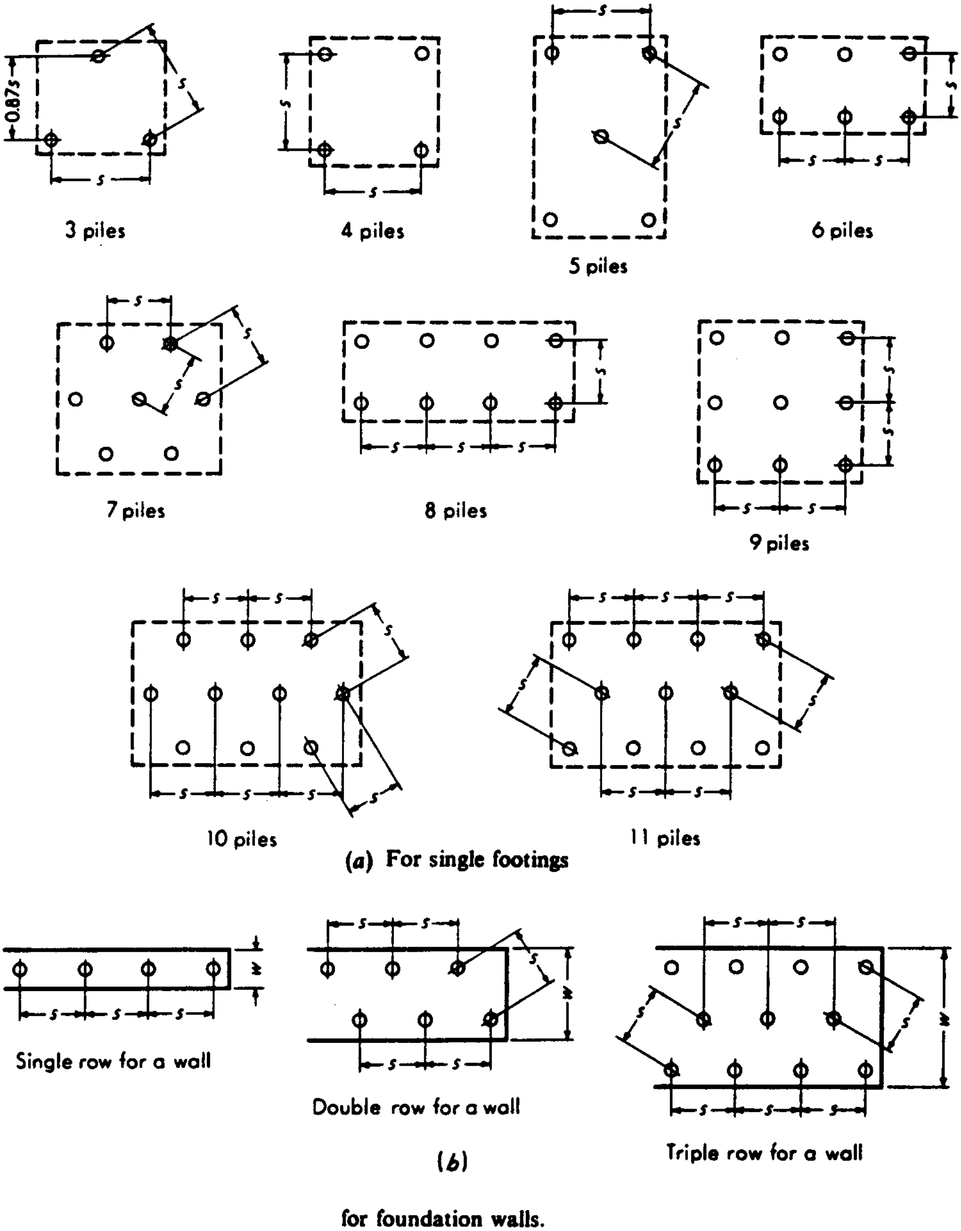


FIGURE 4C.10 Typical pile group patterns. (*a*) Single footings. (*b*) Foundation walls. (*From Bowles, 1982*.)

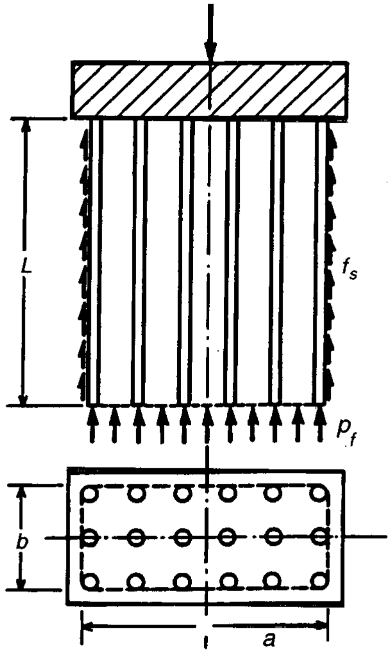


FIGURE 4C.11 Load capacity of a pile group. (*From Terzaghi and Peck, 1967.*)

Unfortunately these equations are not consistently reliable. The main reasons for this are:

1. The pile elastic compression is calculated using a static approach.
2. A portion of the input energy is used in displacing the soil laterally.
3. The total resistance is assumed acting on the pile tip.
4. The effects of driving velocity and duration of intermissions on the dynamic penetration are neglected.

In spite of these limitations, application of the driving formula could be beneficial to compare the dynamic resistance of piles driven in a site. This would give the engineer a way to judge the uniformity of the site subsoil. The most commonly used driving formulas are the *Engineering News* formula,

$$R = \frac{1.25e_hE_h}{S + 0.1}\,\frac{W_r + (n \cdot n)W_p}{W_r + W_p} \tag{4C.14}$$

and the Danish formula

$$R = \frac{e_hE_h}{S + C_1} \tag{4C.15}$$

where R = load capacity of pile (just after driving)
e_h = hammer efficiency
E_h = hammer energy rating
S = amount of point penetration per blow

W_r = weight of ram
W_p = weight of pile
n = coefficient of restitution
$C_1 = (e_h E_h / 2AE)^{1/2}$ with AE and L being the pile cross section, modulus of elasticity, and length.

Currently the best means for estimating the pile capacity dynamically consists of recording the pile-driving history and then load testing the pile. It would be a reasonable assumption to expect that piles with similar driving history will develop the same load capacity.

4C.4.2 Dynamics Considerations

The vibration attenuation and the liquefaction potential due to construction dynamic loading should be investigated. The U.S. Army Corps of Engineers requires the removal or densification of liquefiable soil (ASCE, 1993). Also the first few natural frequencies of the structure-foundation assemblage should be determined and compared to the construction operations frequencies in order to avoid resonance.

4C.5 PILE LOAD TEST

As mentioned previously, load testing a pile is considered the most dependable way of determining its carrying capacity. The pile load test consists of applying a series of increasing load values and measuring the corresponding settlements to obtain a pile load-settlement curve (Fig. 4C.12). Many empirical methods have been proposed to determine the pile capacity from the pile load-settlement data. Table 4C.7 includes a list of most of these methods. The methods used by the U.S. Army Corps of Engineers consists of taking the average of three load values obtained from the load test data as the pile load capacity. These three load values are

1. The load causing a movement of 0.25 in on the net settlement curve
2. The load corresponding to the point on the curve with a significant change in slope
3. The load matching the point on the curve that has a slope of 0.01 in per ton

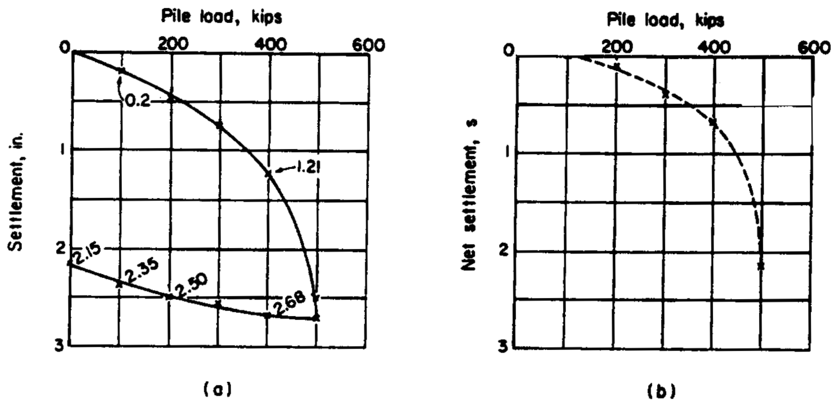


FIGURE 4C.12 Typical pile load test data. (a) Total settlement curve. (b) Net settlement curve.

TABLE 4C.7 Methods of Pile Load Test Interpretation

1. Limiting total butt settlement
   a. 1.0 in (Holland)
   b. 10% of tip diameter (United Kingdom)
   c. Elastic settlement+D/30 (Canada)
2. Limiting plastic settlement
   a. 0.25 in (AASHTO, N.Y. State, Louisiana)
   b. 0.5 in (Boston) [complete relaxation of pile assumed]
3. Limiting ratio: plastic/elastic settlement 1.5 (Christiani and Nielson of Denmark)
4. Limiting ratio: settlement/unit load
   a. Total 0.01 in/ton (California, Chicago)
   b. Incremental 0.03 in/ton (Ohio)
   0.05 in/ton (Raymond International)
5. Limiting ratio: plastic settlement/unit load
   a. Total 0.01 in/ton (N.Y. City)
   b. Incremental 0.003 in/ton (Raymond International)
6. Load-settlement curve interpretation
   a. Maximum curvature: Plot log total settlement versus log load; choose point of maximum curvature.
   b. Tangents: Plot tangents to general slopes of upper and lower portions of curves; observe point of intersection.
   c. Break point: Observe point at which plastic settlement curve breaks sharply; observe point at which gross settlement curve breaks sharply (Los Angeles).
7. Plunge Find loading at which pile "plunges" (i.e., the load increment could not be maintained after pile penetration was greater than 0.2B).
8. Texas quick load Construct tangent to initial slope of load versus gross settlement curve; construct tangent to lower portion of the load versus gross settlement curve at 0.05 in/ton slope. The intersection of the two tangent lines is the ultimate bearing capacity.

Source: ASCE, 1993.

4C.6 NEGATIVE SKIN FRICTION

In some cases, piles are driven into compressible soil before its consolidation is complete. If a fill is placed on this compressible soil, the soil will move downward against the pile. This relative movement creates a skin friction between the pile and the moving soil that increases the axial load acting on the pile. This mechanism is known as negative skin friction (Fig 4C.13). In excessive soil consolidation cases a gap may form between the bottom of the pile cap and the fill. This results in the full cap weight being transferred directly to the piles and could alternate the stresses in the pile cap. The value of the negative skin friction can be computed as follows:

$$Q_{\mathrm{NF}} = cLP' \tag{4C.16}$$

where c = shear strength of soil
L = length of pile in contact with compressible layers
P' = perimeter of pile

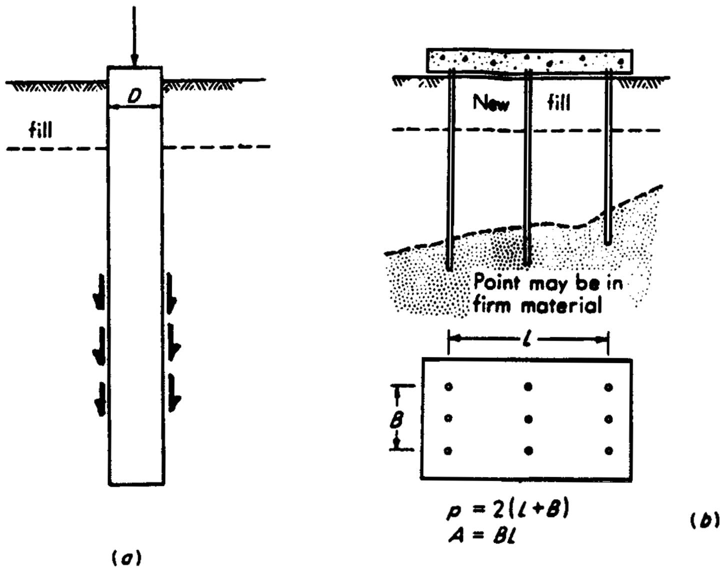


FIGURE 4C.13 Negative skin friction. (*a*) Single pile. (*b*) Pile-group effect.

Thus the total applied load on the pile becomes

$$Q_{TOTAL} = Q + Q_{NF} \tag{4C.17}$$

where Q is the load transmitted from the superstructure.

If piles are spaced a small distance apart, the developed negative skin friction may also be represented as the dragging force acting on the perimeter of the pier formed by the group of piles and the enclosed soil. In these situations, two modes of negative skin friction require investigation:

1. $$Q_{NF} = N(Q_{NF}/\text{pile}) \tag{4C.18a}$$

where Q_{NF}/pile is as given as by Eq. (4C.16) and N is the number of piles.

2. $$Q_{NF} = AL\gamma + cLP \tag{4C.18b}$$

where A = area bounded by pile group
L = length of pile in contact with compressible layers
γ = unit weight of compressible layer
c = shear strength of soil
P = perimeter of area A.

In the presence of expansive soils the negative skin friction phenomenon can generate upward tension stresses in the pile. These stresses could be larger if no or an insufficient gap is left between the expansive soil and the bottom of the pile cap.

4C.7 LATERALLY LOADED PILES

Piles are slender vertical members that have only limited capability to resist nonvertical loads. Therefore batter piles are used to resist large inclined or horizontal loads when acting on a structure. Beresantsev et al. (1961) suggested the following practice to transmit inclined loads in terms of the angle α, where α is the angle of the force to be transmitted with the vertical (Fig. 4C.14):

1. Vertical piles $\alpha < 5°$
2. Batter piles $5° \leq \alpha < 15°$
3. Deadman $\alpha < 15°$

Brooms (1965) presented charts that give the limit lateral load to act on a vertical pile versus the ratio of the pile embedment length to its diameter. These diagrams, applied to short piles, are presented in Fig. 4C.15 for cohesive and cohesionless soils. The term "short piles" refers to rigid piles where the lateral capacity is dependent mainly on the soil resistance. Long piles are those whose lateral capacity is primarily dependent on the yield moment of the pile itself. Figure 4C.16 shows the relationship between the limit lateral load and the yield bending moment of the pile for cohesive and cohesionless soils. In these figures the dashed lines represent the case of a fixed pile head, whereas the full lines indicate different e/l ratios, where e is the height of the line of action of the force P above ground surface and l is the length of pile in the ground.

The symbols used in these figures are

d = diameter of pile
γ = soil bulk unit weight
k_p = coefficient of passive earth pressure
H_u = limit value of horizontal load
c_u = undrained shear strength
L = pile embedment length

Very few test results are available for inclined forces acting on vertical piles or on a batter pile. Petrasovits and Awad (1968) conducted model tests on piles having a length of 50cm and a diameter of 1.3 to 3.5cm embedded in a soil with an angle of internal friction $\phi = 37.5°$. Figure 4C.17 gives

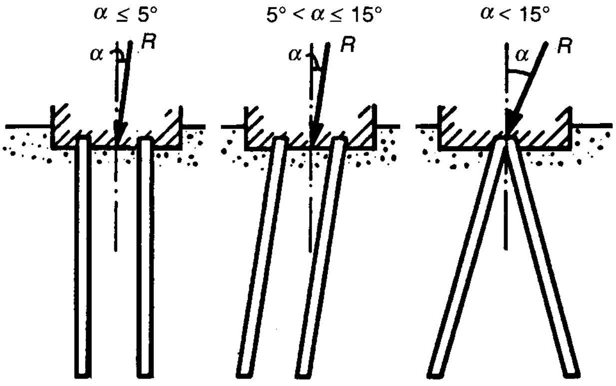


FIGURE 4C.14 Recommended practice to transmit inclined loads to soil mass. (*From Beresantsev et al.*, 1961.)

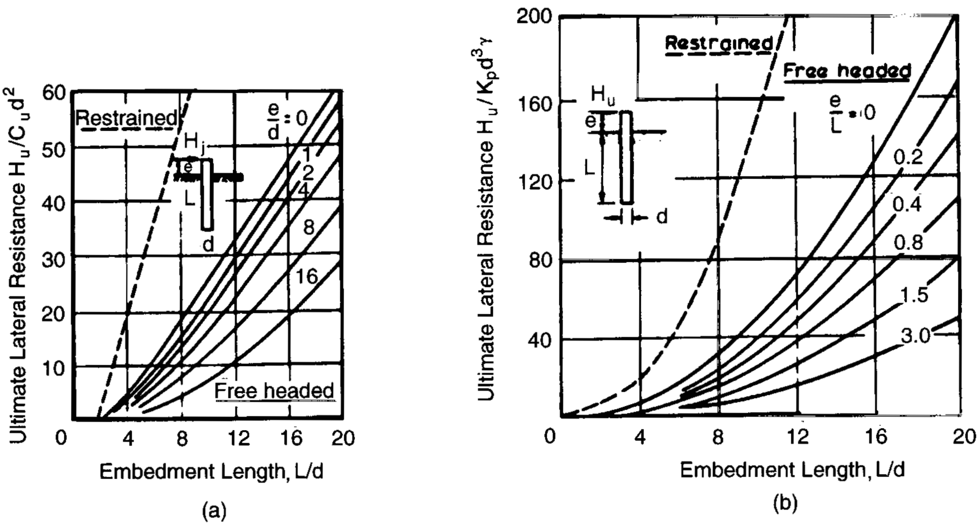


FIGURE 4C.15 Ultimate lateral resistance of short piles. (a) Cohesive soils. (b) Cohesionless soils. (*From Brooms, 1965.*)

the percentage of increase for the applied load for different cases when the inclination angle β of the pile is varied.

4C.8 PILE CAP DESIGN

Pile caps are used to distribute the loads and moments acting on the column to all of the piles in the group. The pile cap is usually made of reinforced concrete and rests directly on the ground, except when the soil underneath is expansive. The design considers the effects of a number of concentrated reactions due to the column load, surcharge load, fill weight, and pile cap weight. For the design of a rigid pile cap it is usual to assume that each pile carries an equal amount of concentric axial load and that a planar stress distribution is valid for nonconcentric loading. This assumption is justified when the pile cap is resting on the ground, the piles are vertical, the load is applied at the center of the pile group, and the pile group is symmetrical. The structural design of a reinforced concrete pile cap requires consideration of the following critical conditions:

1. Punching shear failure at sections located at a distance $d/2$ from the face of the column and around each pile
2. Beam shear failure at sections at a distance d from the face of the column
3. Bending failure at the sections located at the face of the column

where d is the effective depth at which the steel layer is placed.

The rules followed during the design are essentially the same as the ones used for spread footings. When deciding on the piles causing shear, attention is drawn to Chap. 15 of ACI 318 (1989), which states that

> Computation of shear on any section through a footing supported on piles shall be in accordance with the following: a) Entire reaction from any pile whose center is located $d_p/2$ (d_p is the pile diameter at the up-

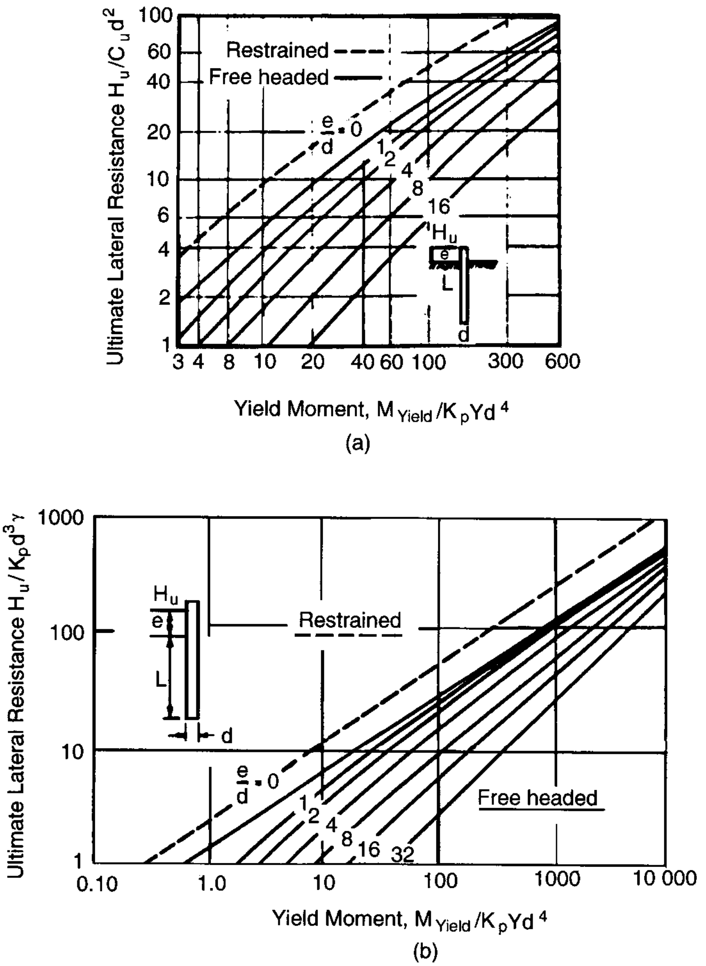


FIGURE 4C.16 Ultimate lateral resistance of long piles. (*a*) Cohesive soils. (*b*) Cohesionless soils. (*From Brooms, 1965.*)

per end) or more outside the section shall be assumed as producing shear on the section, b) reaction from any pile whose center is located $d_p/2$ or more inside the section shall be assumed as producing no shear on the section, c) for intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line interpolation between full value at $d_p/2$ outside the section and zero at $d_p/2$ inside the section.

The designer is urged to keep the pile cap design on the safe side because the individual actual pile reaction may differ from the value used in design due to group action and possible differences between layout on drawings and driven piles.

Example 4C.4 A 28-in-square (7 10-mm^2) column carries the following loads: P_D = 500 kips (2224 kN), P_L = 700 kips (3114 kN), M_D = 200 ft · kips (271 kN · m), and M_L = 300 ft · kips (406.8 kN · m). The column

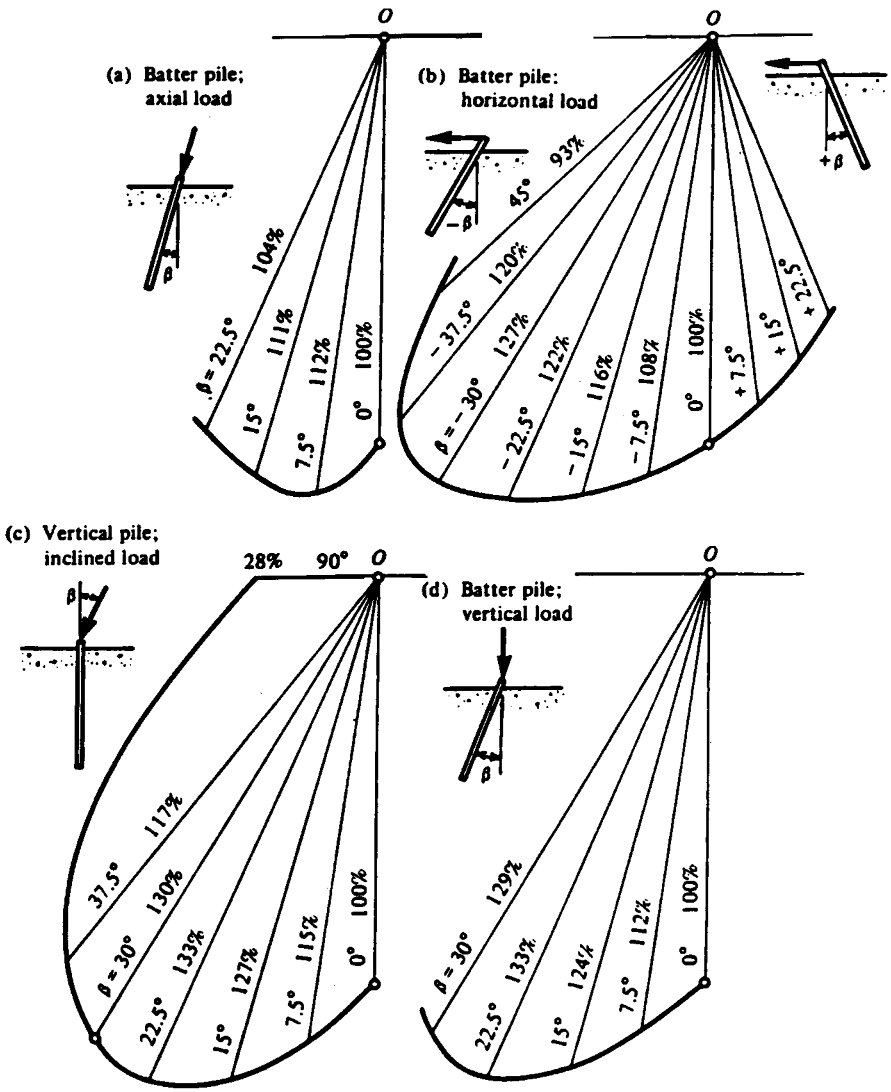


FIGURE 4C.17 Relative bearing capacity for batter piles or vertical piles subjected to inclined forces. (*From Petrasovits and Awad. 1968.*)

is to rest on a 4-ft-thick (1.2-m) cap supported by piles having an allowable load capacity of 100 kips (445 kN) and a diameter of 12 in (304 mm). The cap is topped with 12 in (304 mm) of fill having a unit weight of 120 lb/ft^3 (1922 kg/m^3) and 6-in concrete (152-mm) slab carrying a surcharge load of 100 psf (4788 Pa) (see Fig. 4C.X.4a). Design the pile cap using $f'_c = 4000$ psi (27.6 MPa) and $f_y = 60{,}000$ psi (414 MPa).

Solution The total vertical load is

$$P_{total} = 500 + 700 = 1200 \text{ kips } (5338 \text{ kN})$$

To account for the bending effects as well as the surcharge and cap weight, choose a total number of 15 piles spaced at 3 ft on centers (Fig. 4C.X.4*b*).

The surcharge load per pile is

$$P_s/\text{pile} = 3^2(0.5 \times 150 + 100 + 1 \times 120 + 4 \times 150) = 8055 \text{ lb} \approx 8.1 \text{ kips (36 kN)}$$

$$\Sigma x^2 = 3(3^2 + 6^2 + 3^2 + 6^2)144 = 38{,}880 \text{ in}^2 \; (250 \times 10^3 \text{ cm}^2)$$

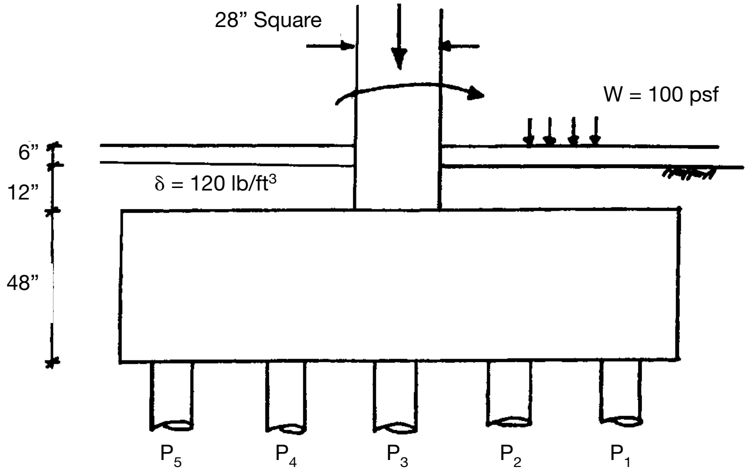


FIGURE 4C.X.4.*a*

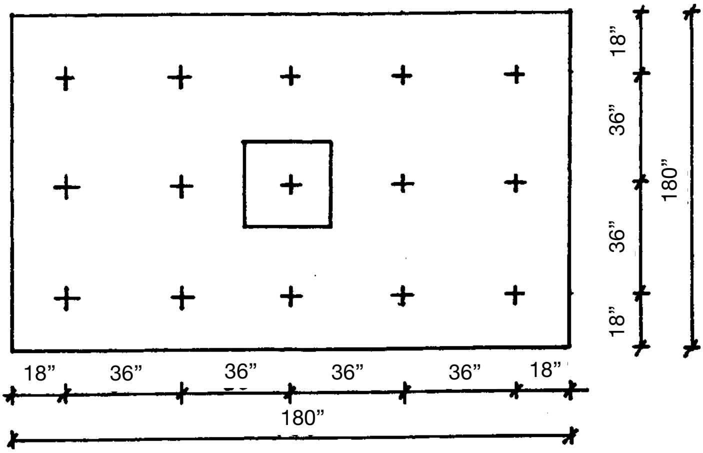


FIGURE 4C.X.4*b*

The net allowable capacity of a pile is

$$P_{\text{net}} = 100 - 8.1 = 91.9 \text{ kips (408.8 kN)}$$

The maximum axial force of a pile is

$$\frac{1200}{15} + \frac{500 \times 12 \times 72}{38{,}880} = 91.1 \text{ kips (405.2 kN)} < 91.9 \text{ kips (408.8 kN)} \qquad \text{O.K.}$$

The factored load on piles p_1 through p_5 is

$$P_1 = \frac{500 \times 1.4 + 700 \times 1.7}{1.5} + \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)72}{38{,}880} = 143.6 \text{ kips (638.7 kN)}$$

$$P_2 = \frac{500 \times 1.4 + 700 \times 1.7}{15} + \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)36}{38{,}880} = 134.8 \text{ kips (599.6 kN)}$$

$$P_3 = \frac{500 \times 1.4 + 700 \times 1.7}{15} = 126 \text{ kips (560.41 kN)}$$

$$P_4 = \frac{500 \times 1.4 + 700 \times 1.7}{15} - \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)36}{38{,}880} = 117.2 \text{ kips (521.3 kN)}$$

$$P_5 = \frac{500 \times 1.4 + 700 \times 1.7}{15} - \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)72}{38{,}880} = 108.4 \text{ kips (482.16 kN)}$$

Pile punching shear check:

$$V_u = 143.6 \text{ kips (639 kN)}$$

$$V_c = 4\sqrt{f_c'}b_0d = \frac{4\sqrt{4000} \times \pi \times 3\,6 \times 44)}{1000} = 1258 \text{ kips}$$

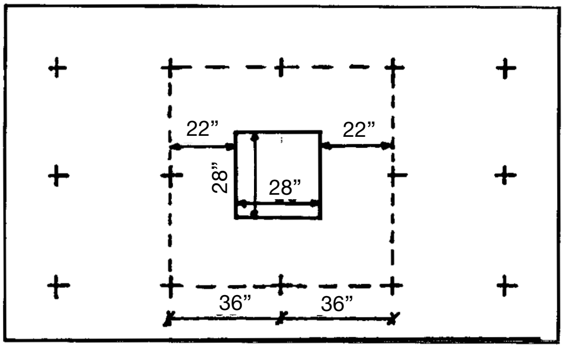
$$\frac{V_u}{\phi} = \frac{143.6}{0.85} = 169 < 1258 \text{ kips } (752 < 5596 \text{ kN}) \qquad \text{O.K.}$$

Two-way shear check (using a conservative approach for computation of applied shear):

$$V_u = 3 \times 143.6 + 3 \times 134.8 + 2 \times 126 + 3 \times 117.2 + 3 \times 108.4 = 1764 \text{ kips (7846 kN)}$$

$$V_c = 4\sqrt{f_c'}b_0d = \frac{4\sqrt{4000} \times 72 \times 4 \times 44}{1000} = 3206 \text{ kips (14,260 kN)}$$

$$\frac{V_u}{\phi} = \frac{1764}{0.85} = 2075 < 3206 \text{ kips } (9229 < 14{,}260 \text{ kN}) \qquad \text{O.K.}$$

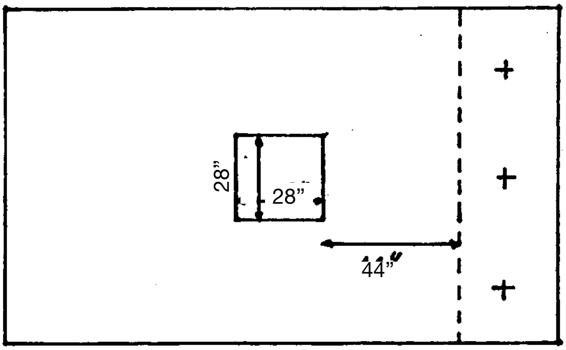


Beam shear check:

$$V_u = 3 \times 143.6 = 430.8 \text{ kips (1916 kN)}$$

$$V_c = 2\sqrt{f'_c}b_0d = \frac{2\sqrt{4000} \times 108 \times 44}{1000} = 601 \text{ kips (2673 kN)}$$

$$\frac{V_u}{\phi} = \frac{430.8}{0.85} = 507 < 601 \text{ kips} \qquad \text{O.K.}$$



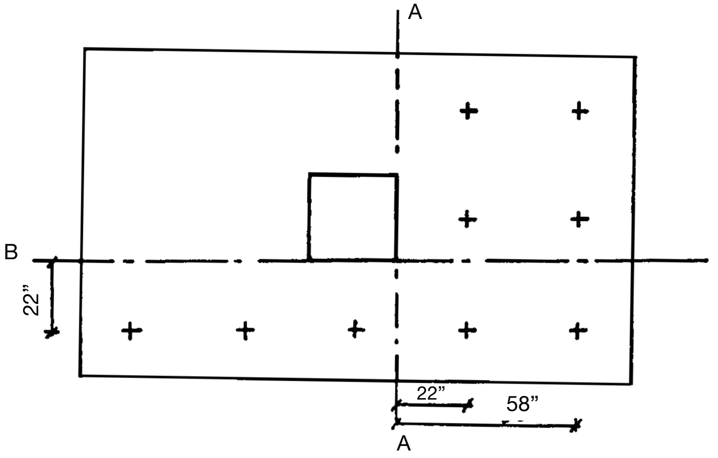
Flexure bending design: Check Sec. 3B.

1. Section *A-A:*

$$M_u = (3 \times 143.6 \times 1000 \times 58 + 3 \times 134.8 \times 1000 \times 22 = 33.9 \times 10^6 \text{ lb} \cdot \text{in (3831 kN} \cdot \text{m)}$$

$$A_s = \frac{M_u}{\phi f_y(0.9d)} = \frac{33.9}{0.9 \times 60{,}000} \times 0.9 \times 44 \times 10^6 = 15.85 \text{ in}^2 \text{ (102.2 cm}^2\text{)}$$

$$(A_s)_{\min} = \left(\frac{200}{f_y}\right)bd = 0.0033 \times 108 \times 44 = 15.68 \text{ in}^2 \text{ (101.2 cm}^2\text{)}$$



Choose A, = 15.85 in^2 (102.2 cm^2)

$$a = \frac{15.85 \times 60{,}000}{0.85 \times 4000 \times 108} = 2.59 \text{ in (65.8 mm)}$$

$$M_n = 15.85 \times 60{,}000\left(44 - \frac{2.59}{2}\right) = 40.6 \times 10^6 > \frac{33.9 \times 10^6}{0.9} = 37.7 \times 10^6 \text{ 1b} \cdot \text{in (4260 kN} \cdot \text{m)}$$

Choose A_s = 15.85 in^2 (101.2 cm^2) in the x direction.

2. Section *B-B:*

$$M_u = (143.6 + 134.8 + 126 + 117.2 + 108.4)1000 \times 22 = 11 \times 10^6 \text{ lb} \cdot \text{in (1243 kN} \cdot \text{m)}$$

$$A_s = \frac{M_u}{\phi f_y(0.9d)} = \frac{11}{0.9 \times 60{,}000 \times 0.9 \times 44} \times 10^6 = 5.18 \text{ in}^2 \text{ (33.42 cm}^2\text{)}$$

$$(A_s)_{\min} = \left(\frac{200}{f_y}\right)bd = 0.0033 \times 180 \times 44 = 26.1 \text{ in}^2 \text{ (168.39 cm}^2\text{)}$$

Choose A_s = 26.1 in^2 (168.39 cm^2) in the y direction.

The selection and distribution of the bars as well as the development length checks are performed as presented in Sec. 3B.

4C.9 REFERENCES

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P • A • R • T • 5

RESIDENTIAL AND LIGHTLY LOADED FOUNDATIONS: DESIGN PARAMETERS AND PROCEDURES

SECTION 5
RESIDENTIAL AND LIGHTLY LOADED FOUNDATIONS

ROBERT B. ANDERSON

5.1 SLAB-ON-GROUND CONSTRUCTION

One of the most common forms of residential foundations is the slab-on-ground. This encompasses slabs both at the surface, which are common in most warmer climates, and below-surface slabs, which form basements. The concept is essentially the same from a design standpoint. In referring to slab-on-ground construction, the name says it all. *The slab is supported on the ground.* This means that a major focus of the design should be on the ground conditions and proper preparation of the ground for receiving a slab.

In most instances, the reinforcing associated with slab-on-ground is fairly minimal as compared to elevated slabs. This should highlight the importance of the supporting soil conditions and their affect on any slab-on-ground design. Paramount importance should therefore be placed on the identification of soil conditions prior to the undertaking of any foundation design, regardless of the size of the project. As a design professional, it is necessary to have proper identification of soil conditions either from personal experience, local code requirements, county soil surveys, or preferably a site-specific soils investigation. Failure to secure such information and citing its source on the design drawings is essential.

The choice of slab-on-ground construction is a function of architectural requirements and soil conditions. If the structure is to be elevated with a crawl space, the foundation will more than likely be pier supported with either spread footings, continuous strip footings, or a deep foundation. If soil conditions are poor either due to compressible clays or other deleterious conditions, structurally supported slabs may be in order.

Other options, depending on the nature of problem soils, are soil removal, soil improvement, or a stiffer and stronger slab-on-ground.

Essentially, residential foundations can be categorized as:

1. Supported on stable granular soils
2. Supported on plastic or compressible clay
3. Structurally supported

Each of these will be described. A fourth category, pier supported residences, will also be treated.

5.1.1 Stable Granular Soil Conditions

5.1.1.1 Required Geotechnical Information

It is recommended that a soils investigation be conducted in order to determine the type of soil on the site, the thickness and distribution of each soil type, and the density of granular soils. Clay soils should also be described and their physical properties determined.

Soils investigation for residential design should be site-specific, with at least one test boring per slab site. In production housing sites, a test program may suffice with fewer borings; however, number and location should be established by the geotechnical engineer. In well-established neighborhoods with consistent soils, the presence of a soil test within 300 feet may be acceptable.

Soils should be classified under the Unified Soil Classification System, thereby delineating the acceptability of a lightly reinforced slab.

5.1.1.2 Minimum Reinforcement Requirements

The Building Research Advisory Board study for the Federal Housing Authority recommends that certain granular and dense materials may entertain an unreinforced slab. According to the BRAB Report, as a general guideline based on the Unified Soil Classification, GW and CP soil types as well as dense or medium dense GM, GC, SW, SP, SM, SC, ML, and MH may support an unreinforced slab-on-ground. Although allowable, such a design is not encouraged by the author. If such soils are encountered, it is recommended that either an appropriate dosage of synthetic fibers or a minimum of 6 × 6 W1.4/W1.4 welded wire reinforcement be used in a slab with a minimum thickness of 4 inches.

Table 5.1 below consists of a suggested modification of BRAB Report recommendations for the selection of slab type based on soil conditions.

TABLE 5.1 Slab-Type Recommendations Based on Soil Conditions[1]

Soil type[2]	Minimum density[3] or PI or q_u	Recommended slab type
GW, GP	All Densities	Minimum 4″ with synthetic fibers or 6 × 6 W1.4/W1.4 WWF
GM, GC, SW, SP, SM, SC, ML, MH	Dense or medium dense	Minimum 4″ with synthetic fibers or 6 × 6 W1.4/W1.4 WWF
GM, GC, SW, SP, SM, SC, ML, MH	Loose	Minimum 4″ with 0.1% steel
CL, OL, CH, OH	PI < 15 and $q_u \geq 2500$ psf	Minimum 4″ with 0.1% steel
	PI > 15 and $q_u < 2500$psf	Reinforced and stiffened
	$q_u < 1000$ psf	Structurally supported slab
Pt	All	Structurally supported slab

[1]Modified from *Criteria for Selection and Design of Residential Slab on Ground* (Table P11), Publication 1571, National Academy of Sciences, Washington, DC, 1968.
[2]As classified under the Unified Soil Classification System.
[3]Unconfined compression strength of undisturbed sample.

For minimum reinforcement of a conventional nature, ACI 360 at the time of this writing recommends a value of 0.1% of the cross-sectional area of concrete. For a 4″ thick slab, this results in 0.048 in^2 per foot or 6 × 6 W2.5/W2.5. A more common selection would be 6 × 6 W2.9/W2.9, which is more popularly designated as 6 × 6 6/6 WWF.

The welded wire reinforcement is preferred in sheets to rolls in order to insure its proper placement in the slab, which is to be no lower than mid-height, but preferably 1½″ from the top.

Loads in excess of 500 psf and point loads should be addressed with supplemental reinforcement or slab thickness to accommodate their function. Static analysis based on the bearing capacity of the soil may be utilized.

5.1.1.3 Sample Problem

A single story residence has the footprint shown in Fig. 5.1. The soil classification is SW Loose with a recommended bearing capacity of 2000 psf. Frost depth is 13″ and distance from ground to top of slab is 8″. No loads except the perimeter shall exceed 500 psf.

1) Select a 4″ thick slab:

$$\text{Area of steel } (A_s) = 4 \times 12 \times 0.001 = 0.048 \text{ in}^2$$

Use 6 × 6 W2.9/W2.9 WWF – A_s = 0.058 in^2/ft > 0.048

2) Select perimeter grade beam:

$$8'' \text{ freeboard} + 13'' = 21''$$

Use 24″ deep grade beam for perimeter

The author recommends a minimum reinforcing for the grade beam of 0.25% in order to comply with the intentions of ACI 318.

Minimum steel = 12 × 24 × 0.0025 = 0.72

Use 4 #4 bars (2 top and 2 bottom)

A_s = 0.80 in^2 > 0.72 in^2

5.1.1.4 Suggested Layout and Details (see Fig. 5.2):

Note: Alternate exterior grade beams for this design may consist of filled concrete masonry units for greater depth requirements as shown in Fig. 5.3.

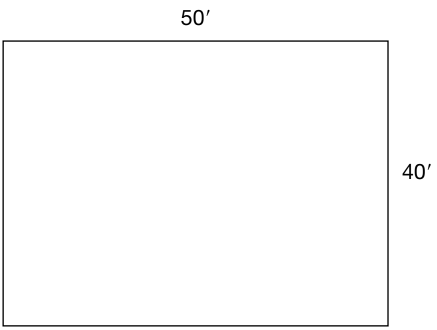
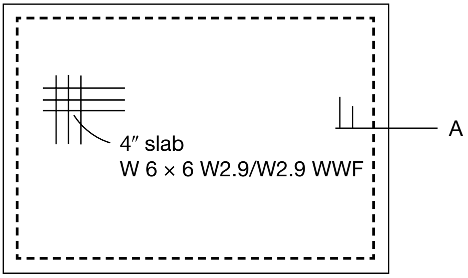


FIGURE 5.1.



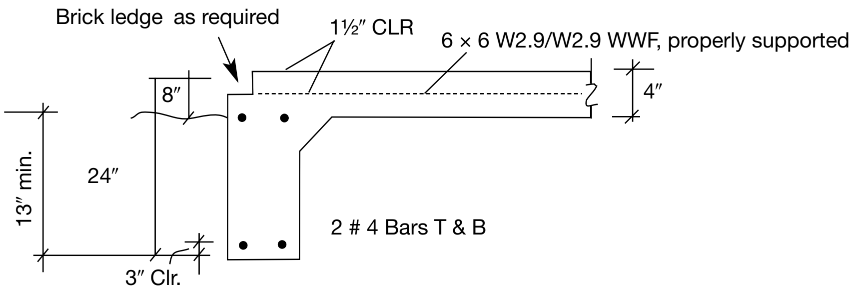


FIGURE 5.2.

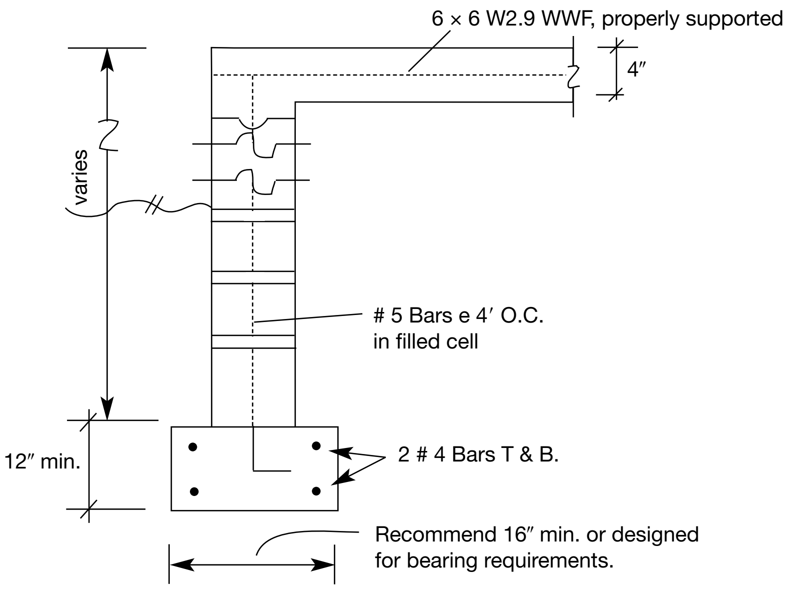


FIGURE 5.3.

5.2.1 Plastic or Compressible Clay Conditions

5.2.1.1 Required Geotechnical Information

As with any residential geotechnical report, the guidelines set forth in Section 5.1.1 should be followed. In plastic or compressible clay conditions, the soil is generally classified as fine-grained, and therefore falls into the category of CL, OL, CH, or OH. The geotechnical investigation should make recommendations relative to bearing capacity, and state if the soil is susceptible to volume change with moisture content or surcharge. Table 5.1 offers a guideline as to what types of information should be present.

In relatively stable high compressive strength clays with a low plasticity index (PI), a slab similar to the one shown in Fig. 5.2 is acceptable. Such recommendations should be made by the geotechnician.

With higher plasticity indices, a reinforced and stiffened slab may be required. It is desirable for the geotechnicial engineer in this instance to provide more detailed information relative to the potential change in volume of the soil. This information is critical in both dry and wet climates for the purpose of evaluating the stiffness necessary for a slab. Two of the more common delineations made in geotechnical investigations are to describe: 1) the potential vertical rise (PVR) of the soil, and 2) recommended values for e_m and y_m in plastic design procedure. A third property that may be reported in highly compressible clays is differential and total settlement.

When a slab is supported on a fine-grained soil (clay), or a collapsing soil (such as silt), the responsibilities of both the structural engineer and the geotechnical engineer become more acute. Care must be taken to anticipate any change in moisture content and drainage as well as irrigation that might affect the characteristics of the soil. The structural engineer may no longer be dealing with a static load being placed on an isotropic homogeneous material. The soil could very well be imposing an uneven load on the slab. This implies use of a reinforced and stiffened slab.

5.2.1.2 Consideration of Stiffened Slab Sections

The reinforced and stiffened slab tends to be a fairly complex design problem if done properly. This type of slab may be either conventionally reinforced or posttensioned. The preferable design procedure is that of the Post Tensioning Institute (PTI). More detailed design examples are found in *Designing Floor Slabs On Grade*, published by the Aberdeen Group, Addison, IL (1996).

It is suggested that the following two publications be referenced for a detailed design analysis for reinforced and stiffened slabs-on-ground:

1. *Design and Construction of Post-Tensioned Slabs-on-Ground*, 2nd ed., Post-Tensioning Institute, Phoenix, AZ, 1995.
2. *Designing Floor Slabs-on-Grade*, Boyd C. Ringo and Robert B. Anderson, 2nd ed., The Aberdeen Group, Addison, IL, 1996.

A simple to follow calculation procedure is considered too lengthy for this publication, and is better dealt with in the latter publication, which devotes 100 pages to two sample problems.

It is recommended that the designer at least become familiar with the Post Tensioning Institute design procedure prior to buying any software for problem solving. Software sources are provided through the Post Tensioning Institute.

5.2.1.3 Design Concept for Stiffened Slabs

The concept behind the PTI design procedure and least acceptable standard conventional designs is that the slab be capable of resisting both an edge lift condition and a center lift condition due to the normal seasonal volume change in the soil. This results in either a positive moment or a negative moment in the slab. This is schematically shown in Fig. 5.4.

Not only must the slab resist both positive and negative moments based on e_m and y_m design parameters, but it must also be stiff enough to function adequately by not inducing cracks in walls and ceilings due to this movement. Therefore, adequate stiffness is the governing concern of this design

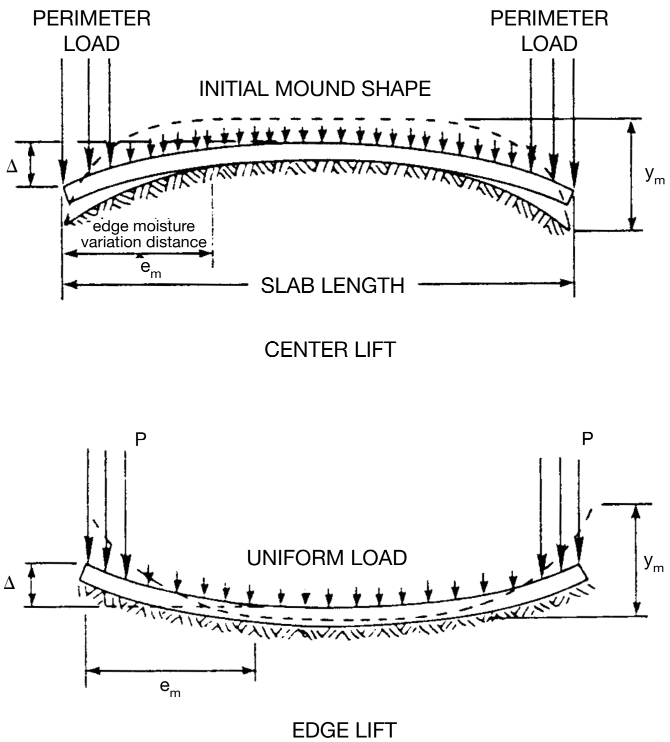


FIGURE 5.4.

procedure. Slabs must be either sufficiently thick or have adequate stiffening beams to control center lift and edge lift movement.

5.2.1.4 Compressible Clays

Slabs placed on compressible clays are designed very similar to those on expansive clays, provided the soil differential settlement is acceptable for ground support. The geotechnical report should predict both maximum settlement and maximum differential settlement. It has been the author's experience that if the predicted differential is less than 2″ or less than a deflection ratio of 1/300 measured from the center of the slab to the edge, with 2″ as the maximum limit, the slab can potentially be ground supported. The clay material must, however, be free of organics, and the procedure must be approved by the geotechnical engineer

Moments and deflections are primarily positive, due to the saucering of the slab associated with consolidation.

5.2 STRUCTURALLY SUPPORTED SLABS

When soil conditions are such that a ground-supported slab is not feasible, it may be necessary to structurally support the slab. This condition could arise if the soils is too weak to support the load.

Highly compressible clays would pose such a difficulty. Another reason to use structurally supported slabs may be extremely high volume change clays. In such instancess, slabs may have to be supported on drilled shafts with voids on the underside to accommodate volume change.

If a concrete slab is used under either of these circumstances, the slab must be designed in accordance with ACI 318. This document addresses elevated slabs. By being structurally supported on either piling or drilled shafts, such residential slabs fall under this code requirement. Failure to completely meet the steel requirements for slabs and beams in ACI 318 for structurally supported slabs is more than likely a concern with of most local code requirements within the United States. The designer should confirm that the minimum requirements of Chapter 7.12 of ACI 318 are met regardless of moment and shear requirements. There is a precedent for deleting minimum bonded steel if ultimate moment fails to produce stresses above $7.5\sqrt{f_c'}$.

5.3 PIER SUPPORTED RESIDENTIAL CONSTRUCTION (PIER-AND-BEAM FOUNDATIONS)

In many areas throughout the country, pier supported construction is both commonplace and desirable. It offers the homeowner a greater ease of access for plumbing and electrical modifications and repairs. The crawl space also maybe used as a heating and air conditioning plenum with some additional work.

Pier supported structures require essentially two design procedures. First, a pier design must be designed. A perimeter beam is then designed whose function is to transfer the perimeter loads to the pier system. This may be either on spread footings, a chain wall, or on a deep foundation. Subsequently a floor system and beams must be designed to transfer the interior load to the piers.

Floor systems may consist of wood floor joists, steel joists, or occasionally a concrete slab. Beams (or girders) transferring the load to piers may also be wood, steel, or concrete.

Care should be taken to ascertain that a pier supported floor meets governing code requirements for clearance from the ground. In many coastal areas, the minimum clearance is a function of flood water elevation, and refers to the lowest elevation of the lowest structural member of the floor.

There are several variations of pier-and-beam design other than those mentioned. One such design, popular in the southern United States is the “low profile” pier-and-beam foundation. Here the crawl space is excavated to permit a low silhouette comparable to the slab foundation.

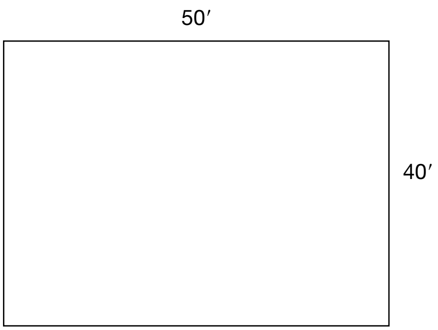


FIGURE 5.5.

Sample Problem

A single-story residence has the footprint shown in Fig. 5.5. The residence is wood frame. The soil classification is SW Loose with a recommended bearing capacity of 1500 psf for strip footings. There is no frost depth concern. Design a pier foundation with 24″ clear from ground level to underside of structural members. Wood construction is acceptable. The roof is wood truss design.

Step 1: Determine strip footing layout (see Fig. 5.6). Determine maximum load on piers that will be 6′3″ on center.

1. *Exterior loading.*

$$\text{Roof truss} = \frac{40}{2} \times (20 \text{ L.L.} + 15 \text{ D.L.})$$

$$= 20 \times 35 \qquad = 700 \text{ \#/l}'$$

$$\text{Floor load} = \frac{13.33}{2} (40 \text{ L.L.} + 20 \text{ D.L.}) \quad = 400 \text{ \#/l}'$$

$$\text{Allow exterior wall} = 100 \text{ \#/l}' \qquad = \underline{100 \text{ \#/l}'}$$

$$\text{S/T} \qquad = 1200 \text{ \#/l}'$$

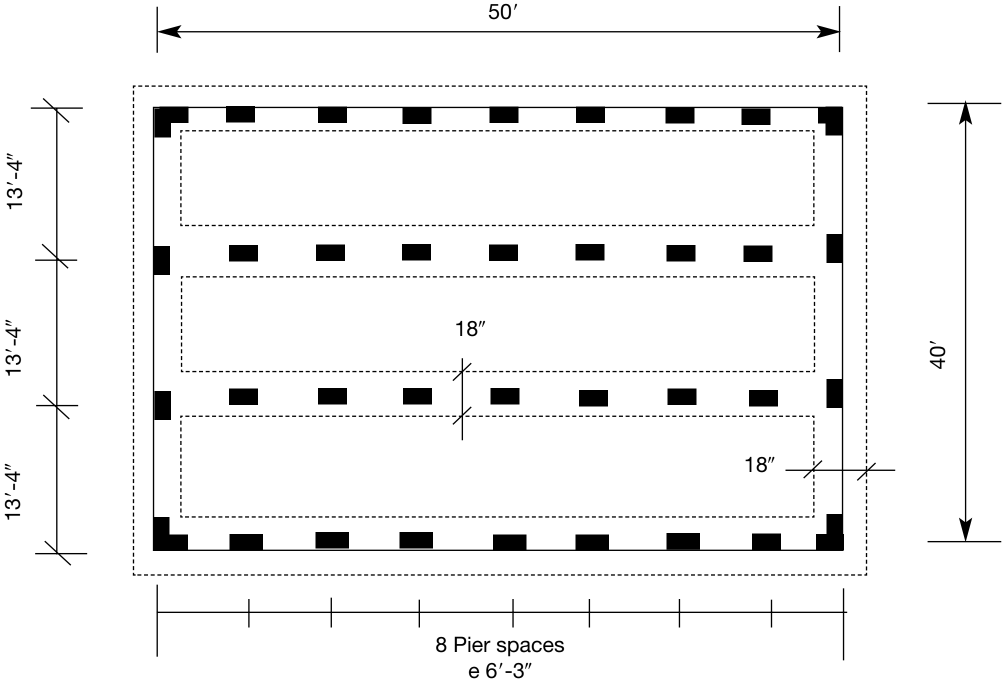


FIGURE 5.6.

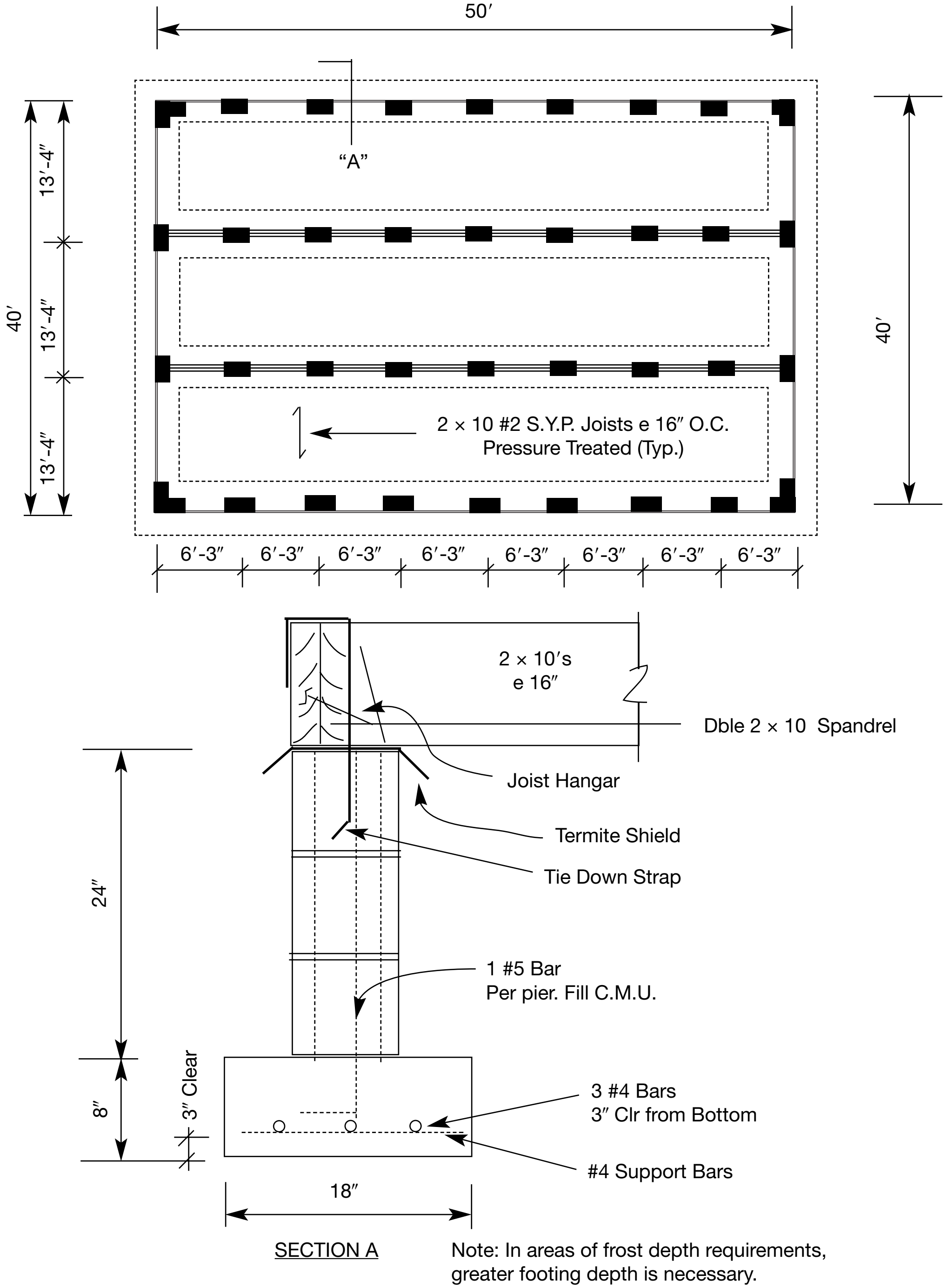

50′
40′
13′-4″
13′-4″
13′-4″
40′
"A"
2 × 10 #2 S.Y.P. Joists e 16″ O.C.
Pressure Treated (Typ.)
6′-3″
6′-3″
6′-3″
6′-3″
6′-3″
6′-3″
6′-3″
6′-3″
2 × 10′s
e 16″
Dble 2 × 10 Spandrel
Joist Hangar
Termite Shield
Tie Down Strap
24″
1 #5 Bar
Per pier. Fill C.M.U.
8″
3″ Clear
3 #4 Bars
3″ Clr from Bottom
#4 Support Bars
18″
SECTION A
Note: In areas of frost depth requirements,
greater footing depth is necessary.

FIGURE 5.7.

Assume 18″ wide × 8″ deep footing = 1.5 – 67 × 150 = 150 #/l′

Total load transferred to typical exterior pier T.L. = 1350 #/l′

1200 × 6.25 = 7500 #
Allow for Pier 150 #

Total load = 7650 #

2. *Check interior pier loading:*

Floor load = 13.33′ × (40 LL + 20 DL) = 800 #/l′
Allow for Interior Walls = 100 #/l′

Total Load = 900 #/l′

900 #/l′ < 1200 #/l′

∴ Use exterior strip footing design for all strip footings.

3. *Check steel requirements:*

For simplicity, assume a simple beam design to account for possible end rotation:

$M = \frac{1}{8} \times 1200 \times 6.25^2$
$M = 930'\#$
$M = 11.26''K$

Check minimum steel requirements:

$A_s = 0.002 \times 8 \times 18 = 0.288 \text{ in}^2$

For an 18″ wide footing we recommend a minimum of 3 #4 bars. $A_s = 0.20 \times 3 = 0.60$. Use grade 40 deformed bars.

Check moment capacity of 3 #4's 3″ clear from bottom. Use 3000 psi concrete.

0.60 × 40 = 24K
0.85 × 3000 = 2550
Ultimate compression block = 24 ÷ 2.55 = 9.41″
$a = 18 \div 9.41 = 1.91$
$a/2 = 0.95$; use $d = 5''$
$M_u = 24K(5 - a/2) = 97.2''K$
Assume ultimate moment = 1.8 × 11.26 = 20.27″K
97.2 > 20.27 ∴ OK—Steel is more than sufficient

4. *Floor Design-Beams*

Maximum span = (6.25 – 1.33 for C.M.U.) = 4.92′
Maximum load = 1200 #/l′
$M = \frac{1}{8} \times 4.92^2 \times 1200\ \#/l' = 3630'\#$
Use two 2 × 10's with a joist hanger
Maximum capacity = 4312′# > 3630′# ∴ OK

The final detail is shown in Figure 5.7.

P • A • R • T • 6

SOIL IMPROVEMENT AND STABILIZATION

SECTION 6A

NONGROUTING TECHNIQUES

EVERT C. LAWTON

6A.1 INTRODUCTION

Increasing urbanization continues to occur throughout much of the world, and at this time this trend appears likely to continue for at least the near future. As the population grows in any metropolitan area, a variety of additional facilities are needed to serve these people. Many of these facilities are in the form of structures—such as houses, apartment buildings, restaurants, and office buildings—that occupy areas within the metropolis. The additional space required for these structures is generally obtained in three ways: (*a*) Existing structures are torn down to make room for new structures; (*b*) new structures are built on land within the existing metropolitan boundaries that were previously "unimproved" (relative term); and (*c*) the boundaries of the metropolis are expanded to provide additional land for development.

One of the primary criteria used to select a site for development is the suitability of the ground for supporting the structure to be built. In most urban areas, the best sites were developed first, and, as urbanization continues, when a previously undeveloped site is purchased, the engineering properties of the existing near-surface materials are often such that the structure cannot be supported by shallow foundations. The traditional solution for these situations is to support the structures on deep foundations—typically piles or drilled piers—where a small portion of the load is transmitted to the poorer near-surface materials and a large portion of the load is transmitted to better bearing materials deeper within the ground. However, even with the development and use of more economical types of deep foundations such as auger-cast piles, the demand has increased for more economical solutions to the unsuitability of near-surface soils for shallow foundations.

To meet this demand, numerous soil stabilization and improvement techniques (also commonly called *ground modification techniques*) have been developed within the past 25 years or so. These techniques involve modifying the engineering properties and behavior of the near-surface soils at a site so that shallow foundations can be used where they previously could not or in some instances so that more economical shallow foundations can be used. The state of the art in this field is currently changing so fast that it is difficult, if not impossible, for any one person to keep up with it. It is likely that significant new technologies or modifications to existing technologies will have occurred between the time of writing and the time of publication of this handbook. Therefore, it is incumbent upon the reader to review the literature frequently to keep up with developments and changes in ground modification techniques.

Many of the techniques described in this section have proprietary restrictions to their use. In many instances, the restrictions are not in the use of the method itself but in other areas such as in the manufacture of a material or product used in the technique or in equipment used to perform the modifications to the soil or to install a particular product. Because patents associated with ground improvement techniques are continuously expiring and new patents are being obtained, it would be onerous and unfruitful to attempt to describe all these restrictions. Few patents are worldwide, and certain restrictions that apply in one country may not apply in the country where the project is being undertaken. Therefore, anyone who wishes to use or recommend the use of a particular soil improvement technique for a project should first perform a thorough examination of all potential proprietary restrictions associated with that technique.

Although many techniques are currently available, and more are likely to become available, not all techniques are appropriate for every project. The primary responsibilities of the foundation engineer, therefore, are to (*a*) determine which techniques can be used to safely support the structure; (*b*) determine which of the suitable methods is most economical for the project; (*c*) design or supervise the design of the details for the technique that best meets criteria (*a*) and (*b*); and (*d*) ensure that the actual modifications produce the desired result. In some instances, especially where proprietary restrictions are involved, the company or individuals who developed a particular method may require that they perform the design themselves, so that the project foundation engineer' s responsibility is to ensure that the design prepared by the company meets criteria (*a*) and (*b*) and that the final product achieves the desired objectives stated in the specifications.

In keeping with the goals of this handbook, soil improvement and stabilization techniques that are applicable to buildings are discussed in this chapter. The chapter is divided into two main sections—nongrouting and grouting techniques. These two sections are organized according to type of technique. In building applications, the most important static properties for bearing soils are strength, compressibility, drainage characteristics (permeability), and potential for wetting- and drying-induced volume changes. In seismically active regions, liquefaction potential of saturated silty sands and sands is extremely important. The discussions in this chapter are primarily aimed at showing how each technique can be used to improve one or more of these properties.

6A.2 OVEREXCAVATION/REPLACEMENT

The technique of overexcavation/replacement is one of the oldest, most intuitive, and simplest methods for modifying bearing materials to increase support for shallow foundations. The method consists of excavating poor or inadequate bearing material and replacing it with a stiffer and stronger material (see Fig. 6A.1). As long as the in-place replacement material is stiffer and stronger than the

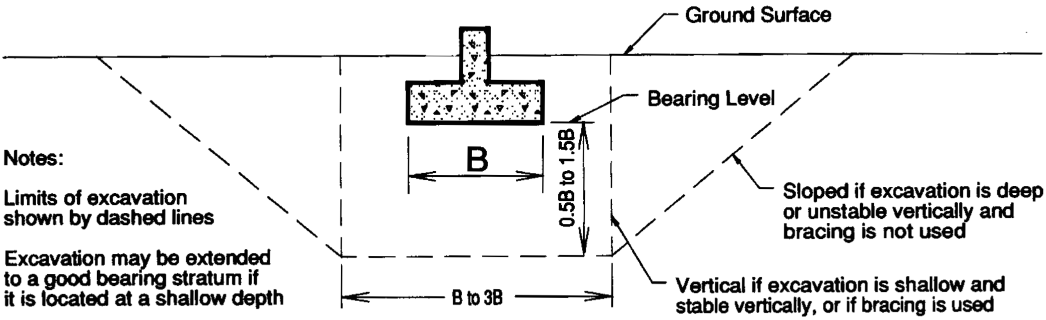


FIGURE 6A.1 Typical dimensions of excavation for overexcavation/replacement.

excavated bearing soil, the settlement that the foundation undergoes when loaded is reduced and the factor of safety against ultimate bearing capacity failure is increased. The greater the stiffness and strength of the replacement material, the greater the reduction in settlement and increase in ultimate bearing capacity.

Overexcavation/replacement is most commonly used when the bearing soils are very weak and highly compressible. The replacement material can be the excavated material that has been modified in some way, or it can be borrow material (obtained from another location on or off the site). The replacement material is usually sand, gravel, or a sand-gravel mixture, especially in situations where the ground-water table is high or when it is desirable to have a free-draining bearing material. To obtain the stiffest and strongest material possible, the replaced sand or gravel is usually compacted in lifts during the replacement process. If good-quality sands and gravels are not readily or economically available, the excavated soils can be chemically stabilized and used as the replacement material. However, chemically stabilized soils are generally not free-draining (they have relatively low permeabilities), so their use as a replacement material may change the local or regional ground-water seepage and precipitation infiltration patterns, which may be an environmental consideration in some instances. It is also possible to use the excavated soil as the replacement material without chemically modifying it, although this is seldom done. If the inadequate bearing soil is cohesionless, several techniques are available for densifying in situ soil that are more economical than removing, drying or wetting, replacing, and compacting it. Cohesive soils can be excavated, dried, and compacted at an appropriate water content to produce a material that is initially stiff and strong, but these soils may be susceptible to wetting-induced volume changes and reductions in stiffness and strength from wetting (see Sec. 6A.3.4).

The excavation is deeper and often wider than is needed to place the foundation, hence the term overexcavation. Typical dimensions for an overexcavation are shown in Fig. 6A.1. The width of the bottom of the excavation typically varies from one to three times the width of the foundation (B to $3B$ in Fig. 6A.1), and the depth below the bearing level is generally about ½ to 1½ times the foundation width ($0.5B$ to $1.5B$). If a good bearing stratum (medium dense sand, dense sand, gravel, or bedrock) exists close to the bearing level, the excavation is usually taken to the top of the bearing stratum or a shallow depth into it.

The replacement material is usually compacted so that the replaced zone is as stiff and strong as possible. A variety of compaction procedures can be used. If the excavation is narrow and shallow, hand-operated compaction equipment appropriate for confined areas is used, including rammers, tampers, vibrating plates, and small rollers (see Sec. 6A.3.1). For deep, narrow excavations, backhoes or hydraulic excavators with special compaction attachments can be used. Other techniques include pounding the material with the bucket of a backhoe and dropping a weight from a crane. For wide excavations, full-size compaction rollers are normally used.

6A.2.1 Ultimate Bearing Capacity

When the overexcavation/replacement process is properly designed and implemented, it results in a composite bearing zone that is stronger and stiffer than the unreinforced material. An increase in ultimate bearing capacity occurs because the potential failure surface must pass either through or around the stronger replaced zone. Three possible failure methods are shown in Fig. 6A.2 for a foundation bearing on a strong replaced zone of finite width and depth. It is possible (but unlikely) that the most critical failure surface is a general bearing failure surface that develops through the replaced zone and into the in situ soil [Fig. 6A.2(*a*)]; a theoretical solution for this case where the foundation bears on a long (continuous) granular trench within a soft saturated clay matrix immediately after loading (the clay is undrained) has been given by Madhav and Vitkar (1978). Their solution is presented in the form of charts from which modified bearing capacity factors (N_{ct}, N_{qt}, and $N_{\gamma t}$) can be obtained (Fig. 6A.3). These factors are valid for the following conditions:

1. The width of the granular trench (W) varies from zero (no trench) to twice the foundation width ($2B$).
2. The friction angle for the granular trench material (ϕ_1) varies from 20° to 50°, and the cohesion intercept (c_1) varies from zero to the same value as for the clay matrix (c_2).
3. The clay matrix is undrained ($\phi_2 = 0$, c_2 = undrained shear strength = s_{u2}).
4. The unit weights of the granular trench material and the clay matrix are the same ($\gamma_1 = \gamma_2$).

With these factors, the ultimate bearing capacity (q_{ult}) can be predicted from the following equation, which has the same form as the general bearing capacity equations for homogeneous bearing soil:

$$q_{ult} = c_2 N_{ct} + qN_{qt} + 0.5\gamma_2 BN_{\gamma t} \tag{6A.1}$$

Hamed and coworkers (1986) conducted laboratory model tests to determine the variation in q_{ult} for a continuous foundation bearing on a granular trench ($W = B$) constructed within a soft clay matrix. By comparing their experimental values of q_{ult} with values predicted from Eq. (6A.1) (see Fig. 6A.4), Hamed and colleagues concluded that Madhav and Vitkar's theory overpredicts q_{ult}. In addition, they found that the minimum height of the granular trench necessary to obtain the maximum value of q_{ult} is about $3B$. Hamed and colleagues also developed their own theory for q_{ult} based on the following assumptions:

1. Failure occurs by bulging at the bearing level along the interface between the trench and matrix materials.
2. Undrained conditions exist at failure in the saturated clay matrix.
3. The principal planes at the failure point are horizontal and vertical in both the trench and matrix materials.
4. The horizontal stress (σ_h) in the trench material is the minor principal stress (σ_3); σ_h in the matrix material is the major principal stress (σ_1). These two values of σ_h are equal.

From these assumptions, the following equation for q_{ult} was derived:

$$q_{ult} = (\gamma_2 D + 2s_{u2}) \tan^2\left(45° + \frac{\phi_1}{2}\right) \tag{6A.2}$$

A comparison of the maximum experimental values of q_{ult} with values calculated from Eq. (6A.2) (Fig. 6A.4) suggests that this theory provides a reasonable (but probably somewhat conservative) estimate of q_{ult} for $W = B$ and $H/B \geq 3$.

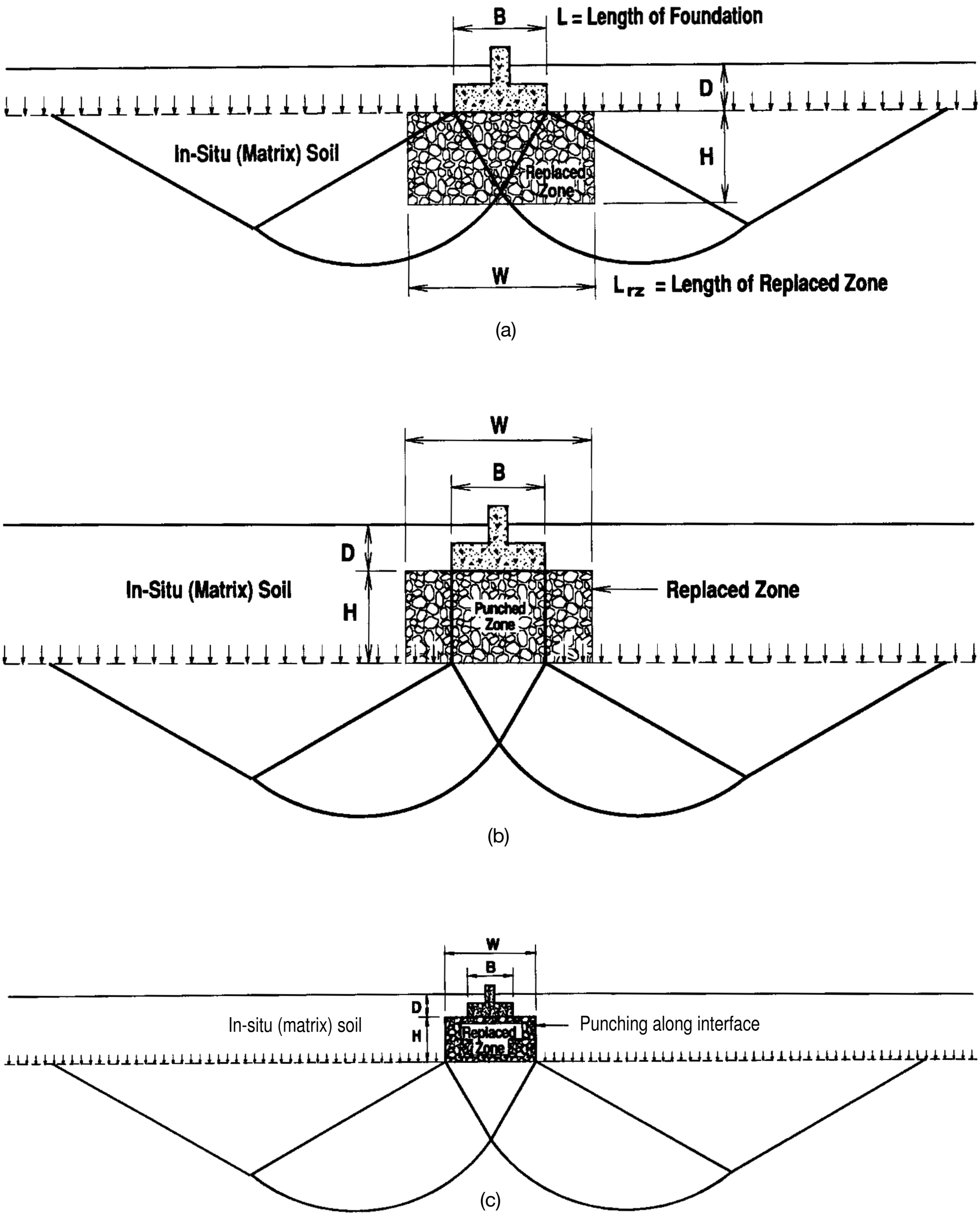
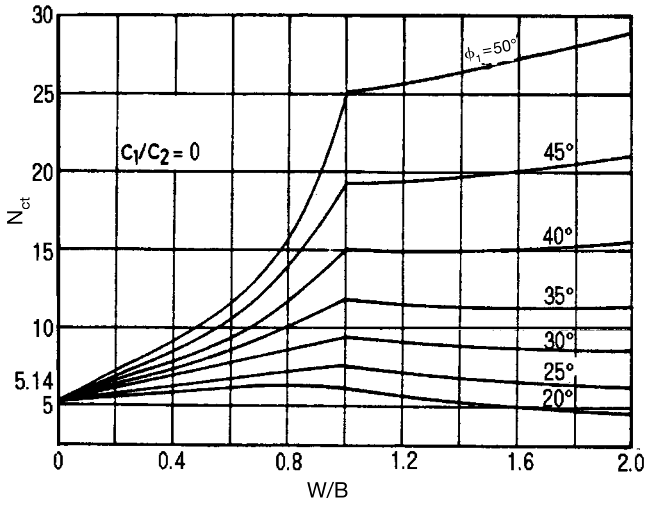
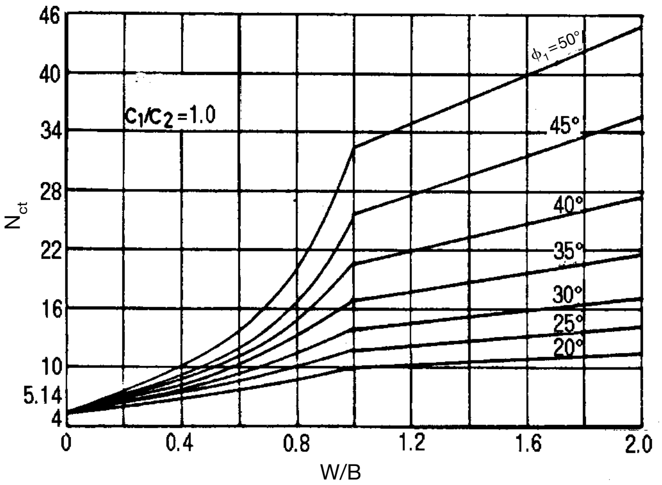


FIGURE 6A.2 Potential bearing shear failure mechanisms for foundation supported by overexcavated and replaced zone: (*a*) general shear failure through replaced zone and in situ soil; (*b*) punching failure through replaced zone; (*c*) punching failure of replaced zone through in situ soil.

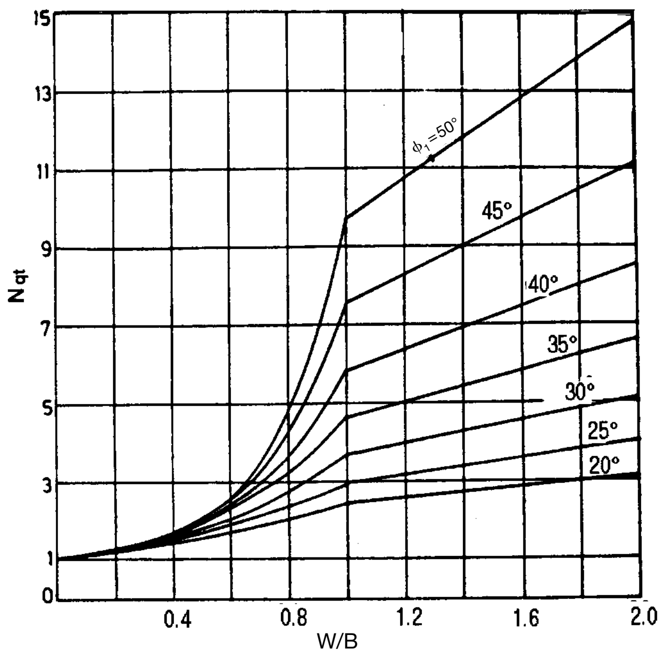




(a)

FIGURE 6A.3 Bearing capacity factors for general shear failure in a weak clay stabilized with a granular trench (from Madhav and Vitkar, 1978): (*a*) N_{ct}, (*b*) N_{qt}, (*c*) $N_{\gamma t}$.

If the in situ soil is a moderate or stiff clay or a granular soil, it is likely that failure would occur by punching through or around the perimeter of the replaced zone combined with general shear failure in the underlying in situ soil [Fig. 6A.2(*b*) and (*c*)]. The wider the replaced zone is, the more likely that punching failure would occur through it rather than around it. If it is not apparent which punching mechanism is more likely to occur, estimates of q_{ult} for both types can be determined and the lesser value used for design. It is also possible that general shear failure would occur completely within the replaced zone (q_{rz}), but this is not likely unless the replaced zone is deep and wide. It is



(a)

FIGURE 6A.3 (*Continued*)

prudent, however, to calculate q_{rz} in all cases to ensure that the computed values of q_{ult} for the two punching cases are less than q_{rz}.

For punching through the replaced zone [Fig. 6A.2(*b*)], the following equation based on Meyerhof and coworkers (Meyerhof and Hanna, 1978; Valsangkar and Meyerhof 1979) and Bowles (1988) can be used to estimate q_{ult}:

$$q_{\text{ult}} = q_b + \frac{pP_h \tan \phi_1 + pHc_1 - W_{pz}}{A_f} \leq q_{rz} \tag{6A.3a}$$

where q_b = ultimate bearing capacity of the in situ soil beneath the replaced zone based on the dimensions of the foundation

p = perimeter length of the punched zone = $2(B + L)$ for a rectangular foundation or πd_0 for a circular foundation (where d_0 diameter)

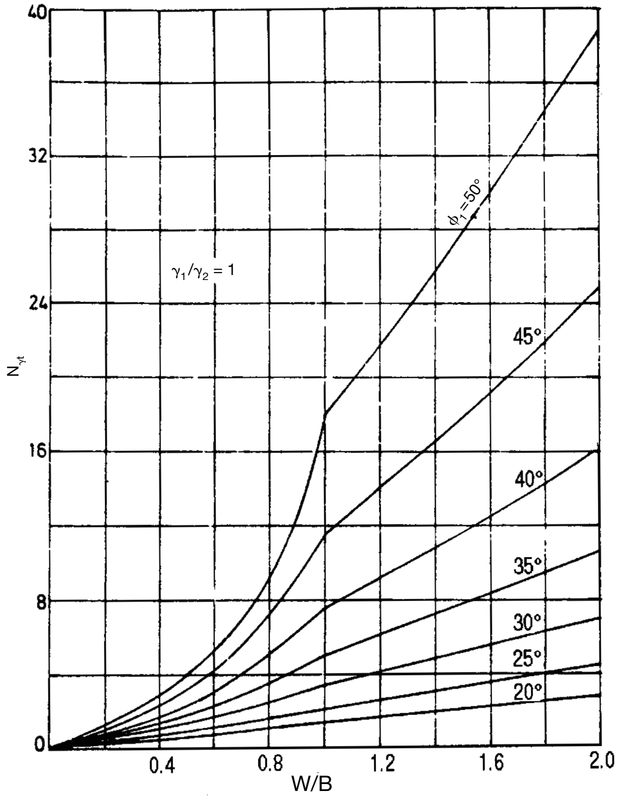
P_h = lateral earth pressure thrust (force per unit horizontal length) acting along the perimeter surface of the punched zone at failure = $\int_0^H K_s \sigma_v dH$

K_s = lateral earth pressure coefficient along the perimeter surface of the punched zone at failure

W_{pz} = weight of the material in the punched zone

A_f = area of the foundation = BL for rectangular and $0.25\pi d_0^2$ for circular

q_{rz} = ultimate bearing capacity for general shear failure within the replaced zone



(c)

FIGURE 6A.3 (*Continued*)

If the ground-water table is below the lowest point on the potential failure surface, the following equations apply:

$$q_b = c_2 N_{c2} S_{c2} D_{c2} + \gamma_2 (D + H) N_{q2} S_{q2} D_{q2} + 0.5 \gamma_2 B N_{\gamma 2} S_{\gamma 2} D_{\gamma 2} \qquad (6A.3b)$$

where N_{c2}, N_{q2}, $N_{\gamma 2}$ are bearing capacity factors (see Sec. 2B) for general shear failure in the in situ soil beneath the punched zone based on ϕ_2 and S_{c2}, S_{q2}, $S_{\gamma 2}$ are shape factors and D_{c2}, D_{q2}, $D_{\gamma 2}$ are depth factors (see Sec. 2B) based on the dimensions of the foundation and an embedded depth of ($D + H$).

The use of γ_2 for the N_{q2} term in Eq. (6A.3*b*) is conservative if W is greater than B, because a portion of the surcharge soil for the general failure surface beneath the punched zone consists of the replacement material (γ_1), which in most cases is denser than the in situ soil ($\gamma_1 > \gamma_2$). The wider the replaced zone, the more conservative is the use of γ_2 in the N_{q2} term. If desired, an equivalent unit weight can be calculated that accounts for the relative portions of the replacement material and in situ soil that are in the surcharge zone, but this refinement is probably not necessary in most instances.

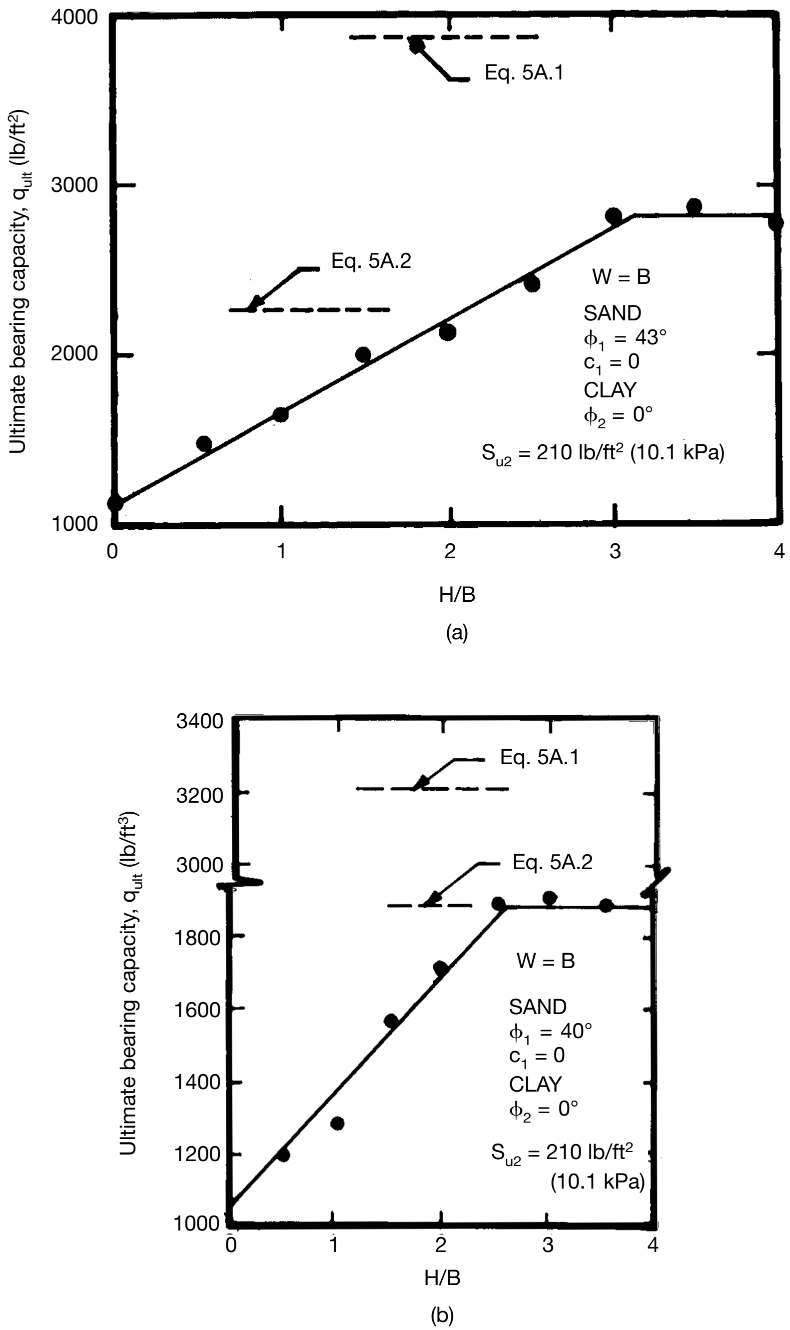


FIGURE 6A.4 Ultimate bearing capacity of a model continuous foundation on a granular trench within a soft clay matrix (from Hamed and coworkers, 1986): (*a*) trench material is dense sand. (*b*) Trench material is medium dense sand.

$$q_{rz} = c_1 N_{c1} S_{c1} D_{c1} + \gamma_2 D N_{q1} S_{q1} D_{q1} + 0.5\gamma_1 B N_{\gamma 1} S_{\gamma 1} D_{\gamma 1} \tag{6A.3c}$$

where N_{c1}, N_{q1}, $N_{\gamma 1}$ are bearing capacity factors for general shear failure within the replaced zone based on ϕ_1 and S_{c1}, S_{q1}, $S_{\gamma 1}$ are shape factors and D_{c1}, D_{q1}, $D_{\gamma 1}$ are depth factors based on the dimensions of the foundation and an embedded depth of D. In addition,

$$W_{pz} = BLH\gamma_1 \qquad \text{for a rectangular foundation} \tag{6A.3d.1}$$

$$W_{pz} = 0.25\pi d_0^2 H\gamma_1 \qquad \text{for a circular foundation} \tag{6A.3d.2}$$

If K_s is assumed to be constant at all depths along the perimeter surface of the punched zone, the following equation applies for P_h:

$$P_h = K_s(\gamma_2 DH + 0.5\gamma_1 H^2) \tag{6A.3e}$$

If the ground-water table is within the potential failure zone, its influence on the effective stresses and unit weights (and hence q_b, q_{rz}, W_{pz}, and P_h) must be considered, which may alter Eqs. (6A.3*b*), (6A.3*c*), (6A.3*d*), and (6A.3*e*).

Selection of a value for K_s is not necessarily simple. K_s probably varies along the depth of the punched zone, so if one value is selected, it constitutes an average or equivalent value. Meyerhof and Hanna (1978) provided values of K_s as a function of ϕ_1 and q_2/q_1 for the case of a continuous foundation bearing on a strong layer of infinite horizontal extent overlying a weak layer (Fig. 6A.5); q_1 and q_2 are the ultimate bearing capacities for a continuous foundation of width B under a vertical centric load on the surfaces of homogeneous thick deposits of the strong and weak soils and can be calculated from the following equations:

$$q_1 = c_1 N_{c1} + 0.5\gamma_1 B N_{\gamma 1} \tag{6A.4a}$$

$$q_2 = c_2 N_{c2} + 0.5\gamma_2 B N_{\gamma 2} \tag{6A.4b}$$

Thus, the term q_2/q_1 represents the relative strengths of the weaker and stronger materials, with $q_2/q_1 = 1$ representing soils of the same strength and $q_2/q_1 = 0$ corresponding to a strong soil that is infinitely stronger than the weak soil.

Values for K_s can be selected from Fig. 6A.5 but should be used with caution, since these values do not strictly apply to either finite length foundations or a replaced zone of finite width. The reader should also note that K_s does not equal Rankine's $K_p = \tan^2(45° + \phi_1/2)$ for two reasons: (*a*) Shear stresses develop along the vertical punched perimeter, and therefore σ_h and σ_v cannot be principal stresses as was assumed by Rankine; and (*b*) values of σ_v along the punched perimeter are likely to be higher than calculated from the weights of the overburden material owing to the application of the foundation load and the shearing action along the punched perimeter which therefore increases σ_h. Item (*a*) tends to reduce the value of K_s, whereas item (*b*) tends to raise it. Hence, K_s may be either greater or lesser than K_p when K_p is based on σ_v calculated from the weights of the overburden materials [as in Eq. (6A.3e)]; for example, the values of K_s for $\phi_1 = 30°$ in Fig. 6A.5 range from about 1.0 for $q_2/q_1 = 0$ to about 5.6 for $q_2/q_1 = 1$, while $K_p = 3.0$. If a conservative value is desired, K_s can be assumed equal to the at-rest coefficient for normally consolidated soils (K_{0-nc}):

$$K_s = K_{0-nc} = 1 - \sin\phi_1 \tag{6A.5}$$

It is unlikely that K_s would ever be less than K_{0-nc}. For a design situation, a better estimate of K_s can be obtained from Fig. 6A.5 than from Eq. (6A.5) so long as a reasonable factor of safety is applied to q_{ult}.

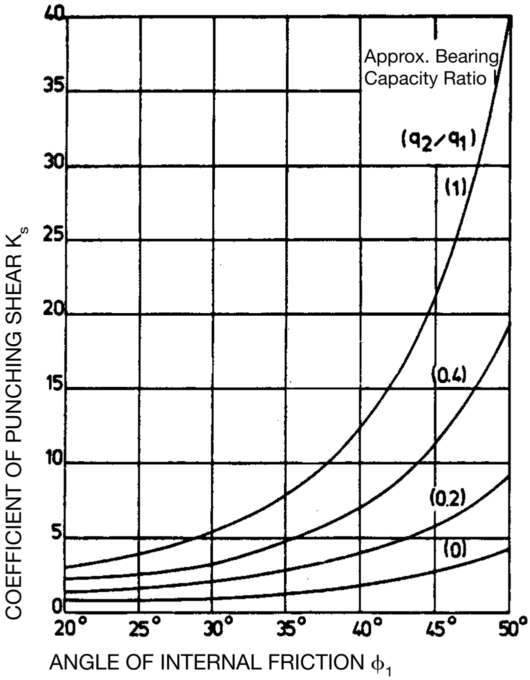


FIGURE 6A.5 Coefficients of punching shear resistance under vertical load (from Meyerhof and Hanna, 1978).

If the replaced zone punches through the matrix soil [Fig. 6A.2(*c*)], punching resistance will develop within the in situ soil adjacent to the vertical perimeter surface of the replaced zone. The following equation applies to this case:

$$q_b = q_b \cdot \frac{A_{rz}}{A_f} + \frac{pP_h \tan \phi_2 + pHc_2 - W_{rz}}{A_f} \leq q_{rz} \tag{6A.6a}$$

where q_b = ultimate bearing capacity of the in situ soil beneath the replaced zone based on the dimensions of the replaced zone

p = perimeter length of the replaced zone = $2(W + L_{rz})$ for a rectangular replaced zone (where L_{rz} is the length of the replaced zone) and πd_{rz} for a circular replaced zone (where d_{rz} is the diameter of the replaced zone)

P_h = lateral earth pressure thrust (force per unit length) acting along the perimeter surface of the replaced zone at failure = $\int_0^H K_s \sigma_v dH$

K_s = lateral earth pressure coefficient along the perimeter surface of the replaced zone at failure

W_{rz} = weight of the material in the replaced zone

A_{rz} = area of the replaced zone = WL_{rz} for a rectangular zone and $0.25\pi d_{rz}^2$ for a circular zone

q_{rz} = ultimate bearing capacity for general shear failure within the replaced zone [from Eq. (6A.3*c*)]

If the ground-water table is below the lowest point on the potential failure surface, the following equations apply:

$$q_b = c_2 N_{c2} S_{c2} D_{c2} + \gamma_2 (D + H) N_{q2} S_{q2} D_{q2} + 0.5 \gamma_2 W N_{\gamma 2} S_{\gamma 2} D_{\gamma 2} \tag{6A.6b}$$

where N_{c2}, N_{q2}, $N_{\gamma 2}$ are bearing capacity factors for general shear failure in the in situ soil beneath the replaced zone based on ϕ_2 and S_{c2}, S_{q2}, $S_{\gamma 2}$ are shape factors and D_{c2}, D_{q2}, $D_{\gamma 2}$ are depth factors based on the dimensions of the replaced zone and an embedded depth of $(D + H)$. In addition,

$$W_{rz} = W L_{rz} H \gamma_1 \qquad \text{for a rectangular replaced zone} \tag{6A.6c.1}$$

$$W_{rz} = 0.25 \pi d_{rz}^2 H \gamma_1 \qquad \text{for a circular replaced zone} \tag{6A.6c.2}$$

If K_s is assumed to be constant at all depths along the perimeter surface of the replaced zone, the following equation applies for P_h:

$$P_h = K_s(\gamma_2 DH + 0.5\gamma_2 H^2) = K_s \gamma_2 (DH + 0.5H^2) \tag{6A.6d}$$

Values for K_s can be obtained from Fig. 6A.5 for $q_1 = q_2$ (since passive resistance develops within the in situ soil) or by assuming $K_s = K_{0-nc}$ and using the following equation:

$$K_s = K_{0-nc} = 1 - \sin \phi_2 \tag{6A.7}$$

The method employed to obtain the friction angles used in Eqs. (6A.3) through (6A.7) should model the type of failure assumed in each equation. Values of ϕ_1 in Eq. (6A.3*a*) and ϕ_2 in Eq. (6A.6*a*) obtained from direct shear tests would be appropriate. Values of ϕ_1 or ϕ_2 used to calculate factors in the other equations should be axisymmetric (triaxial) values if the foundation or replaced zone is square or circular or plane-strain if rectangular and L/B or L_{rz}/W is greater than about 5. Most values for ϕ found in tables based on relative density or Standard Penetration Test (SPT) blow-counts are correlated to triaxial values. Conservative values for plane-strain friction angle (ϕ_{ps}) can be estimated from triaxial values (ϕ_{tx}) using the following equations (Lade and Lee, 1976):

$$\phi_{ps} = \phi_{tx} \qquad \text{for } \phi_{tx} \leq 34° \tag{6A.8a}$$

$$\phi_{ps} = 1.5\phi_{tx} - 17° \qquad \text{for } \phi_{tx} > 34° \tag{6A.8b}$$

Values of ϕ from direct shear tests are usually about 1° or 2° greater than ϕ_{tx} for the same range of confining stresses.

6A.2.2 Settlement

When properly designed and implemented, overexcavation and replacement results in reduced settlement owing to one or both of the following factors: (*a*) The replaced zone is stiffer so it settles less than the replaced in situ soil would have, and (*b*) the vertical stresses induced in the in situ soil beneath the replaced zone may be less than without the replaced zone, so the settlement of this underlying soil may also be less. As will be discussed subsequently, the vertical stresses induced in the underlying in situ soil may be greater with a replaced zone than without it, so care must be exercised when using overexcavation/replacement where saturated fine-grained soils are within the zone of influence for settlement.

Several solutions are available in the literature to calculate settlement for a uniform, circular load on the surface of two- and three-layer elastic systems (e.g. Burmister, 1945; Burmister, 1962; Thenn de Barros, 1966; Ueshita and Meyerhof, 1967). The solutions for two-layer systems take the following general form:

$$S_i = \frac{q_0 d_0}{E_2} I_s \tag{6A.9}$$

where q_0 = uniform stress applied to the surface of the uppermost layer
d_0 = diameter of the loaded area
E_2 = stress-strain modulus for the lower layer
I_s = influence factor for settlement beneath the center of the loaded area

The magnitude of I_s depends on the height of the stiff layer relative to the diameter of the loaded area (H/d_0), the modulus ratio for the two layers (E_2/E_1), and the Poisson's ratios for the two layers (ν_1, ν_2). Values of I_s for a two-layer system with ν_1 = 0.2 and ν_2 = 0.4 are given in Fig. 6A.6 (Burmister, 1962), from which it can be seen that settlement decreases as E_1/E_2 increases or H/d_0 increases.

The applicability of charts such as Fig. 6A.6 for estimating the settlement of a foundation bearing on a replaced zone is limited in the following ways:

1. The charts are not applicable to replaced zones of finite width unless the replaced zone is wide enough to get the full reduction in settlement.
2. Embedment of the foundation, which results in reduced settlement, is not considered.
3. The possible existence of a rigid base (such as bedrock) within the zone of influence, which also reduces settlement, is not considered.
4. The charts apply only to foundations with circular or square shapes. (Square foundations can be considered by converting to an equivalent circle with the same area.)

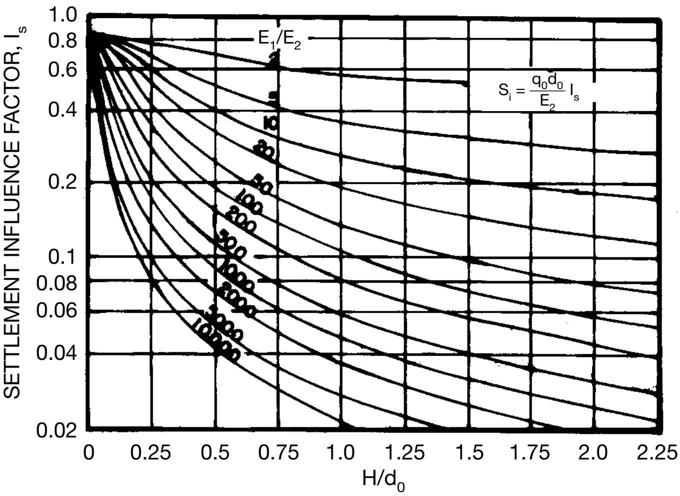
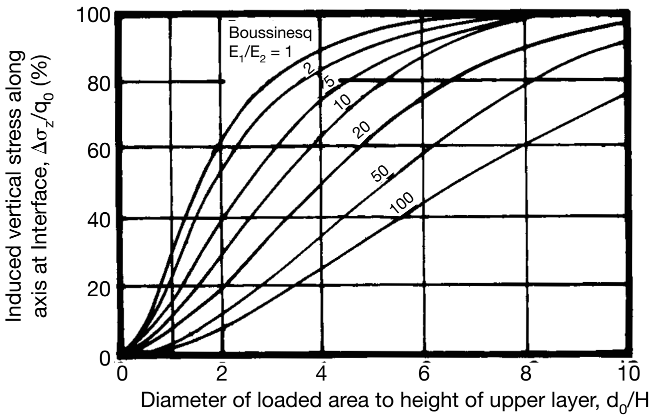


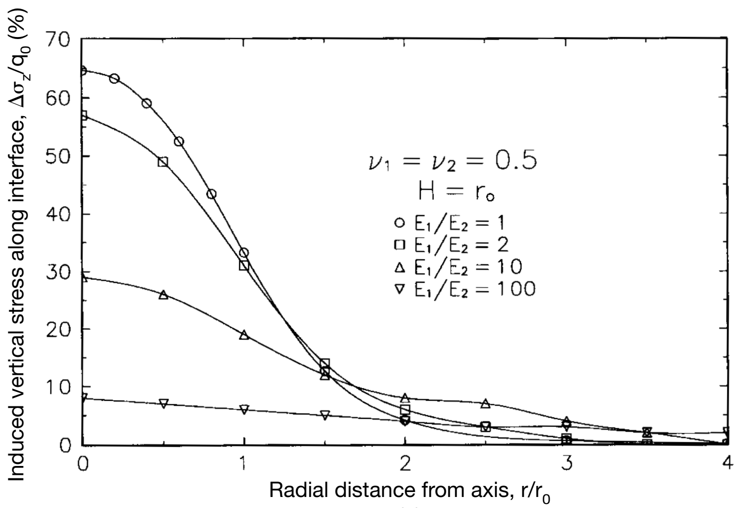
FIGURE 6A.6 Settlement influence factors for a circular, uniform load on the surface of a two-layer elastic system (ν_1 = 0.2, ν_2 = 0.4) (from Burmister, 1962).

No solutions are available in the literature to estimate settlement for a replaced zone of finite width or for infinitely wide layered soils when the foundation is either embedded or rectangular. Solutions are available for calculating immediate settlement for semi-infinite, homogeneous soils that consider the effect of depth of embedment for both circular and rectangular foundations [Nishida (1966) for circular and E. Fox (1948) for rectangular]. Settlement of a circular foundation bearing on the *surface* of a two-layer system underlain by a rough, rigid base can be estimated from the influence chart or tables given in Uzan and coworkers (1980). An extensive collection of elastic solutions for stresses and displacements of geologic materials can be found in Poulos and Davis (1974).

The induced vertical stresses ($\Delta\sigma_z$) in a two-layer elastic system subjected to a uniform, circular stress on the surface of the upper layer are shown in Fig. 6A.7 for $\nu_1 = \nu_2 = 0.5$ and the following two



(a)



(b)

FIGURE 6A.7 Induced stresses along the interface of a two-layer elastic system subjected to a circular, uniform load on the surface ($\nu_1 = \nu_2 = 0.5$): (*a*) along axis for varying d_0/H and E_1/E_2 (from Burmister, 1958); (*b*) as a function of r/r_0 and E_1/E_2 for $H = r_0$ (data from L. Fox, 1948).

distributions: (*a*) Along the axis at the interface between the two layers as a function of d_0/H and E_1/E_2, and (*b*) horizontal distribution along the interface for $H = r_0$ and varying E_1/E_2. Figure 6A.7 shows that increasing either the height or the stiffness of the upper layer distributes the load over a larger area and reduces the maximum $\Delta\sigma_z$ (along the axis of the loaded area). To obtain an estimate of the approximate slope of the vertical stress distribution [α(vertical):1(horizontal)] as a function of E_1/E_2, the curves in Fig. 6A.7 were integrated numerically by the author to determine the radial distance within which 95% of the applied load is distributed, with the results shown in Fig. 6A.8. This technique is similar in concept to the 2:1 or 60° stress distribution methods. Thus, an estimate of the minimum dimensions of the replaced zone needed to obtain the full reduction in settlement can be determined with the aid of Fig. 6A.8 and the following equations. For a rectangular foundation use

$$W \geq B + \frac{2H}{\alpha} \tag{6A.10a.l}$$

$$L_{rz} \geq L + \frac{2H}{\alpha} \tag{6A.10a2}$$

For a circular foundation use

$$d_{rz} \geq d_0 + \frac{2H}{\alpha} \tag{6A.10b}$$

In many cases where overexcavation/replacement is used, the replaced zone is not wide enough to develop the full reduction in settlement. As an example, consider the case of a circular foundation bearing on a cylindrical replaced zone with $H = d_0$, and $E_1/E_2 = 20$. From Fig. 6A.8:

$$\alpha \cong 0.42$$

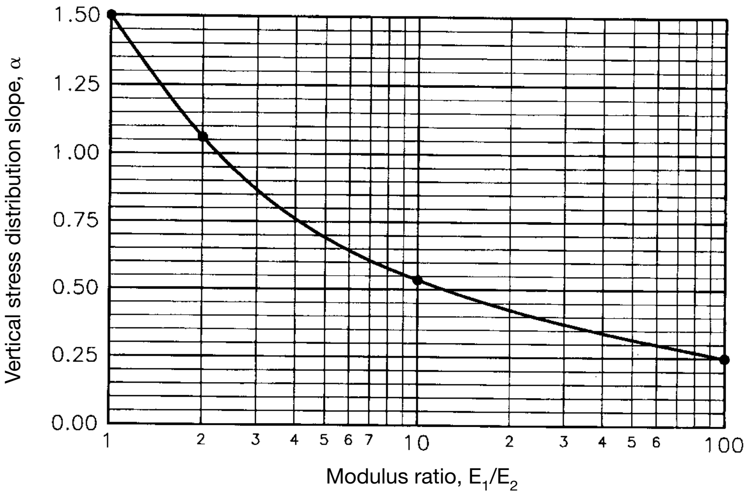


FIGURE 6A.8 Vertical stress distribution slope for a two-layer elastic system.

From Eq. (6A.10*b*):

$$d_{rz} \geq d_0 + \frac{2d_0}{0.42} = 5.8d_0$$

d_{rz} is typically in the range of d_0 to $3d_0$, which is less than the minimum diameter of about $5.8d_0$ required for full reduction in settlement. Estimating S_i from Fig. 6A.6 and Eq. (6A.9) for this case would therefore tend to underestimate the settlement.

To determine the influences of depth of embedment and width, height, and stiffness of the replaced zone on S_i, a parametric finite element study was performed by the author for a flexible, uniform, circular stress applied to the surface of a cylindrical replaced zone ($v_1 = 0.2$) within an otherwise homogeneous, isotropic, elastic soil mass ($v_2 = 0.4$). In this investigation, the following values of parameters were used:

$$\frac{E_1}{E_2} = 5, 10, 20, 50, \text{ and } 100$$

$$\frac{D}{d_0} = 0, 0.5, 1.0, \text{ and } 1.5$$

$$\frac{H}{d_0} = 0.5, 1.0, \text{ and } 1.5$$

$$\frac{d_{rz}}{d_0} = 0.25, 0.5, 0.75, 1, 2, 3, 4, 6, 8, \text{ and } 10$$

The results are presented in Table 6A.1 in the form of settlement influence factors (I_s), which can be used in Eq. (6A.9) to estimate S_i for an embedded circular or square foundation bearing on a finite-width replaced zone within a relatively homogeneous in situ soil. Values of I_s are given for settlement at the center of the loaded area and the mean settlement over the entire loaded area. The values of mean I_s represent a better estimate for real foundations, which are neither perfectly flexible nor perfectly rigid. The values of I_s at the center of the loaded area for $d_{rz}/d_0 = 10$ in Table 6A.1 match the corresponding theoretical values for an infinitely wide upper layer obtained from Fig. 6A.6. In addition, the values of central I_s for a homogeneous soil ($E_1/E_2 = 1$) are within 2% of the theoretical values obtained from Nishida's (1966) equations for a uniform, circular stress embedded within a homogeneous half-space. These comparisons indicate that the finite element results are reasonable.

The values of $v_1 = 0.2$ and $v_2 = 0.4$ used in the parametric study were selected as reasonable estimates for most overexcavation/replacement cases. For other values of v~ and v_2, adjustments may be needed to the values shown in Table 6A.1. A limited investigation has indicated that v_2 has a significant influence on S but that v_1 has very little effect. This can be illustrated by the following finite element results for $E_1/E_2 = 20$, $D/d_0 = 1$, $H/d_0 = 1$, and $d_{rz}/d_0 = 2$:

v_1	v_2	I_s center	I_s mean
0.2	0.4	0.24	0.23
0.3	0.4	0.23	0.23
0.4	0.4	0.23	0.23
0.2	0.3	0.29	0.28
0.3	0.3	0.28	0.28
0.4	0.3	0.28	0.28

v_1	v_2	I_s center	I_s mean
0.2	0.2	0.31	0.31
0.3	0.2	0.31	0.31
0.4	0.2	0.31	0.30

Therefore, values of I_s from the above table can be used for $v_2 = 0.4$ and any reasonable value of v_1. Preliminary results suggest that for $v_2 \neq 0.4$, a *rough* estimate for I_s can be obtained from the following equation:

$$I_{s(v_2 \neq 0.4)} = \frac{1 - v_2}{1 - 0.4} \cdot I_{s(v_2 = 0.4)} \tag{6A.11}$$

For homogeneous soils, I_s is proportional to $(1 - v_2)$, so the immediate settlement for overexcavated and replaced soils appears to be affected more by the Poisson's ratio of the matrix soil than are homogeneous soils.

I_s also depends on the shape of the loaded area. Reasonable values of I_s for square foundations and rectangular foundations with IJB less than about 1.5 can be obtained by converting to an equivalent circle with the same area. However, this method is not applicable to higher *L/B* ratios. To illustrate the effect that shape of the loaded area has on I_s, results from plane-strain finite element analyses (strip loading on an infinitely long replaced zone) are compared with axisymmetric results for the case of $v_1 = 0.2$, $v_2 = 0.4$, $E_1/E_2 = 20$, $D/d_0 = D/B = 1$, and $H/d_0 = H/B = 1$ as follows:

Axisymmetric [$S_i = (q_0 d_0/E_2)I_s$]

d_{rz}/d_0	I_s center	I_s average
0.25	0.41	0.44
0.5	0.38	0.40
0.75	0.35	0.35
1	0.32	0.32
1.5	0.27	0.26
2	0.24	0.23
3	0.21	0.20
4	0.20	0.19
6	0.19	0.18
8	0.19	0.18
10	0.18	0.18

Plane-Strain [$S_i = (q_0 B/E_2)I_s$]

W/B	I_s center	I_s average
0.25	0.16	0.16
0.5	0.15	0.15
0.75	0.15	0.15
1	0.14	0.14
1.5	0.14	0.14
2	0.13	0.13
3	0.13	0.13
4	0.12	0.12
6	0.12	0.12
8	0.12	0.12
10	0.12	0.12

Therefore, for comparable conditions, a long rectangular foundation will settle substantially less than a circular foundation with the same width ($d_0 = B$). No additional information is currently available for estimating S_i for rectangular foundations of finite length bearing on replaced zones of finite extent, so engineering judgment is required for these cases. A very conservative estimate for I_s could be obtained by using Table 6A.1 with $D/d_0 = D/B$, $D/d_0 = H/B$, and $d_{rz}/d_0 = W/B$. One could also perform plane-strain finite element analyses for comparable conditions to obtain a lower-bound value for S_i, with S_i from the axisymmetric case as an upper-bound value, and use engineering judgment to estimate S_i for the actual *L/B* ratio. A better method would be to perform a three-dimensional finite element analysis for the actual conditions; unfortunately, few engineers have ready access to a three-dimensional finite element code suited to this type of problem.

If the in situ bearing soils within the depth of influence for settlement are highly stratified with

TABLE 6A.1 Settlement Influence Factors for a Flexible, Uniform, Circular Stress Applied to the Surface of a Cylindrical Replaced Zone within an Otherwise Homogeneous, Isotropic, Elastic Half-Space ($v_1 = 0.2$, $v_2 = 0.4$)

$$S_i = \frac{q_0 d_0}{E_2} I_s$$

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
1	0	0	0	0.84	0.71
1	0.5	0	0	0.65	0.59
1	1	0	0	0.54	0.50
1	1.5	0	0	0.50	0.46
5	0	0.5	0.25	0.71	0.69
5	0	0.5	0.5	0.65	0.65
5	0	0.5	0.75	0.61	0.59
5	0	0.5	1	0.57	0.54
5	0	0.5	1.5	0.53	0.49
5	0	0.5	2	0.51	0.47
5	0	0.5	3	0.50	0.46
5	0	0.5	4	0.50	0.46
5	0	0.5	6	0.50	0.46
5	0	0.5	8	0.50	0.46
5	0	0.5	10	0.50	0.46
5	0	1	0.25	0.67	0.67
5	0	1	0.5	0.60	0.60
5	0	1	0.75	0.54	0.53
5	0	1	1	0.49	0.47
5	0	1	1.5	0.43	0.41
5	0	1	2	0.39	0.37
5	0	1	3	0.37	0.35
5	0	1	4	0.37	0.34
5	0	1	6	0.36	0.34
5	0	1	8	0.36	0.34
5	0	1	10	0.36	0.33
5	0	1.5	0.25	0.66	0.66
5	0	1.5	0.5	0.57	0.58
5	0	1.5	0.75	0.51	0.50
5	0	1.5	1	0.46	0.44
5	0	1.5	1.5	0.40	0.38
5	0	1.5	2	0.36	0.34
5	0	1.5	3	0.33	0.31
5	0	1.5	4	0.32	0.30
5	0	1.5	6	0.31	0.29
5	0	1.5	8	0.31	0.29
5	0	1.5	10	0.31	0.28
5	0.5	0.5	0.25	0.57	0.56
5	0.5	0.5	0.5	0.54	0.53
5	0.5	0.5	0.75	0.51	0.49
5	0.5	0.5	1	0.48	0.46
5	0.5	0.5	1.5	0.44	0.42
5	0.5	0.5	2	0.43	0.40
5	0.5	0.5	3	0.42	0.39
5	0.5	0.5	4	0.42	0.39
5	0.5	0.5	6	0.42	0.39
5	0.5	0.5	8	0.42	0.39
5	0.5	0.5	10	0.42	0.39

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
5	0.5	1	0.25	0.55	0.55
5	0.5	1	0.5	0.51	0.50
5	0.5	1	0.75	0.46	0.45
5	0.5	1	1	0.43	0.41
5	0.5	1	1.5	0.38	0.36
5	0.5	1	2	0.35	0.33
5	0.5	1	3	0.33	0.31
5	0.5	1	4	0.32	0.30
5	0.5	1	6	0.32	0.30
5	0.5	1	8	0.32	0.30
5	0.5	1	10	0.32	0.30
5	0.5	1.5	0.25	0.55	0.55
5	0.5	1.5	0.5	0.49	0.49
5	0.5	1.5	0.75	0.44	0.44
5	0.5	1.5	1	0.41	0.39
5	0.5	1.5	1.5	0.35	0.34
5	0.5	1.5	2	0.33	0.31
5	0.5	1.5	3	0.30	0.28
5	0.5	1.5	4	0.29	0.27
5	0.5	1.5	6	0.28	0.26
5	0.5	1.5	8	0.28	0.26
5	0.5	1.5	10	0.28	0.26
5	1	0.5	0.25	0.49	0.48
5	1	0.5	0.5	0.46	0.46
5	1	0.5	0.75	0.45	0.43
5	1	0.5	1	0.42	0.40
5	1	0.5	1.5	0.40	0.37
5	1	0.5	2	0.39	0.36
5	1	0.5	3	0.38	0.35
5	1	0.5	4	0.38	0.35
5	1	0.5	6	0.38	0.35
5	1	0.5	8	0.38	0.35
5	1	0.5	10	0.38	0.35
5	1	1	0.25	0.47	0.47
5	1	1	0.5	0.44	0.44
5	1	1	0.75	0.41	0.40
5	1	1	1	0.38	0.37
5	1	1	1.5	0.35	0.33
5	1	1	2	0.32	0.31
5	1	1	3	0.31	0.29
5	1	1	4	0.30	0.28
5	1	1	6	0.30	0.28
5	1	1	8	0.30	0.28
5	1	1	10	0.29	0.28

TABLE 6A.1 (*Continued*)

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
5	1	1.5	0.25	0.47	0.47
5	1	1.5	0.5	0.43	0.43
5	1	1.5	0.75	0.39	0.39
5	1	1.5	1	0.37	0.35
5	1	1.5	1.5	0.33	0.31
5	1	1.5	2	0.30	0.29
5	1	1.5	3	0.28	0.26
5	1	1.5	4	0.27	0.25
5	1	1.5	6	0.26	0.25
5	1	1.5	8	0.26	0.25
5	1	1.5	10	0.26	0.24
5	1.5	0.5	0.25	0.45	0.45
5	1.5	0.5	0.5	0.43	0.43
5	1.5	0.5	0.75	0.42	0.41
5	1.5	0.5	1	0.40	0.38
5	1.5	0.5	1.5	0.37	0.35
5	1.5	0.5	2	0.36	0.34
5	1.5	0.5	3	0.36	0.34
5	1.5	0.5	4	0.36	0.33
5	1.5	0.5	6	0.36	0.33
5	1.5	0.5	8	0.36	0.33
5	1.5	0.5	10	0.36	0.33
5	1.5	1	0.25	0.44	0.44
5	1.5	1	0.5	0.41	0.41
5	1.5	1	0.75	0.38	0.38
5	1.5	1	1	0.36	0.35
5	1.5	1	1.5	0.33	0.31
5	1.5	1	2	0.31	0.29
5	1.5	1	3	0.29	0.28
5	1.5	1	4	0.29	0.27
5	1.5	1	6	0.29	0.27
5	1.5	1	8	0.28	0.27
5	1.5	1	10	0.28	0.27
5	1.5	1.5	0.25	0.44	0.44
5	1.5	1.5	0.5	0.40	0.40
5	1.5	1.5	0.75	0.37	0.37
5	1.5	1.5	1	0.35	0.33
5	1.5	1.5	1.5	0.31	0.29
5	1.5	1.5	2	0.29	0.27
5	1.5	1.5	3	0.27	0.26
5	1.5	1.5	4	0.26	0.25
5	1.5	1.5	6	0.26	0.24
5	1.5	1.5	8	0.25	0.24
5	1.5	1.5	10	0.25	0.24
10	0	0.5	0.25	0.68	0.68
10	0	0.5	0.5	0.61	0.62
10	0	0.5	0.75	0.57	0.56
10	0	0.5	1	0.51	0.50
10	0	0.5	1.5	0.45	0.43
10	0	0.5	2	0.42	0.40
10	0	0.5	3	0.41	0.39
10	0	0.5	4	0.40	0.38
10	0	0.5	6	0.40	0.38
10	0	0.5	8	0.40	0.38
10	0	0.5	10	0.40	0.38

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
10	0	1	0.25	0.61	0.64
10	0	1	0.5	0.53	0.55
10	0	1	0.75	0.48	0.48
10	0	1	1	0.43	0.42
10	0	1	1.5	0.35	0.34
10	0	1	2	0.32	0.30
10	0	1	3	0.28	0.27
10	0	1	4	0.27	0.26
10	0	1	6	0.27	0.25
10	0	1	8	0.26	0.25
10	0	1	10	0.26	0.25
10	0	1.5	0.25	0.59	0.62
10	0	1.5	0.5	0.49	0.52
10	0	1.5	0.75	0.43	0.44
10	0	1.5	1	0.39	0.38
10	0	1.5	1.5	0.32	0.31
10	0	1.5	2	0.28	0.27
10	0	1.5	3	0.24	0.23
10	0	1.5	4	0.23	0.22
10	0	1.5	6	0.22	0.21
10	0	1.5	8	0.21	0.20
10	0	1.5	10	0.21	0.20
10	0.5	0.5	0.25	0.55	0.55
10	0.5	0.5	0.5	0.51	0.51
10	0.5	0.5	0.75	0.48	0.47
10	0.5	0.5	1	0.44	0.42
10	0.5	0.5	1.5	0.39	0.37
10	0.5	0.5	2	0.37	0.35
10	0.5	0.5	3	0.36	0.34
10	0.5	0.5	4	0.36	0.34
10	0.5	0.5	6	0.35	0.33
10	0.5	0.5	8	0.35	0.33
10	0.5	0.5	10	0.35	0.33
10	0.5	1	0.25	0.51	0.53
10	0.5	1	0.5	0.46	0.47
10	0.5	1	0.75	0.42	0.41
10	0.5	1	1	0.38	0.37
10	0.5	1	1.5	0.32	0.31
10	0.5	1	2	0.29	0.28
10	0.5	1	3	0.26	0.25
10	0.5	1	4	0.25	0.24
10	0.5	1	6	0.25	0.24
10	0.5	1	8	0.24	0.23
10	0.5	1	10	0.24	0.23
10	0.5	1.5	0.25	0.50	0.52
10	0.5	1.5	0.5	0.43	0.45
10	0.5	1.5	0.75	0.39	0.39
10	0.5	1.5	1	0.35	0.34
10	0.5	1.5	1.5	0.30	0.29
10	0.5	1.5	2	0.26	0.25
10	0.5	1.5	3	0.23	0.22
10	0.5	1.5	4	0.22	0.21
10	0.5	1.5	6	0.21	0.20
10	0.5	1.5	8	0.20	0.19
10	0.5	1.5	10	0.20	0.19

(*continued*)

TABLE 6A.1 (*Continued*)

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean	E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
10	1	0.5	0.25	0.47	0.47	10	1.5	1.5	0.25	0.40	0.42
10	1	0.5	0.5	0.44	0.45	10	1.5	1.5	0.5	0.36	0.38
10	1	0.5	0.75	0.42	0.41	10	1.5	1.5	0.75	0.33	0.33
10	1	0.5	1	0.39	0.38	10	1.5	1.5	1	0.30	0.30
10	1	0.5	1.5	0.35	0.34	10	1.5	1.5	1.5	0.26	0.25
10	1	0.5	2	0.34	0.32	10	1.5	1.5	2	0.24	0.23
10	1	0.5	3	0.33	0.31	10	1.5	1.5	3	0.21	0.20
10	1	0.5	4	0.32	0.31	10	1.5	1.5	4	0.20	0.19
10	1	0.5	6	0.32	0.30	10	1.5	1.5	6	0.19	0.18
10	1	0.5	8	0.32	0.30	10	1.5	1.5	8	0.19	0.18
10	1	0.5	10	0.32	0.30	10	1.5	1.5	10	0.19	0.18
10	1	1	0.25	0.44	0.46	20	0	0.5	0.25	0.65	0.66
10	1	1	0.5	0.40	0.42	20	0	0.5	0.5	0.59	0.61
10	1	1	0.75	0.37	0.37	20	0	0.5	0.75	0.54	0.53
10	1	1	1	0.34	0.34	20	0	0.5	1	0.48	0.47
10	1	1	1.5	0.30	0.29	20	0	0.5	1.5	0.40	0.38
10	1	1	2	0.27	0.26	20	0	0.5	2	0.36	0.35
10	1	1	3	0.25	0.24	20	0	0.5	3	0.34	0.33
10	1	1	4	0.24	0.23	20	0	0.5	4	0.33	0.32
10	1	1	6	0.23	0.22	20	0	0.5	6	0.33	0.31
10	1	1	8	0.23	0.22	20	0	0.5	8	0.32	0.31
10	1	1	10	0.23	0.22	20	0	0.5	10	0.32	0.31
10	1	1.5	0.25	0.43	0.45	20	0	1	0.25	0.56	0.61
10	1	1.5	0.5	0.38	0.40	20	0	1	0.5	0.49	0.52
10	1	1.5	0.75	0.35	0.35	20	0	1	0.75	0.44	0.44
10	1	1.5	1	0.32	0.31	20	0	1	1	0.39	0.39
10	1	1.5	1.5	0.27	0.27	20	0	1	1.5	0.31	0.31
10	1	1.5	2	0.25	0.24	20	0	1	2	0.27	0.26
10	1	1.5	3	0.22	0.21	20	0	1	3	0.23	0.22
10	1	1.5	4	0.21	0.20	20	0	1	4	0.22	0.21
10	1	1.5	6	0.20	0.19	20	0	1	6	0.21	0.20
10	1	1.5	8	0.20	0.19	20	0	1	8	0.20	0.20
10	1	1.5	10	0.19	0.18	20	0	1	10	0.20	0.19
10	1.5	0.5	0.25	0.44	0.44	20	0	1.5	0.25	0.52	0.58
10	1.5	0.5	0.5	0.41	0.42	20	0	1.5	0.5	0.44	0.48
10	1.5	0.5	0.75	0.39	0.39	20	0	1.5	0.75	0.39	0.39
10	1.5	0.5	1	0.37	0.35	20	0	1.5	1	0.35	0.34
10	1.5	0.5	1.5	0.33	0.32	20	0	1.5	1.5	0.28	0.27
10	1.5	0.5	2	0.32	0.30	20	0	1.5	2	0.24	0.23
10	1.5	0.5	3	0.31	0.29	20	0	1.5	3	0.20	0.19
10	1.5	0.5	4	0.31	0.29	20	0	1.5	4	0.18	0.17
10	1.5	0.5	6	0.31	0.29	20	0	1.5	6	0.17	0.16
10	1.5	0.5	8	0.31	0.29	20	0	1.5	8	0.16	0.16
10	1.5	0.5	10	0.31	0.29	20	0	1.5	10	0.15	0.15
10	1.5	1	0.25	0.41	0.43	20	0.5	0.5	0.25	0.53	0.54
10	1.5	1	0.5	0.38	0.39	20	0.5	0.5	0.5	0.50	0.50
10	1.5	1	0.75	0.35	0.35	20	0.5	0.5	0.75	0.46	0.45
10	1.5	1	1	0.32	0.32	20	0.5	0.5	1	0.41	0.41
10	1.5	1	1.5	0.28	0.27	20	0.5	0.5	1.5	0.35	0.34
10	1.5	1	2	0.26	0.25	20	0.5	0.5	2	0.33	0.31
10	1.5	1	3	0.24	0.23	20	0.5	0.5	3	0.31	0.30
10	1.5	1	4	0.23	0.22	20	0.5	0.5	4	0.30	0.29
10	1.5	1	6	0.23	0.21	20	0.5	0.5	6	0.30	0.29
10	1.5	1	8	0.22	0.21	20	0.5	0.5	8	0.30	0.29
10	1.5	1	10	0.22	0.21	20	0.5	0.5	10	0.30	0.28

TABLE 6A.1 (*Continued*)

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
20	0.5	1	0.25	0.48	0.50
20	0.5	1	0.5	0.43	0.45
20	0.5	1	0.75	0.39	0.39
20	0.5	1	1	0.35	0.35
20	0.5	1	1.5	0.29	0.28
20	0.5	1	2	0.25	0.25
20	0.5	1	3	0.22	0.21
20	0.5	1	4	0.21	0.20
20	0.5	1	6	0.20	0.19
20	0.5	1	8	0.19	0.19
20	0.5	1	10	0.19	0.18
20	0.5	1.5	0.25	0.45	0.49
20	0.5	1.5	0.5	0.39	0.41
20	0.5	1.5	0.75	0.35	0.35
20	0.5	1.5	1	0.32	0.31
20	0.5	1.5	1.5	0.26	0.26
20	0.5	1.5	2	0.23	0.22
20	0.5	1.5	3	0.19	0.19
20	0.5	1.5	4	0.17	0.17
20	0.5	1.5	6	0.16	0.16
20	0.5	1.5	8	0.16	0.15
20	0.5	1.5	10	0.15	0.15
20	1	0.5	0.25	0.46	0.47
20	1	0.5	0.5	0.43	0.44
20	1	0.5	0.75	0.41	0.40
20	1	0.5	1	0.37	0.36
20	1	0.5	1.5	0.32	0.31
20	1	0.5	2	0.30	0.29
20	1	0.5	3	0.29	0.27
20	1	0.5	4	0.28	0.27
20	1	0.5	6	0.28	0.27
20	1	0.5	8	0.28	0.26
20	1	0.5	10	0.28	0.26
20	1	1	0.25	0.41	0.44
20	1	1	0.5	0.38	0.40
20	1	1	0.75	0.35	0.35
20	1	1	1	0.32	0.32
20	1	1	1.5	0.27	0.26
20	1	1	2	0.24	0.23
20	1	1	3	0.21	0.20
20	1	1	4	0.20	0.19
20	1	1	6	0.19	0.18
20	1	1	8	0.19	0.18
20	1	1	10	0.18	0.18
20	1	1.5	0.25	0.39	0.43
20	1	1.5	0.5	0.35	0.37
20	1	1.5	0.75	0.32	0.32
20	1	1.5	1	0.29	0.29
20	1	1.5	1.5	0.24	0.24
20	1	1.5	2	0.21	0.21
20	1	1.5	3	0.18	0.18
20	1	1.5	4	0.17	0.16
20	1	1.5	6	0.16	0.15
20	1	1.5	8	0.15	0.15
20	1	1.5	10	0.15	0.14
20	1.5	0.5	0.25	0.43	0.44
20	1.5	0.5	0.5	0.40	0.41
20	1.5	0.5	0.75	0.38	0.38
20	1.5	0.5	1	0.35	0.34
20	1.5	0.5	1.5	0.30	0.29
20	1.5	0.5	2	0.28	0.27
20	1.5	0.5	3	0.27	0.26
20	1.5	0.5	4	0.27	0.25
20	1.5	0.5	6	0.26	0.25
20	1.5	0.5	8	0.26	0.25
20	1.5	0.5	10	0.26	0.25
20	1.5	1	0.25	0.38	0.41
20	1.5	1	0.5	0.36	0.37
20	1.5	1	0.75	0.33	0.33
20	1.5	1	1	0.30	0.30
20	1.5	1	1.5	0.25	0.25
20	1.5	1	2	0.23	0.22
20	1.5	1	3	0.20	0.19
20	1.5	1	4	0.19	0.18
20	1.5	1	6	0.18	0.18
20	1.5	1	8	0.18	0.17
20	1.5	1	10	0.18	0.17
20	1.5	1.5	0.25	0.37	0.40
20	1.5	1.5	0.5	0.33	0.35
20	1.5	1.5	0.75	0.30	0.30
20	1.5	1.5	1	0.28	0.27
20	1.5	1.5	1.5	0.23	0.23
20	1.5	1.5	2	0.21	0.20
20	1.5	1.5	3	0.18	0.17
20	1.5	1.5	4	0.16	0.16
20	1.5	1.5	6	0.15	0.15
20	1.5	1.5	8	0.15	0.15
20	1.5	1.5	10	0.15	0.14
50	0	0.5	0.25	0.63	0.65
50	0	0.5	0.5	0.58	0.60
50	0	0.5	0.75	0.52	0.52
50	0	0.5	1	0.45	0.45
50	0	0.5	1.5	0.36	0.35
50	0	0.5	2	0.31	0.30
50	0	0.5	3	0.27	0.26
50	0	0.5	4	0.26	0.25
50	0	0.5	6	0.25	0.25
50	0	0.5	8	0.25	0.24
50	0	0.5	10	0.24	0.24
50	0	1	0.25	0.52	0.58
50	0	1	0.5	0.47	0.50
50	0	1	0.75	0.42	0.42
50	0	1	1	0.37	0.37
50	0	1	1.5	0.29	0.29
50	0	1	2	0.24	0.24
50	0	1	3	0.19	0.19
50	0	1	4	0.17	0.17
50	0	1	6	0.16	0.15
50	0	1	8	0.15	0.15
50	0	1	10	0.14	0.14

(*continued*)

TABLE 6A.1 (*Continued*)

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean	E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
50	0	1.5	0.25	0.46	0.54	50	1	1	0.25	0.39	0.43
50	0	1.5	0.5	0.40	0.45	50	1	1	0.5	0.36	0.39
50	0	1.5	0.75	0.36	0.36	50	1	1	0.75	0.34	0.34
50	0	1.5	1	0.32	0.32	50	1	1	1	0.30	0.30
50	0	1.5	1.5	0.25	0.25	50	1	1	1.5	0.25	0.24
50	0	1.5	2	0.21	0.21	50	1	1	2	0.21	0.21
50	0	1.5	3	0.17	0.17	50	1	1	3	0.18	0.17
50	0	1.5	4	0.15	0.14	50	1	1	4	0.16	0.16
50	0	1.5	6	0.13	0.13	50	1	1	6	0.15	0.15
50	0	1.5	8	0.12	0.12	50	1	1	8	0.14	0.14
50	0	1.5	10	0.12	0.11	50	1	1	10	0.14	0.13
50	0.5	0.5	0.25	0.52	0.53	50	1	1.5	0.25	0.36	0.40
50	0.5	0.5	0.5	0.49	0.49	50	1	1.5	0.5	0.33	0.35
50	0.5	0.5	0.75	0.45	0.44	50	1	1.5	0.75	0.30	0.30
50	0.5	0.5	1	0.39	0.39	50	1	1.5	1	0.27	0.27
50	0.5	0.5	1.5	0.32	0.32	50	1	1.5	1.5	0.22	0.22
50	0.5	0.5	2	0.28	0.28	50	1	1.5	2	0.19	0.19
50	0.5	0.5	3	0.26	0.25	50	1	1.5	3	0.16	0.16
50	0.5	0.5	4	0.25	0.24	50	1	1.5	4	0.14	0.14
50	0.5	0.5	6	0.24	0.23	50	1	1.5	6	0.12	0.12
50	0.5	0.5	8	0.24	0.23	50	1	1.5	8	0.12	0.12
50	0.5	0.5	10	0.23	0.23	50	1	1.5	10	0.11	0.11
50	0.5	1	0.25	0.45	0.48	50	1.5	0.5	0.25	0.42	0.43
50	0.5	1	0.5	0.41	0.43	50	1.5	0.5	0.5	0.40	0.41
50	0.5	1	0.75	0.37	0.37	50	1.5	0.5	0.75	0.37	0.37
50	0.5	1	1	0.33	0.33	50	1.5	0.5	1	0.33	0.33
50	0.5	1	1.5	0.27	0.26	50	1.5	0.5	1.5	0.27	0.27
50	0.5	1	2	0.23	0.22	50	1.5	0.5	2	0.25	0.24
50	0.5	1	3	0.19	0.18	50	1.5	0.5	3	0.23	0.22
50	0.5	1	4	0.17	0.16	50	1.5	0.5	4	0.22	0.21
50	0.5	1	6	0.15	0.15	50	1.5	0.5	6	0.22	0.21
50	0.5	1	8	0.15	0.15	50	1.5	0.5	8	0.21	0.21
50	0.5	1	10	0.14	0.14	50	1.5	0.5	10	0.21	0.21
50	0.5	1.5	0.25	0.41	0.46	50	1.5	1	0.25	0.36	0.40
50	0.5	1.5	0.5	0.36	0.39	50	1.5	1	0.5	0.34	0.36
50	0.5	1.5	0.75	0.33	0.33	50	1.5	1	0.75	0.32	0.32
50	0.5	1.5	1	0.29	0.29	50	1.5	1	1	0.29	0.28
50	0.5	1.5	1.5	0.24	0.24	50	1.5	1	1.5	0.23	0.23
50	0.5	1.5	2	0.20	0.20	50	1.5	1	2	0.20	0.20
50	0.5	1.5	3	0.16	0.16	50	1.5	1	3	0.17	0.17
50	0.5	1.5	4	0.14	0.14	50	1.5	1	4	0.16	0.15
50	0.5	1.5	6	0.13	0.12	50	1.5	1	6	0.15	0.14
50	0.5	1.5	8	0.12	0.12	50	1.5	1	8	0.14	0.14
50	0.5	1.5	10	0.12	0.11	50	1.5	1	10	0.14	0.13
50	1	0.5	0.25	0.45	0.46	50	1.5	1.5	0.25	0.33	0.38
50	1	0.5	0.5	0.42	0.43	50	1.5	1.5	0.5	0.31	0.33
50	1	0.5	0.75	0.40	0.39	50	1.5	1.5	0.75	0.29	0.29
50	1	0.5	1	0.35	0.35	50	1.5	1.5	1	0.26	0.26
50	1	0.5	1.5	0.29	0.29	50	1.5	1.5	1.5	0.21	0.21
50	1	0.5	2	0.26	0.25	50	1.5	1.5	2	0.18	0.18
50	1	0.5	3	0.24	0.23	50	1.5	1.5	3	0.15	0.15
50	1	0.5	4	0.23	0.22	50	1.5	1.5	4	0.14	0.13
50	1	0.5	6	0.23	0.22	50	1.5	1.5	6	0.12	0.12
50	1	0.5	8	0.22	0.22	50	1.5	1.5	8	0.12	0.12
50	1	0.5	10	0.22	0.21	50	1.5	1.5	10	0.11	0.11

TABLE 6A.1 (*Continued*)

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean	E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
100	0	0.5	0.25	0.63	0.65	100	0.5	1.5	0.25	0.38	0.44
100	0	0.5	0.5	0.58	0.59	100	0.5	1.5	0.5	0.35	0.38
100	0	0.5	0.75	0.52	0.51	100	0.5	1.5	0.75	0.32	0.32
100	0	0.5	1	0.44	0.44	100	0.5	1.5	1	0.28	0.28
100	0	0.5	1.5	0.34	0.34	100	0.5	1.5	1.5	0.23	0.23
100	0	0.5	2	0.28	0.28	100	0.5	1.5	2	0.19	0.19
100	0	0.5	3	0.23	0.23	100	0.5	1.5	3	0.15	0.15
100	0	0.5	4	0.22	0.21	100	0.5	1.5	4	0.13	0.13
100	0	0.5	6	0.21	0.21	100	0.5	1.5	6	0.11	0.11
100	0	0.5	8	0.21	0.20	100	0.5	1.5	8	0.11	0.10
100	0	0.5	10	0.20	0.19	100	0.5	1.5	10	0.10	0.10
100	0	1	0.25	0.50	0.57	100	1	0.5	0.25	0.44	0.46
100	0	1	0.5	0.46	0.49	100	1	0.5	0.5	0.42	0.43
100	0	1	0.75	0.41	0.41	100	1	0.5	0.75	0.39	0.39
100	0	1	1	0.36	0.36	100	1	0.5	1	0.35	0.35
100	0	1	1.5	0.28	0.28	100	1	0.5	1.5	0.28	0.28
100	0	1	2	0.23	0.23	100	1	0.5	2	0.24	0.24
100	0	1	3	0.18	0.18	100	1	0.5	3	0.21	0.21
100	0	1	4	0.15	0.15	100	1	0.5	4	0.20	0.20
100	0	1	6	0.13	0.13	100	1	0.5	6	0.19	0.19
100	0	1	8	0.13	0.13	100	1	0.5	8	0.19	0.19
100	0	1	10	0.12	0.12	100	1	0.5	10	0.18	0.18
100	0	1.5	0.25	0.43	0.52	100	1	1	0.25	0.38	0.42
100	0	1.5	0.5	0.39	0.43	100	1	1	0.5	0.36	0.38
100	0	1.5	0.75	0.35	0.35	100	1	1	0.75	0.33	0.33
100	0	1.5	1	0.31	0.31	100	1	1	1	0.30	0.30
100	0	1.5	1.5	0.25	0.24	100	1	1	1.5	0.24	0.24
100	0	1.5	2	0.20	0.20	100	1	1	2	0.20	0.20
100	0	1.5	3	0.16	0.16	100	1	1	3	0.16	0.16
100	0	1.5	4	0.13	0.13	100	1	1	4	0.14	0.14
100	0	1.5	6	0.11	0.11	100	1	1	6	0.13	0.13
100	0	1.5	8	0.11	0.11	100	1	1	8	0.12	0.12
100	0	1.5	10	0.10	0.10	100	1	1	10	0.12	0.12
100	0.5	0.5	0.25	0.52	0.53	100	1	1.5	0.25	0.34	0.39
100	0.5	0.5	0.5	0.48	0.49	100	1	1.5	0.5	0.32	0.34
100	0.5	0.5	0.75	0.44	0.44	100	1	1.5	0.75	0.29	0.29
100	0.5	0.5	1	0.39	0.39	100	1	1.5	1	0.26	0.26
100	0.5	0.5	1.5	0.31	0.30	100	1	1.5	1.5	0.22	0.21
100	0.5	0.5	2	0.26	0.26	100	1	1.5	2	0.18	0.18
100	0.5	0.5	3	0.22	0.22	100	1	1.5	3	0.15	0.15
100	0.5	0.5	4	0.21	0.21	100	1	1.5	4	0.13	0.13
100	0.5	0.5	6	0.20	0.20	100	1	1.5	6	0.11	0.11
100	0.5	0.5	8	0.20	0.19	100	1	1.5	8	0.11	0.10
100	0.5	0.5	10	0.19	0.19	100	1	1.5	10	0.10	0.10
100	0.5	1	0.25	0.43	0.47	100	1.5	0.5	0.25	0.41	0.43
100	0.5	1	0.5	0.40	0.42	100	1.5	0.5	0.5	0.39	0.41
100	0.5	1	0.75	0.37	0.37	100	1.5	0.5	0.75	0.37	0.37
100	0.5	1	1	0.33	0.32	100	1.5	0.5	1	0.33	0.32
100	0.5	1	1.5	0.26	0.26	100	1.5	0.5	1.5	0.26	0.26
100	0.5	1	2	0.22	0.22	100	1.5	0.5	2	0.23	0.22
100	0.5	1	3	0.17	0.17	100	1.5	0.5	3	0.20	0.20
100	0.5	1	4	0.15	0.15	100	1.5	0.5	4	0.19	0.19
100	0.5	1	6	0.13	0.13	100	1.5	0.5	6	0.19	0.18
100	0.5	1	8	0.13	0.12	100	1.5	0.5	8	0.18	0.18
100	0.5	1	10	0.12	0.12	100	1.5	0.5	10	0.18	0.17

(*continued*)

TABLE 6A.1 (*Continued*)

E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean	E_1/E_2	D/d_0	H/d_0	d_{rz}/d_0	I_s center	I_s mean
100	1.5	1	0.25	0.35	0.39	100	1.5	1.5	0.25	0.32	0.37
100	1.5	1	0.5	0.34	0.36	100	1.5	1.5	0.5	0.30	0.33
100	1.5	1	0.75	0.31	0.31	100	1.5	1.5	0.75	0.28	0.28
100	1.5	1	1	0.28	0.28	100	1.5	1.5	1	0.25	0.25
100	1.5	1	1.5	0.23	0.23	100	1.5	1.5	1.5	0.21	0.20
100	1.5	1	2	0.19	0.19	100	1.5	1.5	2	0.18	0.18
100	1.5	1	3	0.16	0.16	100	1.5	1.5	3	0.14	0.14
100	1.5	1	4	0.14	0.14	100	1.5	1.5	4	0.13	0.12
100	1.5	1	6	0.13	0.12	100	1.5	1.5	6	0.11	0.11
100	1.5	1	8	0.12	0.12	100	1.5	1.5	8	0.11	0.10
100	1.5	1	10	0.12	0.11	100	1.5	1.5	10	0.10	0.10

widely varying stiffnesses, or if there is one or more layers of saturated clay within the zone of influence, more involved techniques are needed. The increased stiffness of a replaced zone that is sufficiently wide to develop the full reduction in settlement [Eq. (6A.10)] can be readily incorporated into most methods for calculating immediate settlement (e.g., Bowles, 1987; Schmertmann, 1970 and Schmertmann et al., 1978; Burland and Burbidge, 1985; Meyerhof, 1956, 1965). If either the Bowles or Schmertmann method is used, a value of equivalent stress-strain modulus (E) for the replaced soil is needed and can be estimated in a variety of ways (see Sec. 2B.4). If a method based on SPT blowcounts (N) is used (Burland and Burbidge or Meyerhof), SPT tests can be conducted within the replaced zone to obtain N values for estimating S_i.

For a replacement zone of finite width within highly stratified matrix soils, Eq. (6A.9) and Table 6A.1 can be used to estimate S_i if an average or equivalent value for E_2 is calculated for the settlement influence zone. A weighted average modulus based on the thicknesses of the layers may be appropriate and can be calculated from the following equation (Bowles, 1987):

$$E_{2-av} = \frac{\sum_{i=1}^{i=n} H_i \cdot E_i}{\sum_{i=1}^{i=n} H_i} \tag{6A.12}$$

where n = number of layers within the settlement influence zone
H_i = thickness of layer i
E_i = stress-strain modulus for layer i

If certain strata are deemed to be more critical than others, additional weighting factors can be applied. For example, if a stratum is considered to be twice as critical as the other strata, its height can be multiplied by two, with the weighted height used in both the numerator and denominator of Eq. (6A.12).

When a saturated clay layer is within the depth of influence for settlement, S_i for a wide replaced zone can be calculated using the undrained modulus for the clay and any appropriate method based on elastic theory. An estimate of $\Delta\sigma_z$ throughout the height of the clay layer is needed to estimate primary consolidation settlement (S_c). Many engineers use Boussinesq-type analyses based on a uniform stress applied to the surface of a homogeneous half-space to estimate $\Delta\sigma_z$ in the clay layer; however, if the replaced zone is much stiffer than the undrained clay, or if the foundation bears below the ground surface, Boussinesq-type analyses are overly conservative and should not be used. Thus, it is better to use a method that considers the effects of the stiffer upper layer and the depth of embedment. A simple method for a wide replaced zone is to determine an equivalent uniform stress induced at the top of the lower zone (q_{LZ}) and dimensions of the loaded area (B_{LZ} and L_{LZ} or d_{LZ}) using the following equations:

$$B_{LZ} = B + \frac{2H}{\alpha} \qquad \text{for a rectangular foundation} \tag{6A.13a.1}$$

$$L_{LZ} = L + \frac{2H}{\alpha} \qquad \text{for a rectangular foundation} \tag{6A.13a.2}$$

$$d_{LZ} = d_0 + \frac{2H}{\alpha} \qquad \text{for a circular foundation} \tag{6A.13b}$$

$$q_{LZ} = q \cdot \frac{BL}{B_{LZ}L_{LZ}} \qquad \text{for a rectangular foundation} \tag{6A.14a}$$

$$q_{LZ} = q \cdot \frac{d_0^2}{d_{LZ}^2} \qquad \text{for a circular foundation} \tag{6A.14b}$$

Methods for estimating $\Delta\sigma_z$ for foundations embedded within homogeneous, semi-infinite soils [Nishida (1966) for circular; Skopek (1961) for rectangular] can be used with Eqs. (6A.13) and (6A.14) to estimate $\Delta\sigma_z$ beneath the replaced zone.

An alternate method for a wide replaced zone is to estimate $\Delta\sigma_z$ using the average bearing stress applied to surface of the replaced zone and solutions for layered elastic systems. All available methods for layered systems are for a circular load on the ground surface, so the effect of embedment is ignored, and square foundations must be converted to equivalent circles. If the replaced zone bears directly on a saturated clay layer and the clay layer is thick (extends to the bottom of the settlement influence zone or deeper), a two-layer analysis is appropriate, and Table 6A.2 can be used to estimate $\Delta\sigma_z$ beneath the center of the loaded area at various depths within the clay layer. If the replaced zone is underlain by a thin clay layer, or if the clay layer is the second layer beneath the replaced zone, a three-layer analysis is appropriate, and $\Delta\sigma_z$ can be estimated from the tables given in Jones (1962) or the charts provided in Peattie (1962).

If the width of the replaced zone is less than the minimum required for full reduction in settlement, the stresses induced within the underlying in situ soil may be either lesser or greater than for the same foundation in a homogeneous soil. The finite element results provided in Fig. 6A.9 illustrate this concept for a cylindrical replaced zone of varying diameter with $E_1/E_2 = 20$, $D/d_0 = 1$, and $H/d_0 = 1$. Along a vertical profile near the axis [Fig. 6A.9(*a*)], the induced vertical stress near the interface between the replaced zone and the underlying in situ soil decreases as the diameter of the replaced zone increases. For a replaced zone with $d_{rz}/d_0 \leq 1$, the induced vertical stresses near the interface are greater than for the homogeneous case ($d_{rz}/d_0 = 0$), and the induced stresses are lesser for $d_{rz}/d_0 \geq 1.5$ than for the homogeneous case. At greater depths, the trends are different. The induced vertical stresses do not change much for $d_{rz}/d_0 \geq 6$; this result is consistent with the approximate value of $d_{rz}/d_0 = 5.8$ calculated previously for minimum diameter required for full reduction in settlement when $H/d_0 = 1$ and $E_1/E_2 = 20$.

The induced vertical stresses along a horizontal profile located a short distance below the bottom of the replaced zone are shown in Fig. 6A.9(*b*). For any replaced zone with $d_{rz}/d_0 \leq 2$, $\Delta\sigma_z/q_0$ increases from the axis to the edge of the footprint for the replaced zone and then decreases with distance from the footprint. For $d_{rz}/d_0 \geq 3$, $\Delta\sigma_z/q_0$ decreases gradually with distance from the axis. The induced vertical stresses for a homogeneous soil fall within the range for replaced zones with $0.25 \leq d_{rz}/d_0$ 10 at all distances from the axis. To obtain an estimate of the relative magnitudes of settlement that would occur in the soil underlying the replaced zone, an estimate of the force transmitted to the underlying soil within the footprint of the loaded area (Q_{tr}) was obtained for each d_{rz}/d_0 by numerically integrating the curves for $\Delta\sigma_z/q_0$ in Fig. 6A.9 from $r/r_0 = 0$ to $r/r_0 = 1$. Values of Q_{tr} are shown as a percentage of the applied load in the table below, along with Q_{tr} for each d_{rz}/d_0 divided by Q_{tr} for a homogeneous soil:

$\frac{d_{rz}}{d_0}$	Q_{tr}(%)	$\frac{Q_{tr}}{Q_{tr(d_{rz}/d_0=0)}}$
0	10.9	1.00
0.25	14.5	1.34
0.50	18.3	1.68
0.75	21.1	1.94
1.00	18.4	1.69
1.50	10.3	0.95
2.00	7.5	0.69
3.00	5.8	0.53
4.00	5.3	0.49
6.00	5.1	0.47

TABLE 6A.2 $\Delta\sigma_z/q_0$ beneath the Center of a Uniform Circular Stress Applied to the Surface of a Two-Layer System with a Perfectly Rough Interface and $v_1 = v_2 = 0.5$

		$\Delta\sigma_z/q_0$ (%)			
H/d_0	z_i/H	$E_1/E_2 = 1$	$E_1/E_2 = 10$	$E_1/E_2 = 100$	$E_1/E_2 = 1000$
0.25	0	91.1	64.4	24.6	7.10
	1	64.6	48.0	20.5	6.06
	2	42.4	34.0	16.5	5.42
	3	28.4	24.4	13.3	4.80
	4	20.0	18.1	10.8	4.28
0.5	0	64.6	29.2	8.1	1.85
	1	28.4	16.8	6.0	1.62
	2	14.5	10.5	4.6	1.43
	3	8.70	7.0	3.6	1.24
	4	5.70	5.0	2.9	1.10
1	0	28.4	10.1	2.38	0.51
	1	8.70	4.70	1.58	0.42
	2	4.03	2.78	1.17	0.35
	3	2.30	1.84	0.91	0.31
	4	1.48	1.29	0.74	0.28

Note: z_i = depth below the interface between the two layers
Source: After L. Fox (1948).

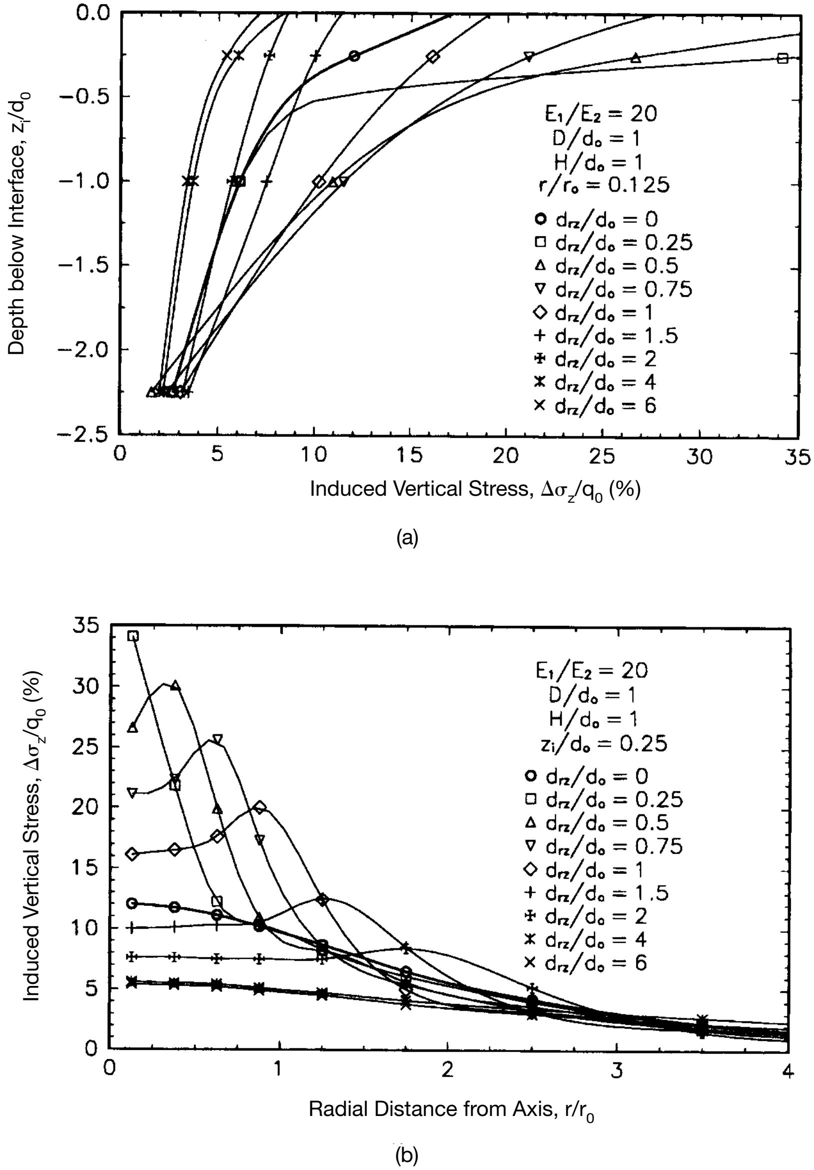


FIGURE 6A.9 Vertical stresses induced in underlying matrix soil by a uniform circular stress applied to the surface of an embedded replaced zone of varying diameter within an otherwise homogeneous half-space as predicted by finite element analysis: (*a*) vertical profile near axis; (*b*) horizontal profile near interface.

Thus, the settlement in the underlying soil would be greater for a replaced zone with d_{rz}/d_0 less than about 1.5 than for the homogeneous case, and is a maximum for $d_{rz}/d_0 \cong 0.75$. Where the induced vertical stresses in the underlying clay matrix are greater for overexcavation/replacement than without it, either a net increase or decrease in settlement could occur depending on whether the increase in primary consolidation settlement is greater or lesser than the decrease in immediate settlement. Thus, more involved analyses such as these are warranted when overexcavation/replacement is being considered for foundations underlain by one or more layers of saturated clay. For cases where a granular replaced zone is constructed within a saturated clay matrix, the replaced zone will act as a drainage well, and the rate of consolidation will be increased owing to increased horizontal drainage.

6A.2.3 Usefulness and Limitations of the Finite Element Method

On many projects where one or more of the ground modification techniques described in this handbook are used to increase bearing support for building foundations, the availability of reliable theoretical or empirical methods for estimating settlement or bearing capacity for the improved ground are either limited or nonexistent. In these cases, the finite element method–if used properly–can provide valuable insight into how ground modification affects settlement and bearing capacity. There are currently several axisymmetric and plane-strain finite element software packages available at relatively low cost that run on microcomputers and are capable of modeling the stress-strain-strength behavior of soils. Thus, the finite element technique is now readily available to many practicing engineers and should be used more frequently in foundation engineering practice than it is. Although it is not a cure-all, the finite element method is a powerful tool that all foundation engineers should consider using when available theoretical or empirical techniques are inadequate.

If the results of finite element analyses are to be meaningful, sufficient data are needed to determine reliable stress-strain-strength parameters for the bearing soils (replaced zone material and in situ soils for overexcavation/replacement). However, even under the best of circumstances, finite element analyses for soils seldom give reliable quantitative predictions for settlement or bearing capacity unless the input parameters for the stress-strain-strength characteristics of the soils are adjusted until the finite element results match the known or expected values. Therefore, an appropriate procedure for using the finite element method to estimate the bearing capacity or settlement of a foundation bearing on a replaced zone is as follows:

1. Calculate the estimated settlement or ultimate bearing capacity for the foundation with and without the replaced zone using the theories given in this section and Sec. 2B.4.
2. Obtain reliable values for the stress-strain-strength parameters for the replacement material and the in situ soils within the zone of influence. For the hyperbolic finite element model (Duncan and Chang, 1970), which is the most frequently used model for soils, the stress-strain-strength parameters are obtained from triaxial tests conducted on specimens of the soils or are estimated from published data on similar soils (e.g., Duncan et al., 1980).
3. Perform finite element (FE) analyses for the foundation without the replaced zone (in situ geologic profile). Compare the FE values for settlement or q_{ult} with those obtained from theoretical analyses (step 1). If the FE results are comparable to the theoretical results, proceed to step 4. If not, adjust the input soil parameters until the FE results are comparable.
4. Conduct finite element analyses for the foundation with the replaced zone. The input parameters for the replaced zone should be adjusted in a manner similar to that used for the in situ soils. These FEM results should provide reasonable estimates for settlement or ultimate bearing capacity.

In many projects, settlement must be estimated from limited soil data. In the United States it is fairly common to have only SPT blowcounts from which to estimate the settlement of granular soils, and the use of nonlinear stress-strain parameters for the finite element analyses is not justified in these cases. Therefore, it is better to estimate a reasonable range in values of elastic stress-strain modulus for the replacement material and each in situ soil and to perform a parametric study to estimate the probable percentage reduction in settlement which would occur from overexcavation/replacement. To provide an estimate of the settlement that would occur with the replaced zone, the range of values for percentage reduction in settlement can be applied to estimated values of settlement for the same foundation without the replaced zone.

6A.2.4 Limitations of the Overexcavation/Replacement Method

Although the overexcavation/replacement method has been used for centuries to improve bearing soils, it has the following limitations (Lawton et al., 1994):

1. Expensive bracing or sloping of the excavation is required if it is deep.
2. The excavation may cause settlement of adjacent existing structures.
3. High ground water may cause instability of the excavation and difficulties in compacting the replacement material. Therefore, bracing may be required to support the sides of the excavation, and pumping may be needed to lower the ground-water level within the excavation.
4. Sufficient high-quality replacement material may not be readily or economically available.
5. The densification that can be achieved in cohesionless replacement soils is sometimes limited to low levels, owing to lack of confinement in wide excavations and limited access in narrow excavations.

Example Problem 6A.1 A 1-m (3.28-ft) diameter footing will support a centric vertical load at the bearing level of 471 kN (106 kips) and will bear at a depth of 1 m (3.28 ft) below the ground surface. The in situ soil consists of a thick deposit of fairly homogeneous sandy micaceous silt. The overexcavation/replacement technique will be used to improve the bearing soils. A replacement zone 1 m (3.28 ft) in height and 2 m (6.56 ft) in diameter is being considered. The replacement material will be compacted gravelly sand. Relevant properties of the in situ and replacement soils are given below:

Replacement soil	In situ soil
Gravelly sand	Sandy micaceous silt
$\phi_1 = 45°$	$\phi_2 = 28°$
$c_1 = 0$	$c_2 = 0$
$E_1 = 100$ MPa (2090 ksf)	$E_2 = 5$ Mpa (104 ksf)
$v_1 = 0.2$	$v_1 = 0.4$
$\gamma_1 = 22$ kN/m^3 (140 pcf)	$\gamma_2 = 16.5$ kN/m^3 (105 pcf)

Estimate the ultimate bearing capacity, factor of safety against bearing capacity failure, immediate settlement, and vertical stresses induced in the underlying in situ soil. Compare results with those for the same footing without the replaced zone.

Solution: Ultimate bearing capacity without replaced zone. Use Meyerhof's (1951, 1963) equations and an equivalent square foundation:

$$B_{eq} = L_{eq} = (0.25\pi d_0^2)^{1/2} = [0.25\pi(1)^2]^{1/2} = 0.8862 \text{ m (2.91 ft)}$$

For $c_2 = 0$,

$$q_{\text{ult}} = \gamma_2 D N_{q2} S_{q2} D_{q2} + 0.5\gamma_2 B_{\text{eq}} N_{\gamma 2} S_{\gamma 2} D_{\gamma 2}$$

$$S_{q2} = S_{\gamma 2} = 1 + 0.1 \tan^2\left(45° + \frac{\phi_2}{2}\right) \cdot \frac{B_{\text{eq}}}{L_{\text{eq}}}$$

$$= 1 + 0.1 \tan^2(59°)(1) = 1.277$$

$$D_{q2} = D_{\gamma 2} = 1 + 0.1 \tan\left(45° + \frac{\phi_2}{2}\right) \cdot \frac{D}{B_{\text{eq}}}$$

$$= 1 + 0.1 \tan(59°)\frac{1}{0.8862} = 1.188$$

$$N_{q2} = e^{\pi \tan\phi_2} \cdot \tan^2\left(45° + \frac{\phi_2}{2}\right) = e^{\pi \tan(28°)} \cdot \tan^2(59°) = 14.72$$

$$N_{\gamma 2} = (N_{q2} - 1)\tan(1.4\phi_2) = (14.72 - 1)\tan(39.2°) = 11.19$$

$$q_{\text{ult}} = (16.5)(1)(14.72)(1.277)(1.188) + 0.5(16.5)(0.8862)(11.19)(1.277)(1.188) = 493 \text{ kPa } (10.3 \text{ ksf})$$

Calculate the average applied stress at bearing level:

$$q_0 = \frac{471}{0.25\pi(1)^2} = 600 \text{ kPa } (12.5 \text{ ksf})$$

$$F_s \text{ (bearing capacity failure)} = \frac{493}{600} = 0.82$$

Solution: Ultimate bearing capacity with replaced zone—punching through replaced zone q_b, is calculated for B_{eq}, L_{eq}, and an embedded depth of 2 m. N_{q2}, $N_{\gamma 2}$, S_{q2}, and $S_{\gamma 2}$ are the same as before.

$$D_{q2} = D_{\gamma 2} = 1 + 0.1 \tan(59°) \cdot \frac{2}{0.8862} = 1.376$$

From Eq. (6A.3*b*):

$$q_b = (16.5)(2)(14.72)(1.277)(1.376) + 0.5(16.5)(0.8862)(11.\ 19)(1.277)(1.376)$$
$$= 997 \text{ kPa } (20.8 \text{ ksf})$$

Calculate the perimeter length based on a circular rather than a square punching area, because the perimeter length for an equivalent square based on area is not the same as for the circle.

$$p = \pi d = \pi(1) = 3.142 \text{ m } (10.3 \text{ ft})$$

From Eq. (6A.4) with $c_1 = c_2 = 0$:

$$\frac{q_2}{q_1} = \left(\frac{\gamma_2}{\gamma_1}\right)\left(\frac{N_{\gamma 2}}{N_{\gamma 1}}\right)$$

$$N_{q1} = e^{\pi \tan(45°)} \cdot \tan^2(67.5°) = 134.9$$

$$N_{\gamma 1} = (134.9 - 1)\tan(63°) = 262.7$$

$$\frac{q_2}{q_1} = \left(\frac{16.5}{22}\right)\left(\frac{11.19}{262.7}\right) = 0.032$$

From Fig. 6A.5 for $q_2/q_1 = 0.032$ and $\phi_1 = 45°$

$$K_s \cong 3.2$$

From Eq. (6A.3*e*):

$$P_h = (3.2)[(16.5)(1)(l) + 0.5(22)(l)^2]$$
$$= 88.0 \text{ kN/m } (6.03 \text{ kips/ft})$$

From Eq. (6A.3*d*):

$$W_{pz} = 0.25\pi(l)^2(1)(22) = 17.28 \text{ kN } (3.88 \text{ kips})$$

$$A_f = \pi\frac{d_0^2}{4} = \pi\frac{(1)^2}{4} = 0.7854 \text{ m}^2 \ (8.454 \text{ ft}^2)$$

From Eq. (6A.3*a*):

$$q_{\text{ult}} = 997 + \frac{(3.142)(88.0)\tan(45°) + 0 - 17.28}{0.7854} = 1327 \text{ kPa } (27.7 \text{ ksf})$$

Solution: Ultimate bearing capacity with replaced zone—punching of replaced zone through in situ soil

$$W_{\text{eq}} = [0.25\pi(2)^2]^{1/2} = 1.772 \text{ m } (5.82 \text{ ft})$$

$$D_{q2} = D_{\gamma 2} = 1 + 0.1 \tan\left(45° + \frac{\phi_2}{2}\right) \cdot \frac{D + H}{W}$$

$$= 1 + 0.1 \tan(59°) \cdot \frac{1 + 1}{1.772} = 1.188$$

From Eq. (6A.6*b*):

$$q_b = (16.5)(2)(14.72)(l.277)(1.188) + 0.5(16.5)(1.772)(11.19)(1.277)(1.188) = 985.1 \text{ kPa } (20.6 \text{ ksf})$$

$$p = \pi d_{rz} = \pi(2) = 6.283 \text{ m (20.6 ft)}$$

From Fig. 6A.5 for $q_2/q_1 = 1$ and $\phi_1 = \phi_2 = 28°$:

$$K_s \cong 4.5$$

From Eq. (6A.6*d*):

$$P_h = 4.5(16.5)[(1)(1) + (1)^2/2] = 111.4 \text{ kN/m (7.63 kips/ft)}$$

From Eq. (6A.6*c*):

$$W_{rz} = 0.25\pi(2)^2(1)(22) = 69.12 \text{ kN (15.5 kips)}$$

$$A_{rz} = 0.25\pi(2)^2 = 3.142 \text{ m}^2 \text{ (33.8 ft}^2\text{)}$$

From Eq. (6A.6*a*):

$$q_{ult} = 985.1 \cdot \frac{3.142}{0.7854} + \frac{(6.283)(111.4)\tan(28°) + 0 - 69.12}{0.7854} = 4326 \text{ kPa (90.4 ksf)}$$

Solution: Ultimate bearing capacity with replaced zone—general shear failure within replaced zone. Because of the limited horizontal extent of the replaced zone, it is unlikely that general shear failure would occur within this zone. However, q_{rz} is calculated here to ensure that it is greater than either of the two values of q_{ult} computed for the punching cases. N_{q1} and $N_{\gamma 1}$ were calculated previously.

$$S_{q1} = S_{\gamma 1} = 1 + 0.1 \tan^2(67.5°)(1) = 1.583$$

$$D_{q1} = D_{\gamma 1} = 1 + 0.1 \tan(67.5°)(1/0.8862) = 1.272$$

From Eq. (6A.3*c*):

$$q_{rz} = (16.5)(1)(134.9)(1.583)(1.272) + 0.5(22)(0.8862)(262.7)(1.583)(1.272) = 9638 \text{ kPa (201 ksf)}$$

Comparing q_{rz} with q_{ult} for the two punching cases, it is seen that the value of q_{ult} for punching through the replaced zone controls. Now calculate the factor of safety against bearing capacity failure:

$$F_s = \frac{q_{\text{ult}}}{q_0} = \frac{1327}{600} = 2.2$$

Most engineers would prefer a higher factor of safety, preferably in the range of 3 to 4. According to Vesic (1975), a minimum factor of safety of 2.0 is acceptable for apartment and office buildings if a thorough and complete soil exploration has been completed, but higher factors of safety are required for other structures and conditions. Thus, for most situations $F_s = 2.2$ is unacceptable. The ultimate bearing capacity could be increased for this case by deepening the replacement zone. A second option would be to use a stronger replacement material, which in this case would likely necessitate chemically stabilizing the replacement material (see Sec. 6A.7) since the replacement material currently selected is a strong granular soil.

Solution: Settlement without replaced zone. From Table 6A.1 for $E_1/E_2 = 1$ and $D/d_0 = 1$, mean = I_s = 0.50. From Eq. (6A.9):

$$S_i = \frac{(600)(1)}{5000} \cdot 0.50 = 0.060 \text{ m} = 60 \text{ mm (2.4 in)}$$

Solution: Settlement with replaced zone. From Fig. 6A.9 for $E_1/E_2 = 20$, $\alpha \cong 0.42$. The minimum diameter required to obtain the full reduction in settlement can be estimated from Eq. (6A.10*b*):

$$d_{rz}(\text{min}) = 1 + \frac{(2)(1)}{0.42} = 5.8 \text{ m (19 ft)}$$

Since the actual d_{rz} is less than d_{rz}(min), the full reduction in settlement will not be achieved. From Table 6A.l for E_1/E_2 =20, $D/d_0 = 1$, $H/d_0 = 1$, and d_{rz}/d_0 =2.0, the mean $I_s \cong 0.23$. From Eq. (6A.9):

$$S_i = \frac{(600)(1)}{5000}(0.23) = 0.028 \text{ m} = 28 \text{ mm (1.1 in)}$$

The effect of the replaced zone is to reduce the estimated immediate settlement from 60 mm (2.4 in) to 28 mm (1.1 in). A maximum tolerable settlement of 25 mm (1.0 in) is frequently specified for buildings by the structural engineer or architect, so this value of immediate settlement may not be acceptable. Since there may be some additional long-term settlement in granular soils (see Burland and Burbidge 1985), a lower estimated value for S_i may be preferable, perhaps in the range of 13 to 19 mm (0.5 to 0.75 in). In this situation the settlement could be reduced by deepening or widening the replaced zone, using a stiffer replacement material, chemically stabilizing the replacement material before placement (see Sec. 6A.7), increasing the diameter of the footing, increasing the depth of embedment, or by some combination of these factors.

A design value of S_i = 19 mm (0.75 in) will be used. A number of changes could be made to reduce estimated S_i to the design value, including the following.

If footing size and depth of embedment remain the same:

$$d_0 = D = 1 \text{ m} = 3.3 \text{ ft} \Rightarrow \frac{D}{d_0} = 1$$

The required value of I_s can be calculated by rearranging Eq. (6A.9) as follows:

$$I_s \leq \frac{S_i E_2}{q_0 d_0} = \frac{(0.019)(5000)}{(600)(1)} = 0.16$$

For the same replacement material, $E_1/E_2 = 20$

$$H = 1.5 \text{ m (4.9 ft)}, d_{rz} = 4 \text{ m (13.1 ft)} \Rightarrow \frac{H}{d_0} = 1.5, \frac{d_{rz}}{d_0} = 4, \text{ and } I_s = 0.16.$$

If the replacement material is chemically stabilized to give E_1 = 250 MPa (5,220 ksf) $\Rightarrow E_1/E_2$ 50:

$$H = 1 \text{ m (3.3 ft)}, d_{rz} = 4 \text{ m (13.1 ft)} \Rightarrow \frac{H}{d_0} = 1, \frac{d_{rz}}{d_0} = 4, \text{ and } I_s = 0.16$$

$$H = 1.5 \text{ m (4.9 ft)}, d_{rz} = 3 \text{ m (9.8 ft)} \Rightarrow \frac{H}{d_0} = 1.5, \frac{d_{rz}}{d_0} = 3, \text{ and } I_s = 0.16$$

If the diameter of the footing is increased to 2 m (6.6 ft) with the same D = 1 m (3.3 ft) $\Rightarrow D/d_0 = 0.5$:

$$q_0 = \frac{471}{\frac{\pi}{4}(2)^2} = 150 \text{ kPa (3.1 ksf)}$$

$$I_s \le \frac{S_i E_2}{q_0 d_0} = \frac{(0.019)(5000)}{(150)(2)} = 0.32$$

For the same replacement material ($E_1/E_2 = 20$), the following dimensions of the replaced zone will work:

$$H = 1 \text{ m (3.3 ft)},\ d_{rz} = 3.6 \text{ m (11.8 ft)} \Rightarrow \frac{H}{d_0} = 0.5,\ \frac{d_{rz}}{d_0} = 1.8, \text{ and } I_s \cong 0.32$$

$$H = 2 \text{ m (6.6 ft)},\ d_{rz} = 2.4 \text{ m (7.9 ft)} \Rightarrow \frac{H}{d_0} = 1,\ \frac{d_{rz}}{d_0} = 1.2, \text{ and } I_s \cong 0.32$$

Solution: Stress distribution, in situ soil beneath replaced zone Although not needed to estimate settlement in this example problem, the stresses along the axis of the applied load are calculated for various depths below the interface to illustrate the procedure. The following four methods are used to calculate values of $\Delta\sigma_z$:

1. Values of $\Delta\sigma_z$ for an infinitely wide replaced zone are calculated from Table 6A.2 for $H/d_0 = 1$. Manual curve fitting is used to interpolate values of $\Delta\sigma_z/q_0$ for $E_1/E_2 = 20$ at each level of z_i/H.
2. Values of $\Delta\sigma_z$ for $d_{rz}/d_0 = 2$ are obtained by interpolation from a curve of $\Delta\sigma_z/q_0$ versus z_i/d_0 [similar to Fig. 6A.9(*a*)] drawn from results of finite element analyses.
3. Values of $\Delta\sigma_z$ for an embedded load within a homogeneous half-space are obtained from the equation given in Nishida (1966).
4. Values of $\Delta\sigma_z$ for a surface-loaded homogeneous half-space (ignoring the embedment) are obtained from the equation given in Foster and Ahlvin (1954).

The results are summarized in the following table:

		Induced vertical stress beneath axis, $\Delta\sigma_z$							
		Wide replaced zone* $E_1/E_2 = 20$		$d_{rz}/d_0 = 2$† $E_1/E_2 = 20$		Homogeneous considering D‡ $E_1/E_2 = 1$		Homogeneous ignoring D§ $E_1/E_2 = 1$	
z_i (m)	z_i/H	kPa	ksf	kPa	ksf	kPa	ksf	kPa	ksf
0	0	41	0.86	50	1.0	98	2.0	171	3.6
1	1	20	0.42	34	0.71	35	0.73	52	1.1
2	2	14	0.29	21	0.44	18	0.38	24	0.50
3	3	9.6	0.20	15	0.31	11	0.23	14	0.29
4	4	7.0	0.15	11	0.23	7.3	0.15	8.9	0.19

*Interpolated by manual curve fitting from Table 6A.2.
‡From results of finite element analyses.
‡Nishida (1966).
§Foster and Ahlvin (1954).

The induced vertical stresses for an infinitely wide replaced zone are less than for the case of $d_{rz}/d_0 = 2$ by an average of 33%. The induced stresses for the embedded homogeneous case are greater by 96% at the interface and lesser by 34% at $z_i/H = 4$. For the homogeneous case where embedment is ignored, the values of $\Delta\sigma_z$ are greater by 242% at the interface and less by 19% at $z_i/H = 4$.

6A.3 NEAR-SURFACE COMPACTION

The density of a soil is one of the primary factors that influences its compressibility and strength. So long as all other characteristics of the soil remain unchanged—such as state of stress, moisture condition, fabric, and physical and chemical nature of the soil particles and pore fluid—the densification of a soil will generally result in a stronger, less compressible material. However, it is generally neither desirable nor possible to densify a soil without changing one or more of these other characteristics. Therefore, the process of densifying a soil to achieve desired engineering characteristics is more complicated than it seems because many other factors will affect the engineering properties of the soil after densification. In addition, these other factors generally vary with time, which not only may affect the density of the soil (resulting in settlement or heave of the soil) but also may substantially alter the strength and compressibility of the soil.

Compaction is the process by which mechanical energy is applied to a soil to increase its density. At the time of compaction, the void spaces of the soil are occupied by air, water, and various chemicals and substances, or some combination thereof. For simplicity, it will be assumed in this discussion that the voids are occupied by air, water, or both. The soil is said to be *dry* if the voids are completely filled with air (water content, w, is 0 and degree of saturation, S_r, is 0) and *saturated* (also called *wet* by some engineers) if entirely occupied by water (S_r is 100%). In the more general case where both water and air occupy the voids ($0 < S_r < 100\%$), the moisture condition of the soil is described as *partially saturated* or *moist*. In reality soils are never dry, so the term *unsaturated* is sometimes used for soils that are not completely saturated ($S_r < 100\%$).

The total volume of the soil is decreased during densification; because the volume of the soil solids remains essentially unchanged, the volume of the voids decreases by the same amount that the total volume decreases. The result is that air, water, or both are expelled from the void spaces of the soil. The soil is usually partially saturated when compaction is initiated. Because the air permeability of a soil is typically much greater than the water permeability, air is expelled from the voids of a partially saturated soil during compaction. If the soil is initially saturated or becomes saturated or essentially saturated during the compaction process, water will be expelled.

Numerous methods and types of equipment are used to apply mechanical energy to soil during compaction. The methods and equipment used on any particular project depend on several factors, including cost, availability of equipment, the contractor's experience and preference, regional practices, volume and types of soil to be compacted, and conditions under which the compaction occurs (such as weather conditions, proximity to structures and utilities, and availability of water). In the following sections, the discussion of compaction is organized according to the equipment and method used to effect the densification of the soil.

The most common type of compaction, and what most engineers and contractors think of when the term compaction is used, is *near-surface* or *shallow compaction*. In open areas where the size of the equipment is not a consideration, near-surface compaction is generally accomplished using self-propelled or towed compaction rollers, although rubber-tired and tracked construction vehicles, sheep, elephants (Meehan, 1967), and human feet (Kyulule, 1983) have also been used. Hand-held compactors, tampers, or rammers are used in confined areas such as narrow trenches and small excavations, around pipes, behind retaining and basement walls, and adjacent to buildings, bridges, and other structures.

Near-surface compaction can be performed on unexcavated site soils or on fill materials placed in thin layers called *lifts* (Fig. 6A.10). Near-surface compaction of unexcavated site soils is usually done to increase the stiffness and strength of granular soils for support of lightly loaded (residential and light commercial) shallow foundations. Two common types of fills—embankments and fills produced from cut-and-fill operations—are illustrated in parts (b) and (c) of Fig. 6A.10.

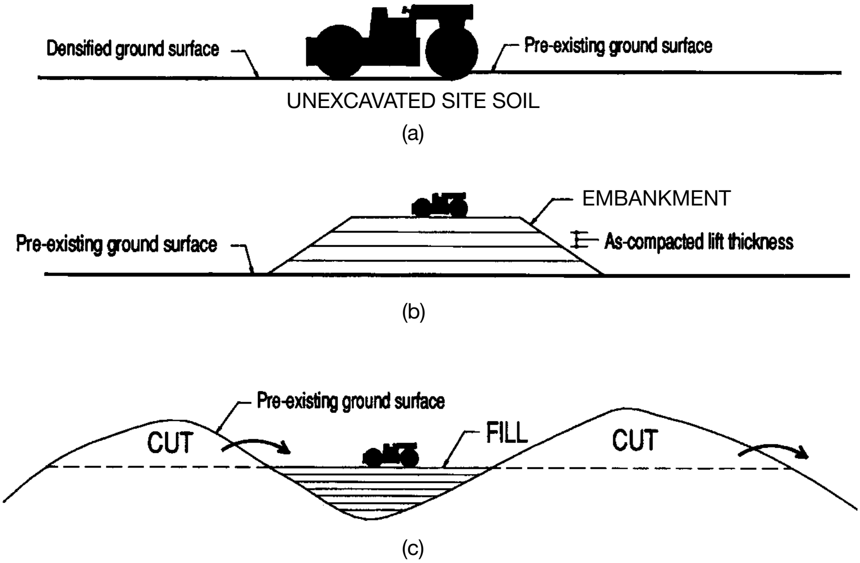


FIGURE 6A.10 Common types of near-surface compaction: (*a*) compaction of unexcavated site soil; (*b*) construction of embankment; and (*c*) cut-and-fill operations.

Embankments may be used to impound water (earth dams, levees, and so on) or to support structures such as highways, buildings, and abutments for bridges. Cut-and-fill operations are conducted in hilly areas to provide relatively flat grades primarily for highways and residential developments.

Fill materials can consist of on-site soils that have been excavated and replaced in the same location (called *overexcavation/replacement*—see Sec. 6A.2) or borrow materials obtained from either on-site or off-site borrow pits. Soils to be used in fills are frequently moisture conditioned, pulverized, chemically modified, blended with other materials, or otherwise modified before compaction. The maximum depth to which soil can be densified using rollers or hand-held equipment is about 6 ft (2 m) and usually is much less (hence the term *near-surface*), depending on the compaction equipment used and the type of soil compacted. Substantial densification generally occurs only to a depth of about 6 to 24 in (150 to 610 mm). The degree of densification of a dune sand with depth as a function of the number of passes of a vibratory roller is illustrated in Fig. 6A.11 (D'Appolonia et al., 1969). Because of the shallow depth to which substantial densification occurs, compaction of unexcavated site soils is generally limited to situations where either the applied loads are light or the depth of influence of the applied load is shallow (the width of the loaded area is small). As a rule of thumb, the depth of substantial densification decreases as the cohesiveness of the soil increases. Lift thicknesses in fill operations range from about 3 to 36 in (75 to 910 mm), with the smallest values for highly plastic (fat) clays and the largest values for select coarse-grained soils.

Engineering properties of the soil that can be affected by compaction and that can be controlled to some extent during the compaction process include the following:

1. Density
2. Strength
3. Compressibility

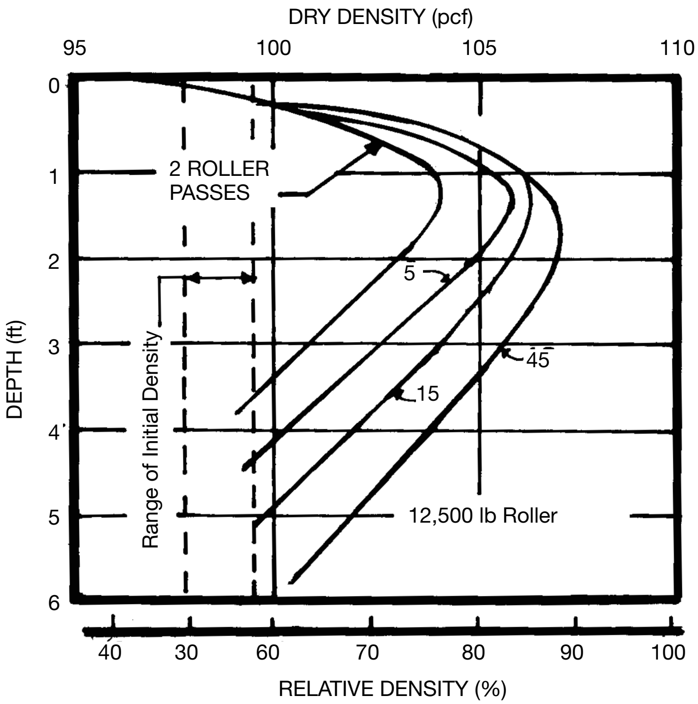


FIGURE 6A.11 Relationship between as-compacted dry density and depth for an increasing number of passes from a 12,500-lb (56-KN) vibratory smooth-drum roller on an 8-ft (2.4-m) lift height of a dune sand (from D'Appolonia et al., 1969).

4. Potential for volume changes caused by changes in moisture condition (swell, collapse, shrinkage)
5. Potential for frost heave
6. Permeability

6A.3.1 Equipment

A variety of construction equipment is used in a near-surface compaction process. For borrow materials, equipment is needed to excavate the material at the borrow location, transport it to the site, place it in lifts, and compact it. If in situ soils are to be compacted in place, only compaction equipment is required. A detailed discussion of the wide variety of methods and equipment used on compaction projects is beyond the scope of this book, so only a general overview is given here.

6A.3.1.1 Excavation, Hauling, Spreading

Borrow materials are usually excavated using some combination of power shovels, draglines, scrapers (pans), loaders, backhoes, and hydraulic excavators (see Figs. 6A.12 to 6A.15). Scrapers are multipurpose vehicles that can also be used to transport the soil and spread it in lifts on the fill area. If scrapers arc not used, borrow materials are usually transported to the fill area in haul trucks (Fig. 6A.13) and spread in lifts using graders (Fig. 6A.16), loaders (Fig. 6A.13), or bulldozers (Fig.

FIGURE 6A.12 Scraper (courtesy of Caterpillar Inc., Peoria, IL).

FIGURE 6A.13 Loader dumping soil into a haul truck (courtesy of Caterpillar Inc., Peoria, IL).

FIGURE 6A.14 Backhoe (courtesy of Deere & Company, Moline, IL).

6A.17). Some compaction rollers have a blade on the front that allows the soil to be spread and compacted in the same process. If the borrow material consists of dry, cohesive soils, it may need to be pulverized using a pulverizer or pugmill prior to spreading. Depending on the application, it may also be necessary to remove cobbles, boulders, undecomposed organic parts, or other undesirable components prior to hauling or spreading.

Moisture conditioning is frequently required to bring the soil to the desired compaction water content. This process consists of either drying or wetting the soil, reworking (mixing) the soil, and allowing the soil sufficient time to achieve relatively uniform moisture distribution. This can be accomplished either after the soil is spread in a loose lift using a water truck and discing equipment or in a stockpile located in a designated processing area. The conditioning time required to achieve moisture equilibrium in a soil after either wetting or drying depends on the soil type, varying from a few minutes for clean coarse sands and gravels to several days or weeks for highly plastic clays. When dry cohesive soils are wetted, the required moisture conditioning time depends primarily on the time required for the water to penetrate the hard clods and is therefore a function of clod size—larger clods need more conditioning time. At least 24 to 72 h are generally needed for proper moisture conditioning of cohesive soils, and more time usually is desirable.

6A.3.1.2 Compaction

Near-surface compaction is accomplished using a variety of equipment that can be classified into two primary categories depending on whether the general shape of the portion of the equipment that contacts the soil is round or flat. The major types of compactors are listed in Fig. 6A.18.

The types of equipment used on any compaction project depend on the grain-size distribution and mineralogy of the soils being compacted, the engineering properties that need to be controlled during the compaction process, and the equipment readily available to the compaction contractor. In some instances the type of equipment to be used is designated in the compaction specifications (usually just general type but sometimes more specifically, including type, weight, length of feet, vibration frequencies, and so on), but more commonly the choice of equipment is left to the compaction contractor. This will be discussed in more detail in Sec. 6A.3.5.

FIGURE 6A.15 Hydraulic excavator (courtesy of Deere & Company, Moline, IL).

Smooth-drum or smooth-wheel rollers (Fig. 6A.19) have smooth steel drums that provide 100% areal coverage over the width of the drum, and compaction occurs primarily from static pressures generated by the weight of the roller. A conceptual drawing illustrating the forces generated by a moving smooth-drum roller is given in Fig. 6A.20. The peripheral force (U) generated by the rotating drum acts horizontally but opposite in direction to the tractive force (Z) generated by the momentum of the roller. Because the peripheral and tractive forces tend to negate each other (although a small net horizontal force may result in either direction), the primary force generated upon the soil results from the weight of the roller (W).

The vertical contact pressure generated by a roller depends on W and the magnitude of the contact area. The shape of the contact area can be approximated by a rectangle whose length is equal to the length of the drum and whose width depends on the magnitude of vertical deflection, which in turn depends on the magnitude of W and the stiffness of the soil. During the initial pass of the roller on a loose lift, the vertical deflection (and hence the width of the contact area) is greatest; during subsequent passes of the roller, the soil is stiffer and the contact area is less, which produces higher contact stresses.

Smooth drum rollers have been used historically to compact soils of all types, but their usefulness in current practice is generally limited to proofrolling (smoothing) the surface of lifts compacted by other types of rollers and compacting granular soils and asphaltic pavements. Their usefulness

FIGURE 6A.16 Grader (courtesy of Caterpillar Inc., Peoria, IL).

FIGURE 6A.17 Tracked bulldozer towing a sheepsfoot roller (courtesy of Case Corporation, Racine, WI).

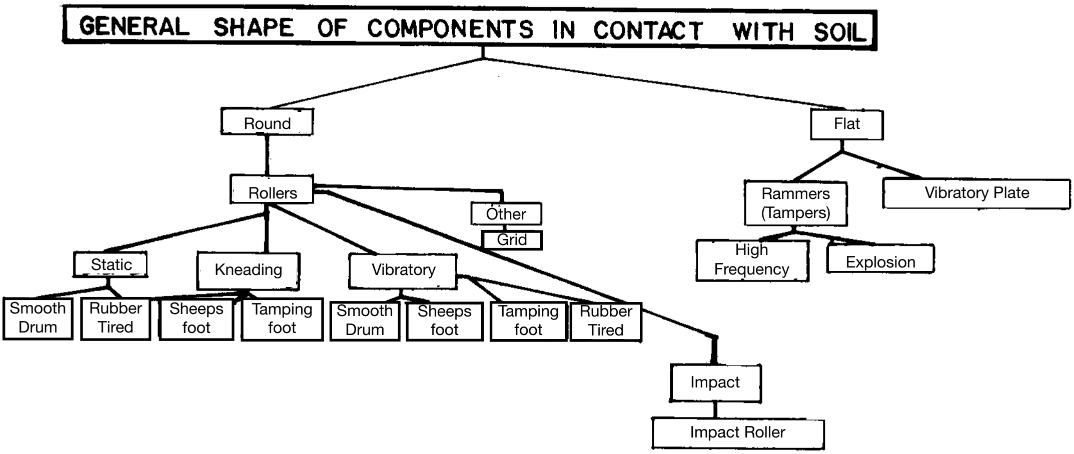


FIGURE 6A.18 Classification of near-surface compaction equipment (modified after Poesch and Ikes, 1975).

for compacting relatively soft cohesive soils is limited owing to poor traction and a "plowing" effect in which soil is displaced laterally without significant compaction (Hausmann 1990). Smooth drum rollers are also typically very slow compared to newer types of rollers.

A wide variety of *rubber-tired* or *pneumatic rollers* (Fig. 6A.21) are available. Most contain a series of tires closely spaced at regular intervals across the width of the roller. The areal coverage is typically about 80%, and tire pressures up to about 100 psi (700 kPa) are used (Holtz and Kovacs,

FIGURE 6A.19 Vibratory smooth-drum roller (courtesy of Vibromax 2000 USA Inc., Racine, WI).

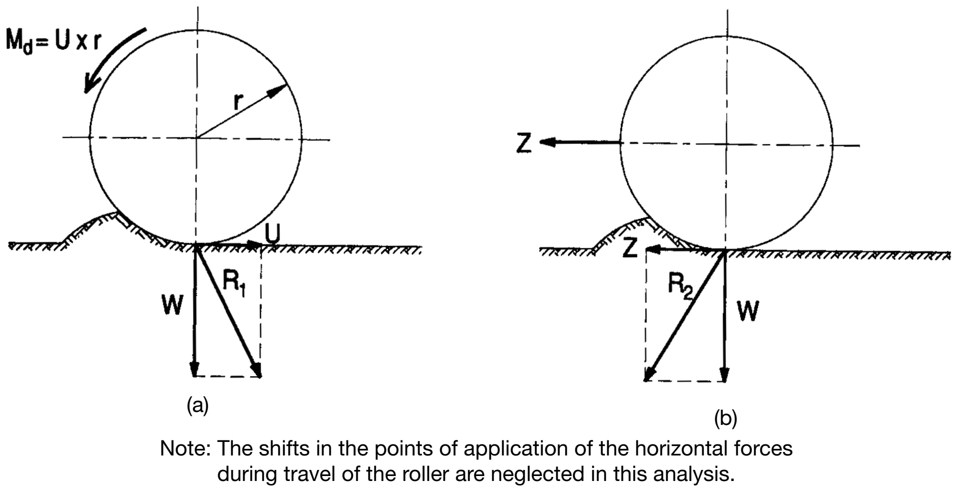


FIGURE 6A.20 Forces applied to the ground by a static smooth-drum roller (after Poesch and Ikes, 1975): (*a*) drum rotating in place; (*b*) drum pulled by a tractive force (*Z*).

FIGURE 6A.21 Rubber-tired roller (courtesy of Caterpillar Inc., Peoria, IL).

1981). Compaction occurs by a combination of static pressure (approximately equal to the inflation pressure of the tires) and kneading action of the tires. *Wobble-wheel rollers* have the wheels mounted such that the tires wobble laterally as the roller is towed forward, which imparts additional kneading action to the soil that facilitates compaction in cohesive soils (Spangler and Handy, 1982). The factors that influence the compactive effort include gross weight of the roller; wheel diameter and load; and tire width, size, and inflation pressure (Hausmann, 1990). Rubber-tired rollers are effective compactors of a wide range of soil types. Large-size tires are desirable in cohesionless soils to avoid shear and rutting. In general, low inflation pressures of 20 to 40 psi (140 to 275 kPa) are desirable for clean sands and gravelly sands with low gravel content, whereas inflation pressures greater than 65 psi (425 kPa) are desirable for cohesive soils and very gravelly soils (Spangler and Handy, 1982). Although not technically classified as compaction equipment, standard rubber-tired and tracked construction vehicles are sometimes used for compaction but are inefficient compared to rubber-tired or other rollers designed specifically for compaction.

In *sheepsfoot rollers* (Fig. 6A.17) and *tamping foot or padfoot rollers* (Fig. 6A.22), protrusions or "feet" of various lengths, sizes, and shapes (Fig. 6A.23) extend outward from a steel drum. Although tamping foot rollers are considered by some to be sheepsfoot rollers, they are categorized separately here because the effectiveness of each type can vary for different cohesive soils and compaction conditions. Sheepsfoot and tamping foot rollers are most effective in cohesive soils and are the best rollers currently available for compacting most cohesive soils. The results from model studies simulating the action of a sheepsfoot in clay and in sand are shown in Fig. 6A.24 (Spangler and Handy, 1982). These results were obtained by photographing the soil movements in a Plexiglas-fronted box caused by vertical penetration of a spherically tipped device and indicate a much larger zone of compaction in clay than in sand.

FIGURE 6A.22 Self-propelled tamping foot roller with leveling blade on front (courtesy of Caterpillar Inc., Peoria, IL).

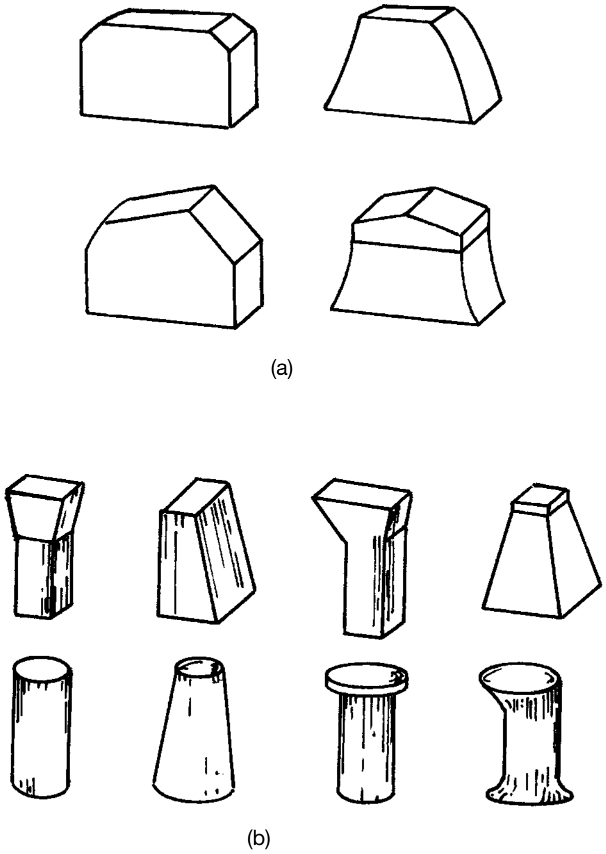


FIGURE 6A.23 Some typical shapes of feet for compaction rollers (from Poesch and Ikes, 1975): (*a*) tamping foot; and (*b*) sheepsfoot.

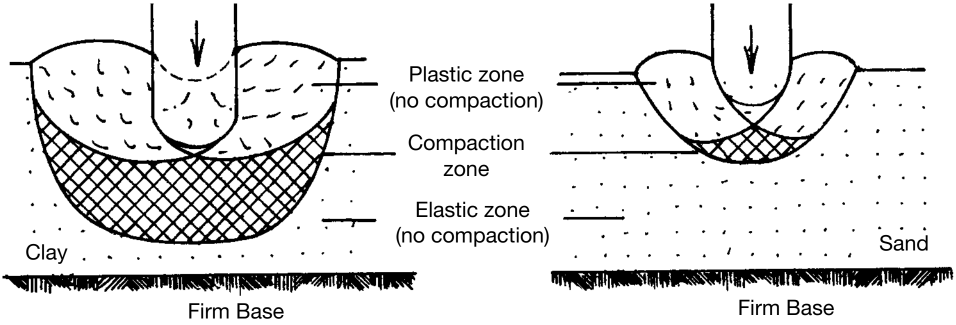


FIGURE 6A.24 Results of model studies simulating the action of a sheepsfoot roller in clay and sand (from Spangler and Handy, 1982; after Butt et al., 1968).

Sheepsfoot rollers generally have feet about 6 to 10 in (150 to 250 mm) long with tip areas from 5 to 12 in^2 (30 to 80 cm^2), provide an areal coverage of 8 to 12%, with contact pressures ranging from about 200 to 1000 psi (1400 to 7000 kPa) (Holtz and Kovacs, 1981). Compaction occurs as a combination of static pressure from the weight of the roller and kneading action of the feet as they penetrate the soil and are rotated through it. In cohesive soils, the high contact pressures permit crushing or breaking of dry clods and remolding of wet clods into a denser soil mass. If the lift thickness is smaller than the length of the feet, densification first occurs along the interface between the previously compacted lift and the new lift as the loose soil along the bottom of the new lift is pushed into and molded with the upper portion of the underlying lift. In this manner a good bond is developed between lifts, which helps prevent sloughing along lift interfaces in fills constructed on slopes and reduces preferential drainage paths for contaminant migration in compacted clay. It is therefore important to compact cohesive soils in thin lifts to promote good interfacial bonding. The upper surface of a newly completed lift is sometimes scarified using discing or other equipment before the next lift is placed to promote better bonding between the lifts. As compaction continues, the soil in the new lift is densified from the bottom upward, and the roller is said to "walk out" of the soil as the upper portion of the lift becomes densified.

The feet on tamping foot rollers are typically shorter, less than 6 in (150 mm), but have greater contact area [20 to 30 in^2 (130 to 190 cm^2)] than those on sheepsfoot rollers. The areal coverage is also much greater, typically about 40%, with contact pressures ranging from about 200 to 1200 psi (1400 to 8400 kPa) (Holtz and Kovacs, 1981). Some tamping foot rollers have hinged feet for greater kneading action. Like sheepsfoot rollers, tamping foot rollers are most effective in compacting cohesive soils. Tamping foot rollers are generally more effective than sheepsfoot rollers in compacting softer, wetter cohesive soils in which the clods need to be remolded rather than crushed or broken; the larger foot contact area and areal coverage of the tamping foot roller provides better and more uniform remolding of the clods. For the same lift thickness, however, the shorter tamping feet are less effective than sheepsfeet in producing interfacial bonding of lifts; therefore, thinner lift thicknesses or scarification of the lift surfaces is needed when using tamping foot rollers if good bonding between lifts is required.

The drums on *grid rollers* consist of heavy steel mesh that provides about 50% coverage and contact pressures from about 200 to 900 psi (1400 to 6200 kPa) (Holtz and Kovacs, 1981). The moderate areal coverage of the mesh allows high contact pressures while helping to prevent excessive shear deformation responsible for producing plastic waves in front of the roller (Hausmann, 1990). Grid rollers are effective in breaking and rearranging gravel and cobble-sized particles and are primarily used in weathered rocks and rocky soils containing large sand, gravel, and cobble fractions. Cohesive soils tend to clog the mesh, rendering it ineffective by essentially changing the roller into a smooth drum roller. By varying the operating speed of the grid roller, more effective compaction can be obtained; faster speeds can be used in the initial passes to break down the material, and lower speeds can be used in the final passes to produce greater densification (Hausmann, 1990).

Smooth-drum, rubber-tired, sheepsfoot, and tamping foot rollers can have vibrators attached to make them *vibratory rollers*. The vibration is typically produced by rotating eccentric weights. Vibratory rollers are the most effective method for near-surface compaction of cohesionless soils. The vibration produces cyclic deformation of the soil, which can result in significant densification beyond that provided by the static weight of the roller, as shown in Fig. 6A.25. Vibration can, in some instances, produce additional densification in soils with some cohesion (Selig and Yoo, 1977). The effectiveness of vibratory compaction on any given project is a function of the characteristics of the compactor, the properties of the soil, and the construction procedures used. The variables that influence vibratory compaction include the following (Forssblad, 1981; Holtz and Kovacs, 1981):

1. Characteristics of the compactor
   a. Static weight
   b. Vibration frequency and amplitude
   c. Ratio between frame mass and drum mass
   d. Drum diameter

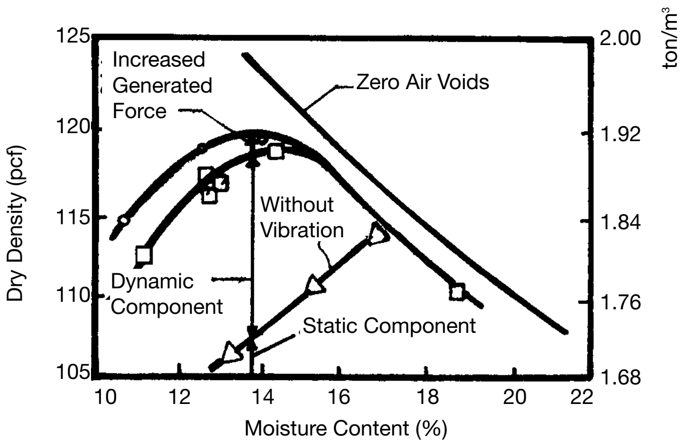


FIGURE 6A.25 Compaction results on 12-in (305-mm) layers of silty sand with and without vibration using a 17,000-lb (76-kN) towed vibratory roller (after Parsons et al. 1962 as cited in Selig and Yoo, 1977).

2. Properties of the soil
   a. Initial density (density prior to compaction)
   b. Grain-size distribution
   c. Shapes of particles
   d. Compaction (molding) water content
3. Construction procedures
   a. Number of passes of the roller per lift
   b. Lift thickness
   c. Speed at which the roller is operated

The relationship between lift thickness and number of passes for a given roller and desired soil density is illustrated in Fig. 6A.26. By decreasing the lift thickness, the number of passes needed to achieve the desired density is reduced; conversely, a thicker lift can be used if the number of passes is increased.

The vibration frequency of the compactor has been shown to affect the as-compacted dry density of a wide variety of soil types. As indicated in Fig. 6A.27, in most soils a peak dry density is achieved at an optimum vibration frequency. However, the differences in dry density achieved at various frequencies are relatively small. Better compaction (higher density and greater depth of influence) results from a combination of a large amplitude and a frequency just over the resonance frequency (usually about 25 Hz) than a combination of high frequency and small amplitude (Forssblad, 1981).

The effect of roller speed and number of passes on the as-compacted dry density of a well-graded sand and a highly plastic clay are shown in Fig. 6A.28. It is apparent from this figure that the dry density increases with increasing number of passes up to a certain point, beyond which little additional densification is achieved from more passes. The influence of roller speed is also obvious; for comparable conditions, reducing the travel speed results in higher as-compacted dry density.

The *impact roller* was developed in the 1950s in South Africa for in situ densification of collapsible sands (see Sec. 6A.3.4.2) (Clifford, 1980). An impact roller typically consists of a four-faced rolling mass that delivers four impact blows per revolution and is towed at an ideal speed of about 6 to 9 mi/h (10 to 14 km/h). The typical shape of an impact roller is shown in Fig. 6A.29 and

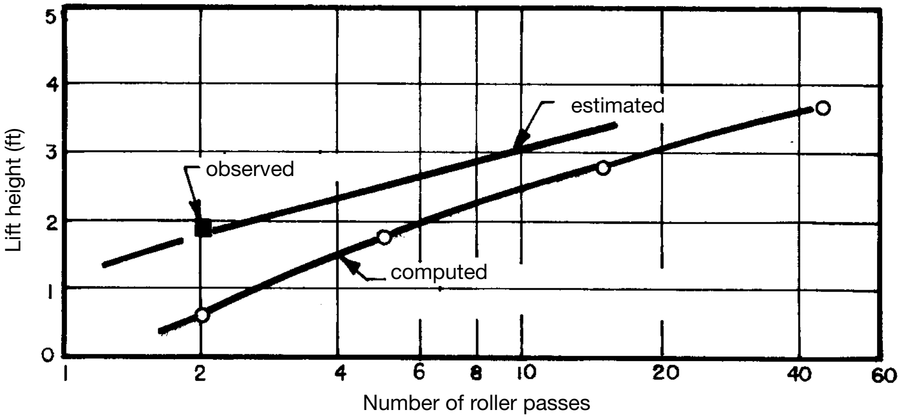


FIGURE 6A.26 Relationship between lift height and number of passes of a 12,500-lb (56-kN) vibratory smooth-drum roller required to achieve a minimum relative density of 75% in a dune sand (from D'Appolonia et al., 1969).

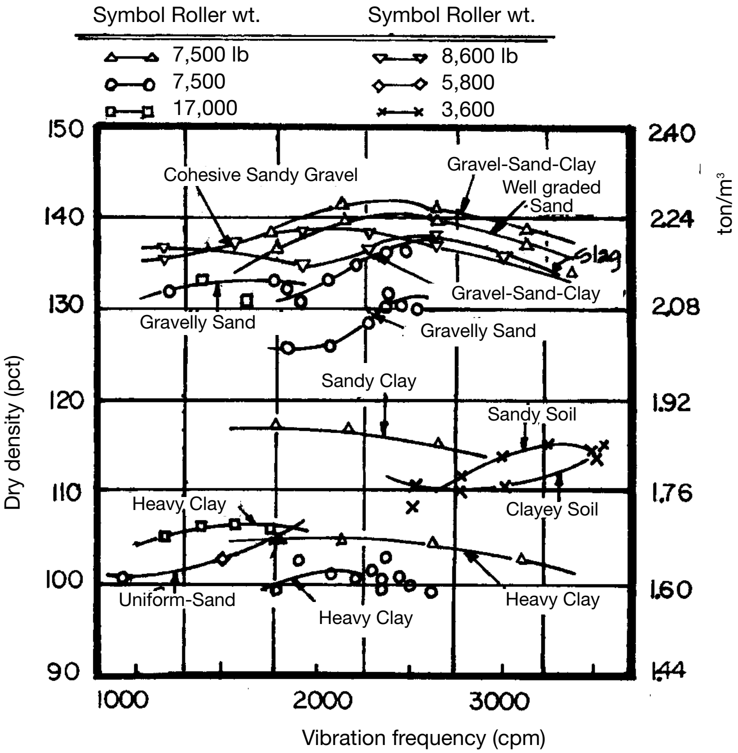


FIGURE 6A.27 Effect of vibration frequency on the as-compacted dry density of different soils (from Selig and Yoo, 1977).

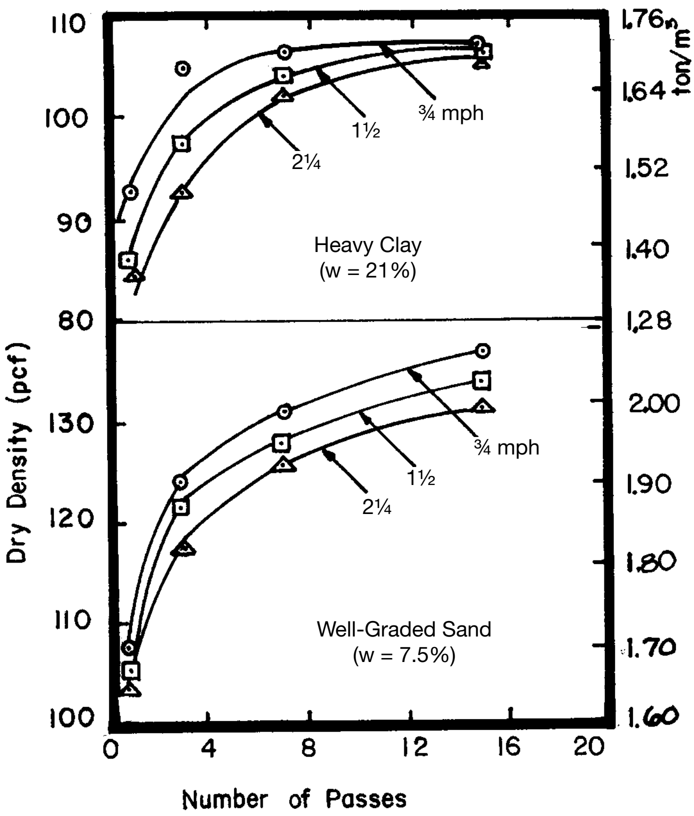


FIGURE 6A.28 Influence of travel speed and number of passes on the as-compacted dry densities of a clay and a sand (from Parsons et al., 1962 as cited in Selig and Yoo, 1977).

includes rounded corners to simplify transition from one lift and fall cycle to the next. A groove is incorporated behind each rolling corner to allow the roller mass to revolve freely during impact even in soft ground when the corner penetrates some distance into the surface of the soil. The impact faces of the roller are rounded so that as the soil densifies, the impact blow is dissipated over a smaller and smaller area, effecting compaction to greater depth. The high energy delivered by an impact roller compacts materials up to about 10 ft (3 m) thick. A corrugated surface is left by the impact roller, which has been found to produce good interfacial bonding of lifts and to reduce potential interfacial shear movements. If a smooth final surface is desired, the top lift must be proofrolled with a smooth-drum roller.

Most *vibratory plate compactors* (Fig. 6A.30) are self-propelled and hand-guided and are used primarily for base and subbase compaction for streets, sidewalks, and the like; street repairs; fills behind bridge abutments, retaining walls, basement walls, and so on; fills below floors; and trench fills (Broms and Forssblad, 1969). Tractor-mounted and crane-mounted models are also available. Compaction is achieved by the application of high-frequency (10 to 80 Hz), low-amplitude vibrations (Wacker, 1987). Vibratory plates are usually powered by gasoline or diesel engines and are used mainly for compacting granular and low-cohesion soils. The hand-guided vibratory plate com-

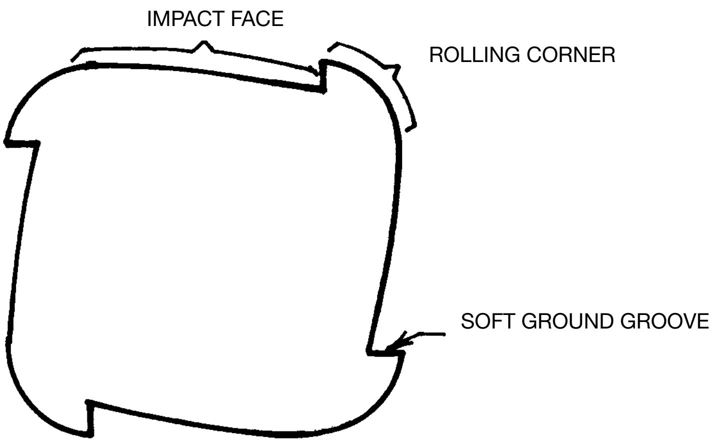


FIGURE 6A.29 Typical shape of an impact roller (from Clifford, 1980).

pactors generally weigh between 100 and 6000 lb (0.4 to 27 kN) and come in a wide variety of plate sizes.

Hand-guided *rammers* or *tampers* densify the soil by a combination of impact and vibratory compaction. The most common applications for rammers are for street repairs; fill behind bridge abutments, retaining walls, basement walls, and so on; and trench fills (Broms and Forssblad, 1969). In *explosion* or *combustion rammers* (Fig. 6A.31), a mixture of either diesel fuel or gasoline and air is ignited in an internal chamber that simultaneously forces the rammer against the ground and lifts

FIGURE 6A.30 Vibratory plate compactor (courtesy of Wacker Corporation, Menomonee Falls, WI).

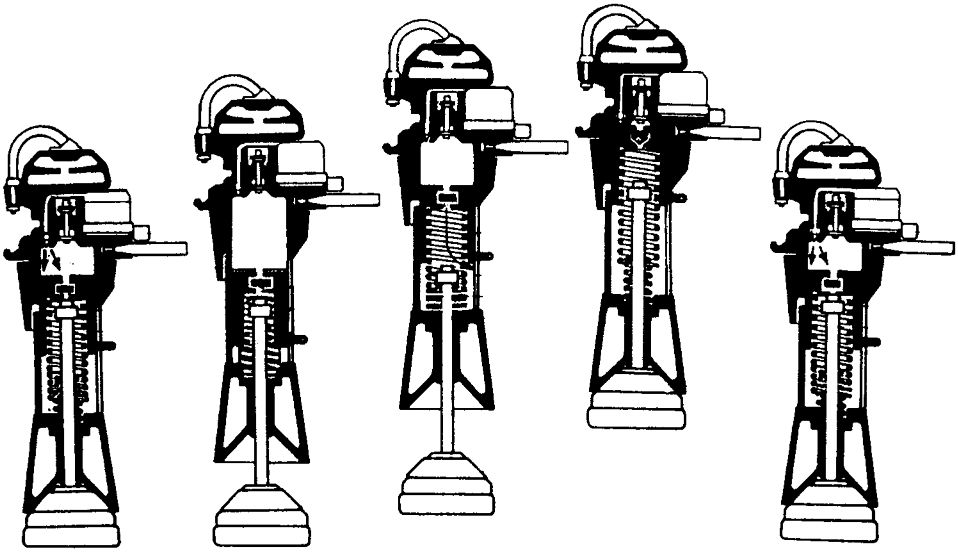


FIGURE 6A.31 Typical action of an explosion rammer (modified after Poesch and Ikes, 1975).

the casing off the ground. After the casing has moved upward a short distance independent of the rammer, the piston rod (which is attached to the rammer) is pulled upward with the casing until the maximum trajectory reached. The casing and rammer then fall freely to the ground, where a second impact occurs. Thus, two impacts are produced from one explosion. Explosion rammers typically weigh about 200 to 2000 lb (1 to 10 kN), with impact surface areas of about 80 to 930 in^2 (500 to 6000 cm^2) (Poesch and Ikes, 1975). *High-frequency rammers* (Fig. 6A.32) are usually powered by gasoline or diesel engines and produce high-frequency impacts through a piston/spring system connected to the engine via a gear transmission system (Wacker, 1987). Typical characteristics of high-frequency rammers are as follows (Broms and Forssblad, 1969; Wacker, 1987):

Frequency: 6 to 14 Hz

Weight: 100 to 300 lb (450 to 1300 N)

Impact area of ramming shoe: 100 to 500 in^2 (650 to 3200 cm^2)

Maximum height rammer comes off ground: 1 to 3 in. (25 to 75 mm)

High-frequency rammers can be used on granular and cohesive soils.

6A.3.2 Moisture-Density Relationships

The quality of a compacted soil is frequently correlated with the dry density of the as-compacted soil; that is, a greater dry density is commonly assumed to mean a better soil. Although this correlation is generally valid for many soils in terms of their strength and compressibility, other parameters (especially water content) may have an important effect, especially in cohesive soils. In addition, some engineering properties of certain soils are only slightly influenced by dry density, and other parameters such as moisture condition have a greater effect. These relationships will be discussed in more detail in Sec. 6A.3.4. Despite these other considerations, the fact remains that dry density is

FIGURE 6A.32 High-frequency rammer.

the property used most often to determine whether a compacted soil will exhibit the desired engineering behavior.

The primary factors that influence the value of dry density obtained by compacting a specific soil are as follows:

1. Moisture condition of the soil during compaction (molding or compaction water content)
2. Compactive energy or effort per unit volume of soil (lift thickness and number of passes, blows, or tamps of the equipment)
3. Method of compaction (compaction equipment)
4. Rate of compaction (rate of travel, frequency of impact, or frequency of vibration of the equipment)

For a given situation in which factors 2, 3, and 4 are held constant, the as-compacted dry density of the soil may vary significantly depending on the molding water content. For many soils, the moisture–density relationship for a given method of compaction and compactive effort can be described by a single peak curve similar to that shown in Fig. 6A.33. The water content and dry density corresponding to the highest peak in the moisture–density curve for one method of compaction and one level of compactive effort are referred to as the *optimum* water content and *maximum* dry density. If the method of compaction or the compactive effort is varied, the moisture–density curve for the same soil will be different. For example, the effect of varying the compactive effort for the same soil and the same method of compaction is illustrated in Fig. 6A.34; similarly, the influence of method of compaction is shown in Fig. 6A.35. Therefore, for a specific soil there are an infinite number of optimum water contents and maximum dry densities, and the terms optimum and maximum are relative rather than absolute and should be used in proper contexts.

For a given method of compaction, the optimum moisture condition for achieving the maximum dry density is represented by the locus of points corresponding to the peak points for various compactive efforts (Fig. 6A.34). This locus of points is known as the *line of optimums* or the *locus of optimums*. Analysis of the general trend of the line of optimums shows that the maximum dry density increases and the optimum water content decreases as the compactive effort is increased. This trend is not without bounds, however, as a point is reached where applying additional compactive effort produces no measurable increase in dry density and in some instances may produce a decrease in dry density.

The line of optimums is generally approximately parallel to the zero air voids line (ZAVL), which represents a degree of saturation of 100%, and as such the line of optimums is approximately a line of constant degree of saturation. In the author's experience, the degree of saturation represented by the line of optimums varies from about 60 to 95%, with the lower values for clean sands and higher values for highly plastic clays; for most soils this value ranges between 70 and 85%. This correlation is important with respect to the engineering behavior of many soils (to be discussed in Sec. 6A.3.4) and suggests that the moisture condition of a soil may, in some circumstances, be better represented by degree of saturation rather than by water content. The approximate degree of saturation represented by the line of optimums is referred to as optimum saturation (S_{opt}). The line of optimums may vary for a given soil depending on the method of compaction, as shown in Figure 6A.36 for a lean clay. The range in degree of saturation for each of the four lines of optimums shown in Fig. 6A.36 are summarized as follows (assuming $G_s = 2.70$):

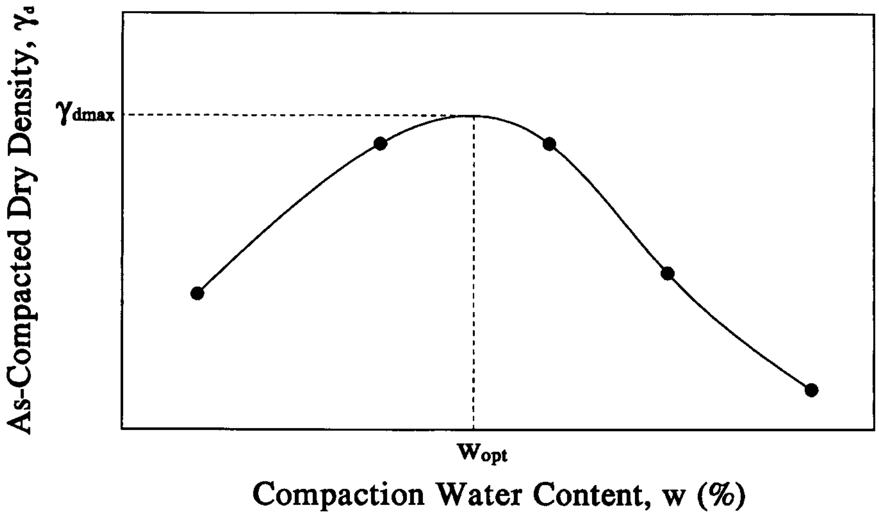


FIGURE 6A.33 Typical moisture–density relationship for compacted soil.

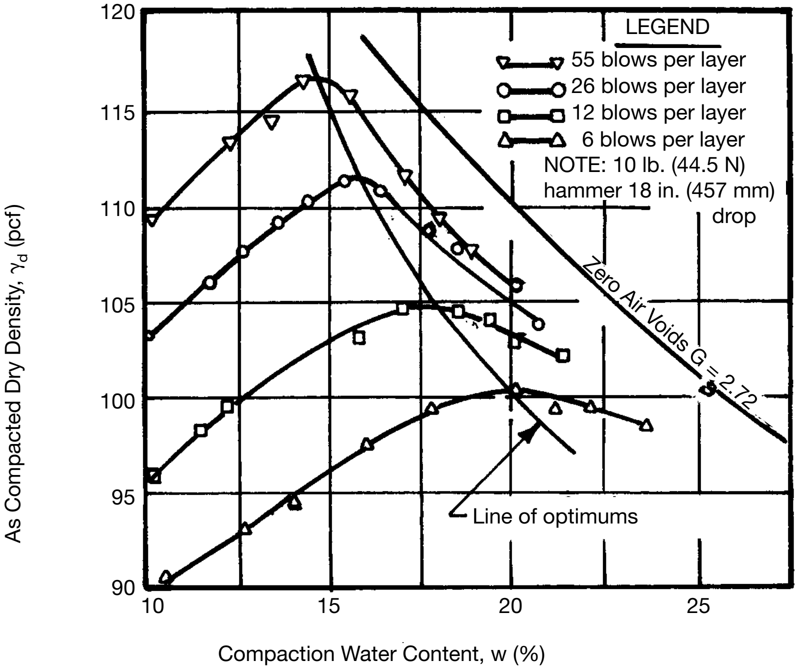


FIGURE 6A.34 Effect of compactive effort on the moisture–density relationship of an impact-compacted soil (from Foster, 1962).

Method of Compaction	Approximate Range of S_{opt}
Sheepsfoot roller	73–83%
Laboratory impact	77–86%
Laboratory static	80–81%
Rubber-tired roller	89–90%

For each method of compaction, S_{opt} varies within a fairly narrow range (≤10%). Although the lines of optimums vary somewhat from each other, the line of optimums from the laboratory impact (Proctor) tests provides a reasonable approximation of the line of optimums for the two field-compacted soils, and this is generally true for most soils.

The ZAVL varies as a function of specific gravity of the soil solids, and the ZAVL can be found from the following equation by setting $S_r = 100\%$:

$$w = \left(\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s} \right) \cdot S_r \tag{6A.15}$$

where γ_d = dry density (by mass or weight) of the soil
γ_w = density (by mass or weight) of water
w = water content

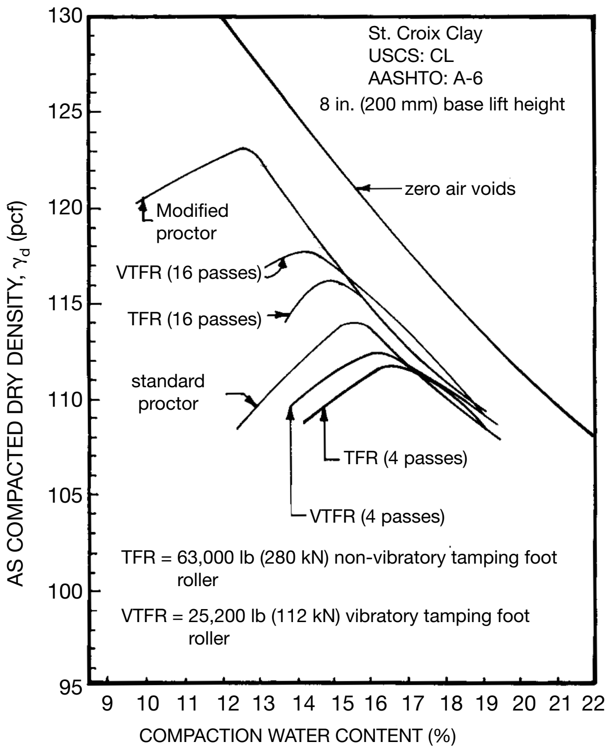


FIGURE 6A.35 Comparison of laboratory and field moisture–density relationships for St. Croix clay (modified after Lin and Lovell, 1981; data from Terdich, 1981).

S_r = degree of saturation
G_s = average specific gravity of the soil solids

Any line of constant saturation (e.g., 60%, 70%, 80%) can be found using Eq. (6A.15).

Although the single peak curve is most common, an extensive laboratory study by Lee and Suedkamp (1972) showed that three irregularly shaped curves can also occur, as illustrated in Fig. 6A.37. The results from their research indicate that (1) soils with a liquid limit between 30 and 70 typically exhibited a single-peak moisture–density curve; (2) both double-peak and oddly shaped curves were present in soils with a liquid limit greater than 70; (3) soils with a liquid limit less than 30 usually produced either a double-peak or a one-and-one-half peak curve; and (4) the length of the moisture conditioning period substantially affected the moisture–density relationships in high-plasticity (high-liquid-limit) soils but had little influence on low-plasticity or nonplastic soils.

The moisture–density curves for free-draining sands and sandy gravels typically have 1½ peaks, as illustrated in Figure 6A.38 (Foster, 1962). The maximum dry density for these soils is obtained in either the air-dried or saturated condition. In the partially saturated state, capillary suction increases the effective stresses in the soil and produces a resistance to compaction called *bulking.* Because it is difficult to keep large quantities of borrow materials in the air-dried condition, sands and sandy gravels are usually placed saturated. Keeping free-draining soils saturated is not easy and generally requires continuous wetting of the soil in front of the compaction equipment.

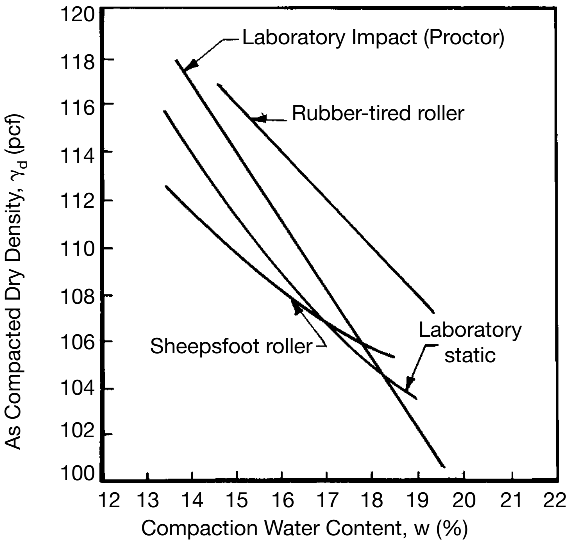


FIGURE 6A.36 Comparison of lines of optimums for four methods of compaction (Foster, 1962).

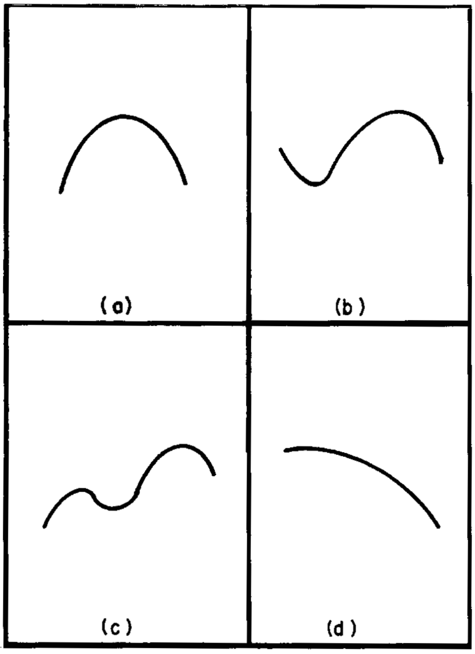


FIGURE 6A.37 Types of moisture–density curves: (*a*) single peak; (*b*) one-and-one-half peaks; (*c*) double peak; and (*d*) oddly shaped (Lee and Suedkamp, 1972).

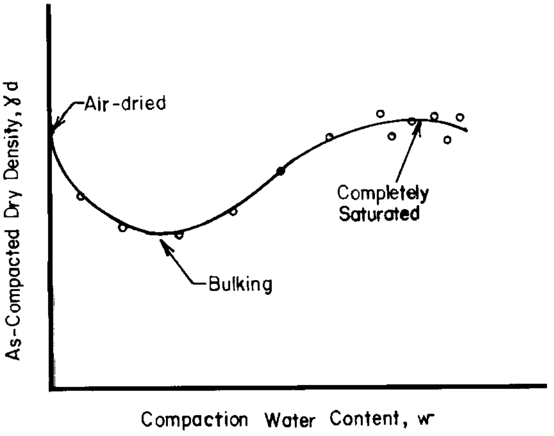


FIGURE 6A.38 Typical compaction curve for free-draining sands and sandy gravel (Foster, 1962).

The temperature of the soil during compaction has a noticeable effect on as-compacted dry density when the temperatures are above freezing—the same compactive effort produces a lower dry density at the same compaction water content (Waidelich, 1990). Thus, more compactive effort is needed to obtain the same value of dry density at lower temperatures. If the temperature of the soil is below freezing, it is extremely difficult, uneconomical, and virtually impossible under some circumstances to obtain the densities required for proper performance of a compacted fill. The influence of temperatures below freezing on the standard and modified Proctor moisture–density relationships of a sand with a trace of silt is illustrated in Fig. 6A.39. At a temperature of 30°F (–1°C) standard Proctor $\gamma_{d\max}$ was 94% of the value at 74°F (23°C), and increasing the compactive effort to modified Proctor (4.5 times that of standard Proctor) at 30°F (–1°C) only increased $\gamma_{d\max}$ to 98% of standard Proctor $\gamma_{d\max}$ at 74°F (23°C). An additional problem with fills compacted at temperatures below freezing is that the interior portions of the fill may take several years to thaw, depending on the type of fill and its dimensions. For these reasons, it is highly recommended that compaction not be conducted during freezing weather; in fact, many government agencies located in freezing climates that engage in earthwork construction do not permit the construction of fills during winter months.

6A.3.3 Laboratory Compaction and Testing

The best method for predicting the engineering behavior of compacted soil is to conduct full-scale field testing on the soil that simulates as closely as possible the actual service conditions. For exam-

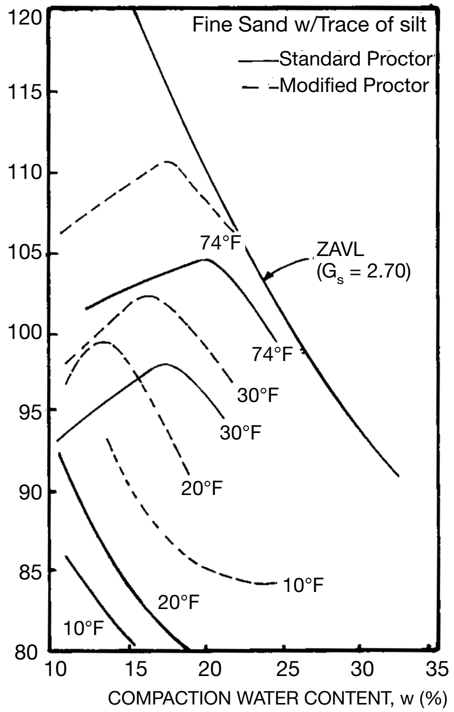


FIGURE 6A.39 Influence of temperature on the moisture–density relationships of a sand with a trace of silt (from Waidelich, 1990).

ple, if the compacted soil is to support shallow foundations for a building, full-scale load tests can be conducted on all footings prior to any construction of the superstructure to ensure that the compacted soil can support the footings without excessive settlement or failure. This method is rarely used in practice for obvious reasons—the cost is extremely high, and a long time is required for testing. For a compacted fill, a test pad is sometimes constructed using the actual soil and compaction equipment, and appropriate tests (field, laboratory, or both) are conducted on the test pad to determine the important engineering properties. For example, many state and federal regulatory agencies currently require that a test pad be constructed to simulate a compacted clay liner for waste containment systems. In situ permeability tests (usually double-ring infiltration tests, ASTM D5093) are then conducted on the test pad to verify that the permeability of the test pad is less than the maximum allowable value (usually 1.0×10^{-7} cm/s = 2.8×10^{-4} ft/day). However, laboratory tests are used on many projects as the primary method for predicting, controlling, and verifying the engineering properties and behavior of compacted soil because laboratory tests generally cost much less than comparable field tests.

It is important to model field conditions as closely as possible when conducting moisture–density tests and when preparing samples to be used in other types of laboratory tests. For example, recent research on compacted soils has shown that clod size and percentage of large-size particles (gravel and larger) have an important effect on the engineering properties of compacted soils (e.g., Benson and Daniel, 1990; Day, 1989, 1991; Houston and Randeni, 1992; Houston and Walsh, 1993; Larson et al., 1993; Shelley and Daniel, 1993). A general rule of thumb for laboratory testing is that the maximum particle or clod size should be less than one-sixth to one-tenth the smallest dimension of the sample. It is therefore important to test samples that are large enough to obtain results that are representative of field behavior. Several methods are available for correcting for oversize particles, where the term oversize refers to particles that are too large to be included in a particular testing apparatus (see ASTM D698, D1557, D4253, and D4718; Hausmann, 1990; Houston and Walsh, 1993; Fragaszy et al., 1992; Siddiqi et al., 1987).

When preparing soils for moisture–density determination or when making specimens for laboratory testing, the soil should be brought to the desired water content and double-sealed in airtight containers for a sufficient time to allow the soil to come to moisture equilibrium (called the *moisture conditioning period*). For proper modeling of field compaction techniques, this moisture conditioning period should duplicate that to be used in the field. As noted previously, for cohesive soils a minimum of 24 h is desirable, and for highly plastic soils 72 h or more may be needed for proper moisture conditioning. When preparing samples for testing at specific values of water content and density, a method of compaction should be used that simulates as closely as possible the field compaction method. A comparison of the most common laboratory and field compaction methods is provided in Table 6A.3, and additional details on laboratory compaction methods are given below.

6A.3.3.1 Static Compaction

For all laboratory compaction methods, the soil is placed loosely into a compaction mold (usually cylindrical with a base plate and generally made of steel or aluminum), and the surface of the soil is leveled as well as possible using a spatula or similar device. In static compaction, a loading plate slightly smaller than the inner diameter of the mold (usually with an attached piston) is placed gently onto the surface of the loose soil and a compressive force is applied to the soil via the loading plate in one of several ways (see Fig. 6A.40). The most common methods are (1) slowly pushing the loading plate using a speed-regulated compression machine; (2) jacking against a reaction frame using a hydraulic jack; or (3) applying constant force using a dead-weight reaction frame. The compressive stress is increased until the desired density has been achieved or a predetermined energy has been applied. A diagram of a mold used by the writer to prepare statically compacted oedometer specimens is shown in Fig. 6A.41. Compactive energy can be determined by numerically integrating the load-deformation curve (Fig. 6A.42). It is difficult to apply a predetermined magnitude of compactive effort, especially if a speed-regulated compression machine is used, as the energy must be calculated immediately after each set of load and deformation readings and the test stopped when the desired energy level has been reached.

TABLE 6A.3 Comparison of Common Laboratory and Field Compaction Methods

Type of Compaction	Laboratory methods	Field methods
Static	Loading plate with outer diameter slightly smaller than inner diameter of mold is pushed onto soil using a speed-regulated compression machine, a hydraulic jack within a loading frame, or a dead weight loading system (Fig. 6A.40).	Smooth drum roller (nonvibratory) Rubber-tired roller*
Impact	Impact hammer is used to drop a rammer with a diameter about one-third to one-half the diameter of the mold from a given height onto the soil (Fig. 6A.43). In some setups, the diameter of the rammer is slightly smaller than the diameter of the mold.	Impact roller Hand-guided rammer† Dynamic compaction
Kneading	Harvard miniature tamper is pushed into the soil until the spring deflects (Fig. 6A.44). California kneading compactor is used to push a compaction foot into the soil at constant pressure for a given time (Fig. 6A.45).	Sheepsfoot roller Tamping foot roller Rubber-tired roller*
Vibratory	Mold containing soil plus surcharge weight on top of the soil is placed on vibrating table and vibrated (Fig. 6A.47). High-frequency rammer compacts the soil within a mold (Fig. 6A.48). The ramming plate has a diameter slightly smaller than the diameter of the mold	Vibratory rollers‡ Vibrating plates Hand-guided rammers†

*Combination static and kneading compaction.
†Combination impact and vibratory compaction.
‡Some vibratory rollers, such as vibratory sheepsfoot and vibratory rubber-tired rollers, compact by using a combination of vibration and kneading action, and require more sophisticated tools to model.

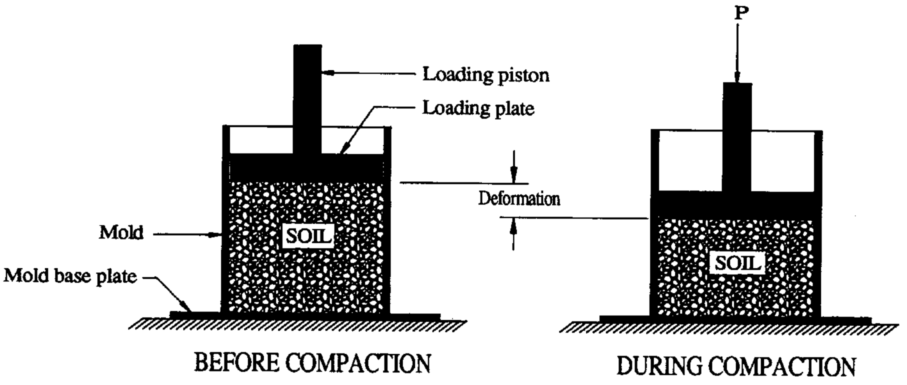


FIGURE 6A.40 Schematic illustration of laboratory static compaction.

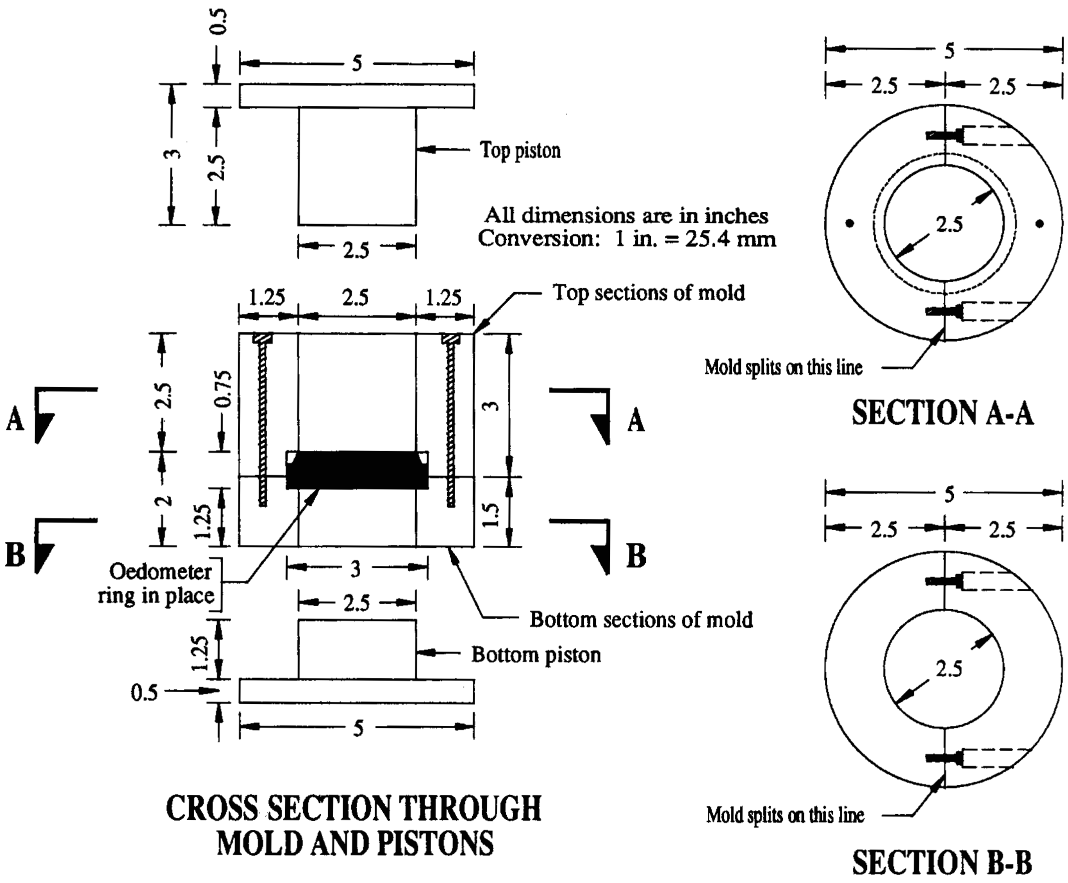


FIGURE 6A.41 Schematic diagram of mold for laboratory static compaction of oedometer specimens.

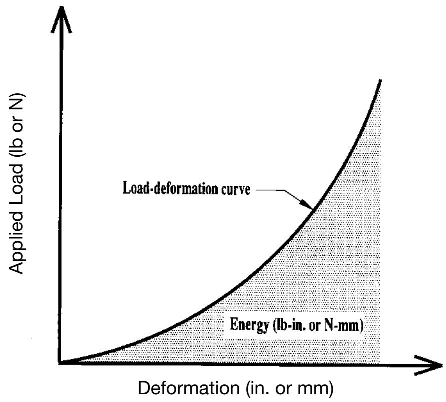


FIGURE 6A.42 Typical load-deformation curve from laboratory static compaction illustrating energy of compaction.

Static compaction is a relatively inefficient laboratory compaction method, because for any given type of soil and compaction water content, the same density can be achieved with less effort using another compaction method. Most samples need to be compacted in thin lifts because the force required to obtain a given density can be enormous. For example, the author once attempted to use static compaction to prepare a 6-in (152-mm) diameter sample of clayey sand in 2-in (50.8-mm) thick lifts to modified Proctor maximum dry density at the modified Proctor optimum water content. Using a 250,000-lb (1.1-MN) capacity compression machine normally used for structural engineering testing, the capacity of the machine was reached before the desired density was achieved. However, static compaction is commonly used in parametric studies because the reproducibility of nominally identical fine-grained specimens is greater than for the other methods. Because the method of compaction can have a significant influence on the as-compacted fabric of soils—and hence their engineering properties (see Sec. 6A.3.4)—static compaction should be used with caution.

6A.3.3.2 Impact Compaction

Impact compaction, usually in the form of Proctor tests, is the most commonly used method for estimating the moisture–density relationships of compacted soil. Of all the types of compaction rollers described in Sec. 6A.3.1, only the impact roller can be classified as an impact method. Therefore, the use of impact tests to predict field behavior of most compaction rollers may create some problems because of differences in moisture–density relationships between the laboratory and field compaction methods. A comparison of the differences in moisture density curves for two compaction rollers and laboratory impact compaction was given previously (Fig. 6A.35). Furthermore, differences in compaction method can have an important effect on the fabric of fine-grained soils and therefore the engineering behavior of compacted soils. Impact methods are used so frequently primarily because of their relatively low cost and the ease and accuracy in calculating compactive effort. However, the potential differences in moisture–density relationships and other engineering characteristics must be considered when deciding which compaction method to use for laboratory and field testing of compacted soils.

The standard test for determining the moisture–density relationships of compacted soils are impact tests in which rammers of a given weight are dropped from a constant height onto soil contained within a mold (Fig. 6A.43). Either hand-held or automatic hammers may be used. These tests are routinely conducted in the laboratory and the field and are commonly referred to as the *Proctor tests* in honor of R. R. Proctor, who in 1933 introduced the principles of modern compaction in a series of

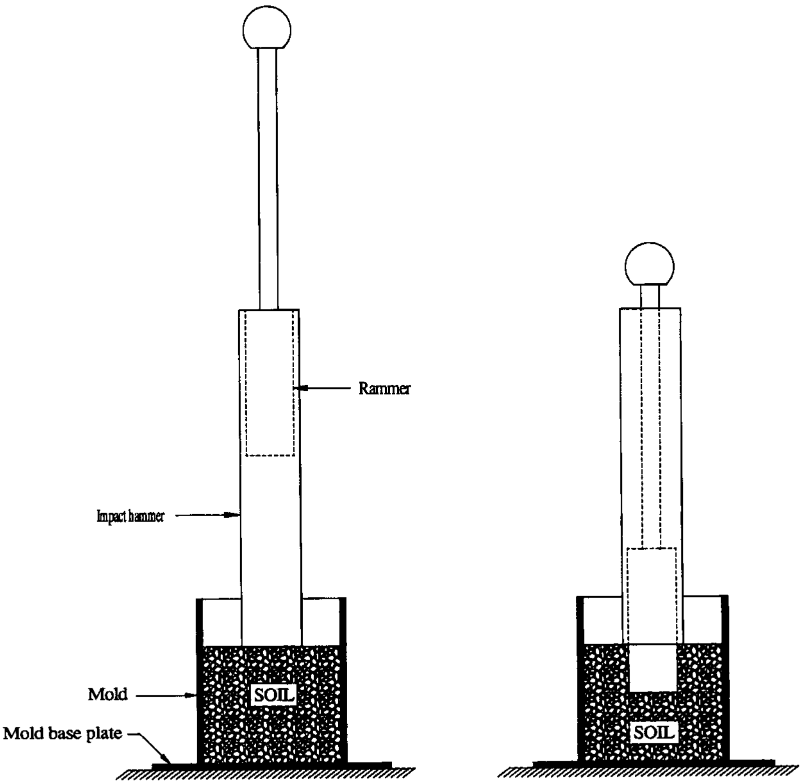


FIGURE 6A.43 Schematic illustration of laboratory impact compaction.

four articles in the *Engineering News-Record* (Proctor, 1933). Two variations of the Proctor test are used—the *standard* and *modified* tests (ASTM D698 and D1557). The basic procedures for the two tests are the same, but the compactive efforts are considerably different. The variation in details for the two tests are outlined in Table 6A.4. The original (standard) Proctor test was developed to model the compactive effort of light rollers in use at that time. The modified Proctor test was developed later to simulate the compactive effort of heavier compaction equipment that subsequently became common.

Two types of procedures can be used to prepare impact-compacted specimens for oedometric, triaxial, and other types of testing. One method involves compacting oversize specimens using a Proctor hammer and mold and trimming the specimens to the desired shape and dimensions. This method is applicable only to cohesive soils and is often difficult to perform on compacted specimens because they do not trim easily at low to moderate water contents (owing to the stiff, brittle nature of the compacted soil). A second procedure involves compacting a specimen in a mold or ring with the desired shape and dimensions using a properly sized impact hammer. For some applications, stock or special order molds and hammers are available from various geotechnical equipment suppliers. If the impact face of the hammer has a diameter that is smaller than the diameter of the confining ring or mold, it is impossible to get the surface of the specimen perfectly flush with the top of the confining ring or exactly perpendicular to the axis of the specimen at the desired height by compaction with the hammer alone. To compensate for this unevenness, the surface of the specimen must be left slightly above the top of the ring or the desired height and brought to the appropriate level in one of the following ways:

TABLE 6A.4 Comparison of Standard and Modified Proctor Tests

Detail	Standard Proctor	Modified Proctor
Rammer		
Diameter*	2.0 in (50.8 mm)*	2.0 in (50.8 mm)*
Drop height	12.0 in (304.8 mm)	18.0 in (457.2 mm)
Weight (mass)	5.5 lb (2.49 kg)	10.0 lb (4.54 kg)
Mold		
Option 1		
Diameter	4.0 in (101.6 mm)	4.0 in (101.6 mm)
Height	4.584 in (116.4 mm)	4.584 in (116.4 mm)
Volume	1/30 ft^3 (944 cm^3)	1/30 ft^3 (994 cm^3)
Option 2		
Diameter	6.0 in (152.4 mm)	6.0 in (152.4 mm)
Height	4.584 in (116.4 mm)	4.584 in (116.4 mm)
Volume	1/13.333 ft^3 (2124 cm^3)	1/13.333 ft^3 (2124 cm^3)
No. of soil layers	3	5
Blows per layer		
Mold Option 1	25	25
Mold Option 2	56	56
Compactive effort	12,375 ft-lb/ft^3 (593 kJ/m^3)	56,250 ft-lb/ft^3 (2693 kJ/m^3)

*Diameter shown is for a hand-held hammer or an automatic compactor used with a 4.0-in (101.6-mm) mold. If an automatic compactor is used with a 6.0-in (152.4-mm) mold, a foot in the shape of a sector of a circle with a radius of 2.90 in (73.7 mm) is used.

1. If the specimen is contained within a confining ring, the specimen can be trimmed level with the top of the ring using a stiff, sharpened, metal trimming bar. For cohesive specimens that will be removed from the mold—such as for unconfined compression or triaxial testing—the specimens can be cut to the proper height using a miter box and wire saw. However, if the specimen is brittle, as is the case for compacted specimens at low to moderate compaction water contents, this method will not work because the specimen will crumble or chunks will break off the specimen, leaving the end uneven and unsuitable for testing.
2. The preferred method is to complete the specimen using static compaction, which involves removing the impact hammer and replacing it with a loading plate that has a diameter slightly smaller than the inside diameter of the ring or mold. The loading plate is then pushed to the top of the ring or to the desired height of specimen via a loading piston (Fig. 6A.40).

6A.3.3.3 Kneading Compaction

Two types of kneading apparatuses are commonly used to compact soils in the laboratory—the Harvard miniature tamper (Wilson, 1970) and the California kneading compactor (ASTM D1561, D2844). The Harvard miniature tamper consists of a 0.5-in (12.7-mm) diameter piston attached to a spring contained within an otherwise hollow handle (Fig. 6A.44). An adjusting nut is used to preset the compression of the spring to a specified force, commonly 40 lb (178 N). Springs of different stiffnesses can be used to provide various values of preset force. The tamper is inserted into the mold and pushed firmly into the soil until the spring just begins to compress; at this point, a force equal to the preset value has been applied to the soil. The tamper is then pulled from the soil, and one “tamp” has been completed.

Specimens for testing can be made in layers within a mold by applying a sufficient number of tamps to each layer to obtain the desired density. Moisture-density relationships can be obtained by

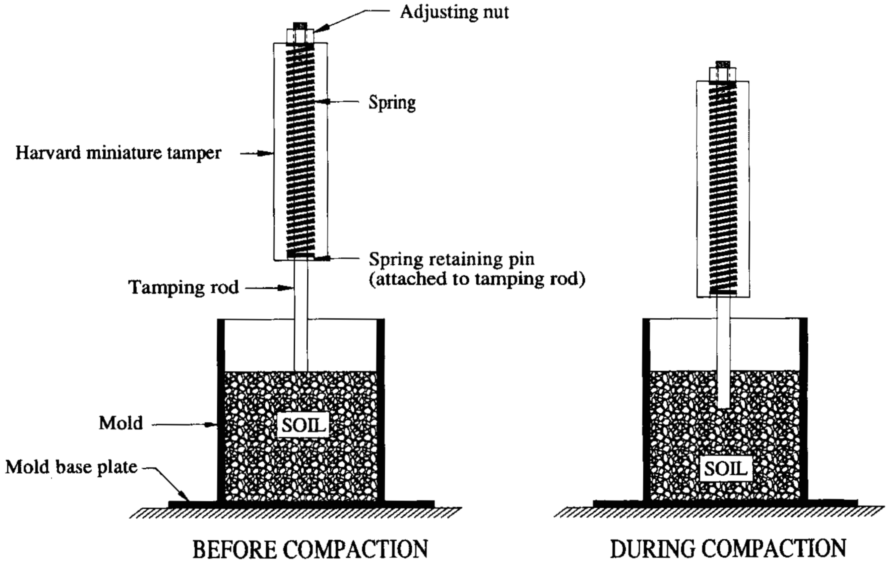


FIGURE 6A.44 Schematic illustration of laboratory kneading compaction using a Harvard miniature tamper.

varying the water content of the soil and applying a predetermined level of compactive effort. Compactive effort is controlled by the volume of soil in the layer and the number of lamps per layer. A standard procedure for conducting moisture–density tests using the Harvard miniature tamper is given in Wilson (1970).

With the California kneading compactor, the soil is compacted within a 4-in (102-mm) diameter mold by applying a constant pressure to the soil via a pie-shaped tamper foot (see Figs. 6A.45 and 6A.46). When each tamp is completed, the mold is rotated an equal angle (usually 5 to 7 tamps per revolution), and another tamp is applied. This process is continued until a given number of tamps

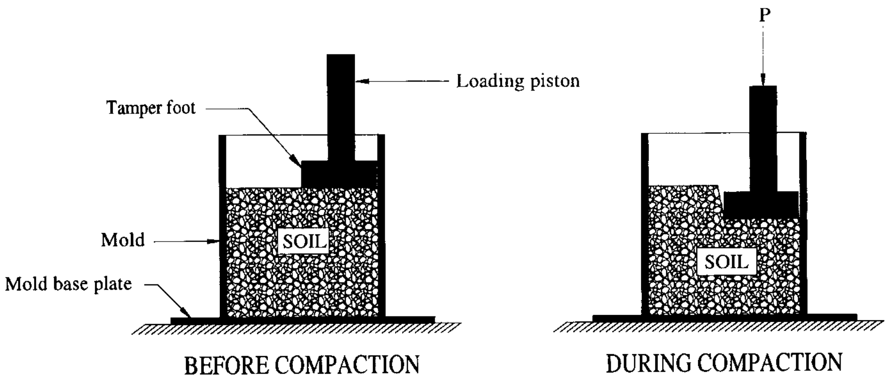


FIGURE 6A.45 Schematic illustration of laboratory compaction using a California kneading compactor.

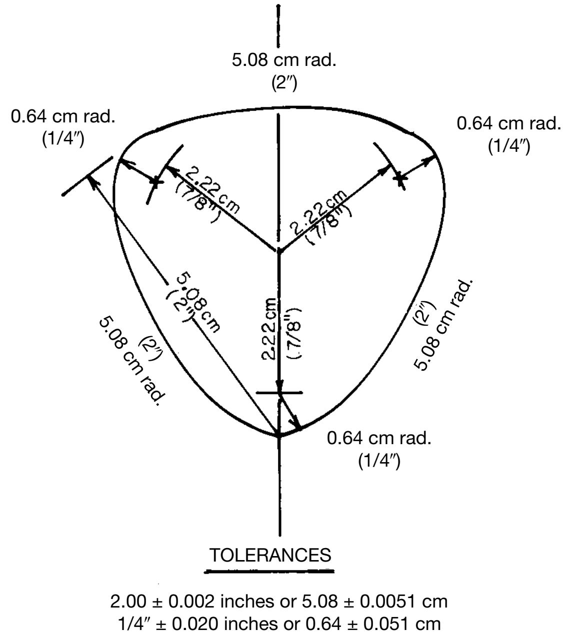


FIGURE 6A.46 Characteristics of pie-shaped tamper foot for California kneading compactor (from ASTM D2844).

have been applied or until additional tamps produce no additional compaction in the lift. The compactive effort applied to the soil can be changed by varying the pressure of application, the lift height, or both. Specimens that will be subjected to additional testing are usually trimmed from the 4-in (102-mm) diameter specimen created during the compaction process.

The top surface of specimens prepared using either the Harvard miniature tamper or the California kneading compactor are generally neither level nor square (perpendicular to the axis). If the specimens are to be tested without being trimmed, a final application of static compaction is needed to provide a level and square surface. Procedures similar to those described for impact compacted specimens in Sec. 6A.3.3.2 can be used.

6A.3.3.4 Vibratory Compaction

In the United States, vibratory compaction of cohesionless soils is most commonly conducted in the laboratory using a vibratory table (Fig. 6A.47). To simulate field compaction, soil at an appropriate water content is placed loosely in a mold, a surcharge weight corresponding to the expected surcharge pressure in the field is placed gently on top of the loose soil, and the mold containing the soil and surcharge weights is secured to a vibratory table and vibrated at a given amplitude and frequency for a period. The soil to be densified can be dry, wet, or moist depending on the field conditions to be modeled. Vibration is continued until the desired density is achieved. If no further densification can be achieved but the desired density has not been obtained, the amplitude of vibration can, in some instances, be varied to achieve a greater densification.

Standard test methods are also available for determining the maximum index density of free-draining soils using a vibratory table (ASTM D4253). Because capillary tension in partially saturated soils creates a resistance to compaction called *bulking* (see Fig. 6A.38), the maximum index den-

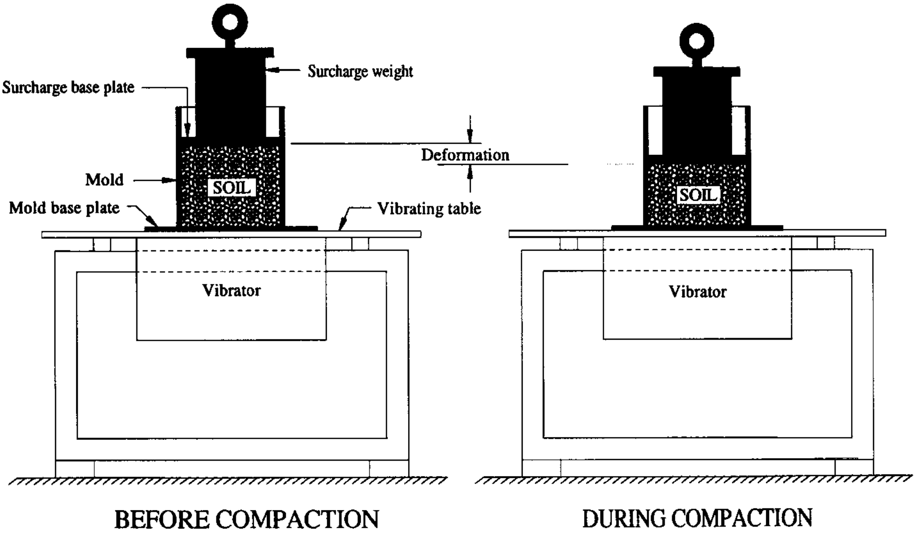


FIGURE 6A.47 Schematic illustration of laboratory compaction using a vibratory table.

sity test is performed on either oven-dried or saturated soil. A surcharge weight corresponding to a pressure of 2 psi (14 kPa) is applied to the soil contained within a mold, and the mold, soil, and surcharge are vibrated vertically at a double amplitude of vertical vibration of 0.013 in (0.33 mm) for 8 min at 60 Hz or at 0.019 in (0.48 mm) for 10 min at 50 Hz. Because the peak density for various soil types and gradations can vary as a function of double amplitude of vibration, ASTM D4253 permits the use of amplitudes other than those given above in special circumstances. It is important to note that an absolute maximum density is not necessarily obtained by these test procedures, rather they provide a standard method for obtaining reproducible values of "maximum" density. The relative density of a compacted soil can be calculated from the actual density of the soil in relation to the minimum (ASTM D4254) and maximum index densities.

Other vibratory methods can be used in the laboratory to compact soils and to determine the maximum index density (Forssblad, 1981). These methods include a vibrating rammer used in Sweden (Fig. 6A.48), a vibrating hammer used in England, and a manually operated steel fork used in Germany.

6A.3.3.5 Special Considerations for Confined-Ring Specimens

When tests are to be conducted on specimens within small confining rings, such as for one-dimensional compression or wetting-induced volume change tests, the samples can be prepared in four ways:

1. The soil can be compacted directly into the confining ring using any of the four laboratory methods of compaction discussed above.
2. A sample can be compacted into an oversize mold in the laboratory using static, impact, or kneading compaction, and the specimen can be trimmed into the ring from the oversize specimen.
3. The ring can be pushed into the field-compacted soil, trimmed, sealed in an air-tight container, and then brought to the laboratory for testing.

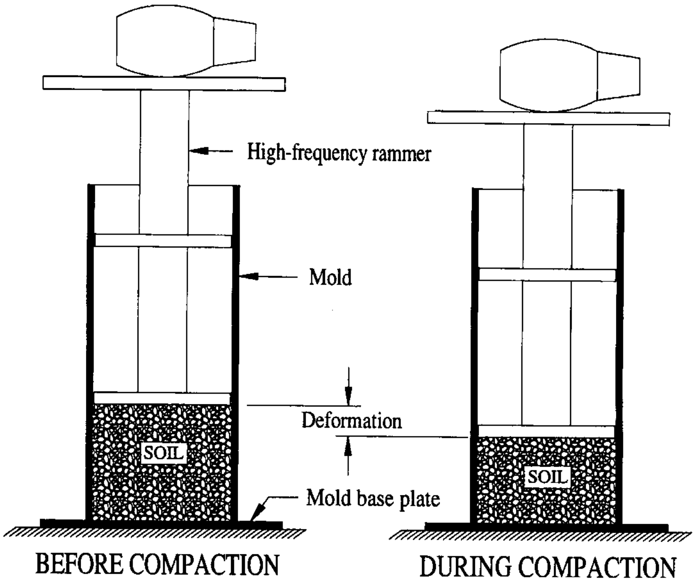


FIGURE 6A.48 Schematic illustration of laboratory compaction using a high-frequency rammer (after Forssblad, 1981).

4. A chunk sample of the field-compacted soil can be brought into the laboratory and a specimen trimmed into the ring from the chunk sample.

For laboratory-compacted specimens, compaction directly into the ring is preferable to trimming an oversize sample because disturbance to the specimen caused during the trimming procedure is eliminated. Typical procedures for compacting specimens directly into confining rings using different methods of compaction can be found in Lawton (1986), Lawton and coworkers (1993), and Booth (1976).

Confined cohesive specimens obtained from the field-compacted soil (methods 3 and 4) are theoretically better than those prepared by either of the two laboratory methods (1 and 2) because the specimens ideally have the same fabric, density, and water content as the soil being modeled (that is, the field-compacted soil). Sometimes, however, the field-compacted soil cannot be trimmed into a confining ring without substantial disturbance, especially if the soil is dry to moderately dry or contains a significant fraction of oversize particles. In addition, a long time may elapse from removal in the field to testing in the laboratory, during which time the specimen may either dry out or bond to the ring. In these instances, laboratory compaction directly into the ring may produce a specimen more representative of the field-compacted soil if a laboratory compaction technique is used that closely simulates the field compaction.

6A.3.4 Engineering Properties and Behavior of Compacted Soils

The engineering behavior and properties of compacted soils are discussed here according to type of soil. For convenience, the soil types are divided into three general categories that differentiate the behavior of most compacted soils: clean coarse-grained soils (those containing less than about 2% fines), granular (cohesionless) soils with fines, and cohesive soils. The reader should bear in mind, however, that deviations from the typical engineering behavior described in the following sections may occur for each type of soil, and it is incumbent upon the engineer for a particular project to determine the relevant engineering properties of the soils to be compacted.

6A.3.4.1 Granular (Cohesionless) Soils

In building applications, the most important static properties for the bearing soil are strength, compressibility, permeability, and potential for volumetric changes induced by variations in available moisture. The engineering behavior of granular soils depends mainly on the relative density of the soil, although cementation of the particles upon aging after compaction may also play an important role. In seismically active regions, liquefaction potential of saturated silty sands and sands is extremely important.

Static Stress-Strain-Strength Behavior. The static settlement of a building to be founded on granular soils can be substantially reduced by compacting the soil to a greater density (lower void ratio) before constructing the building. The influence of relative density on the one-dimensional and isotropic compressibility of sand is shown in Figs. 6A.49 and 6A.50. For example, according to the

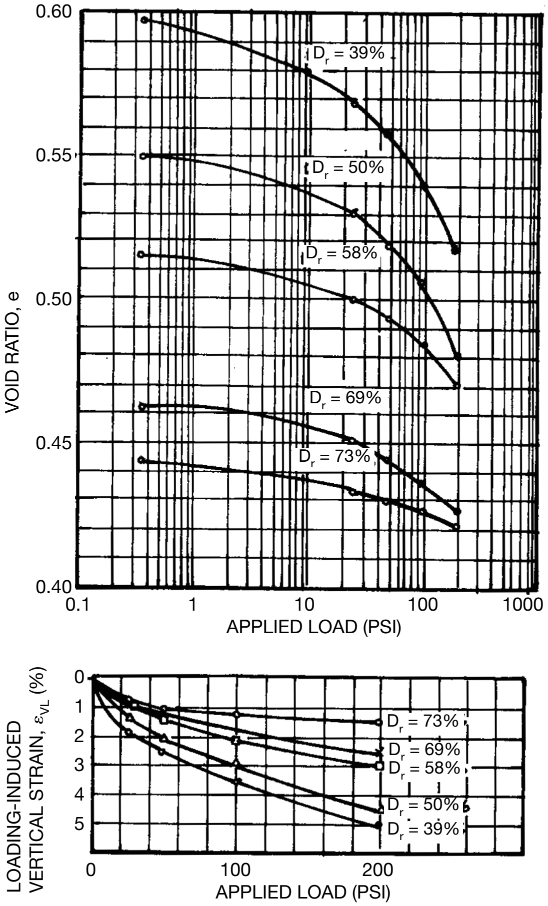


FIGURE 6A.49 Compressibility of a clean medium sand at various relative densities (from Hilf, 1991).

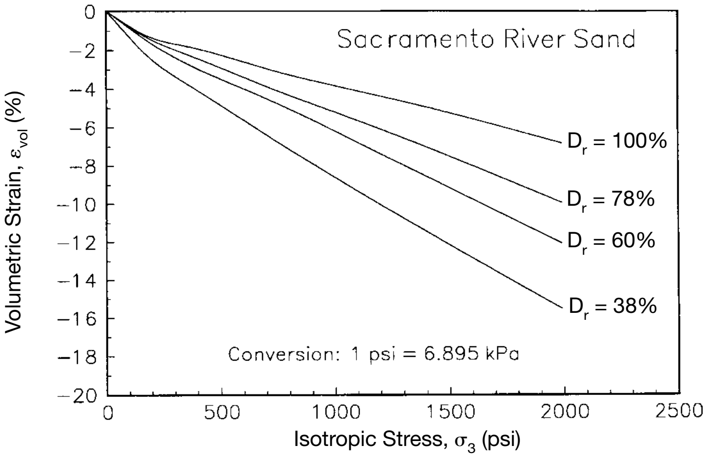


FIGURE 6A.50 Influence of density on the isotropic compressibility of Sacramento River sand (data from Lee and Seed, 1967*b*).

data shown in Fig. 6A.49, the one-dimensional loading-induced vertical strain for Platte River sand (a clean, medium sand) at an applied stress of 25 psi (172 kPa) was about 58% less for $D_r = 73\%$ ($\varepsilon_{VL} = 0.8\%$) than for $D_r = 39\%$ ($\varepsilon_{VL} = 1.9\%$). Similar trends are shown in Fig. 6A.50 for the isotropic compressibility of Sacramento River sand (fine, uniform sand). The $\varepsilon_{vol} - \sigma_3$ plots for all four relative densities are reasonably straight for u3 greater than about 350 psi (2400 kPa), so a measure of the increased stiffness of the soil with increased density can be obtained by calculating values for bulk compressive modulus ($M_B = -\Delta\sigma_3/\Delta\varepsilon_{vol}$), with the following results:

	Bulk modulus, M_B	
D_r, %	ksi	MPa
38	13.9	96
60	17.4	120
78	21.2	146
100	32.5	224

Thus, M_B for $D_r = 100\%$ is more than twice the value of M_B for $D_r = 38\%$.

For foundation conditions in which the width of the foundation is small compared to the total thickness of the compressible strata beneath the bearing level, the strains induced by loading will be three-dimensional, and the magnitudes of the horizontal stresses within the zone of influence for settlement are important. Compaction increases both the density and horizontal stresses within the compacted soil, and both effects increase the stiffness of the soil, as shown by the results in Fig. 6A.51 for drained triaxial tests on Sacramento River sand. Thus, compaction is especially effective in reducing settlement for situations in which horizontal strains contribute significantly to vertical settlement.

The strength and hence ultimate bearing capacity of a granular bearing stratum can be substantially enhanced by a reduction in void ratio produced by compaction. The variation in angle of internal friction between the loose and dense conditions for most naturally occurring granular soils is about 4 to 10°. The influence of density on the drained shearing strength of Sacramento River sand

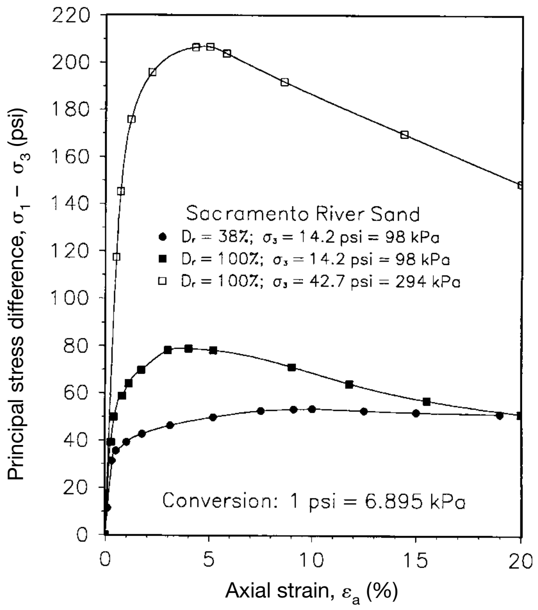


FIGURE 6A.51 Effect of density and confining pressure on the drained triaxial stress-strain behavior of Sacramento River sand (data from Lee and Seed, 1967*b*).

is illustrated in Fig. 6A.52, for which the angle of internal friction at low confining pressures increased from 34° in the loose condition (D_r = 38%) to 41° in the dense condition (D_r = 100%). The magnitude of potential increase in ultimate bearing capacity can be measured by the change in the bearing capacity factors [using Meyerhof's (1951, 1963) factors]:

Condition	D_r, %	ϕ', degrees	N_q	N_γ
Loose	38	34	29.4	31.1
Dense	100	41	73.9	114.0

It can be seen that N_q and N_γ are 151 and 267% greater, respectively, in the dense condition than in the loose condition, indicating that the increase in ultimate bearing capacity would be within this range.

Contrary to common perception, the fabric of coarse-grained soils may have an important effect on compressibility and strength. Thus, method of compaction may affect the fabric of a compacted coarse-grained soil and hence its stress-strain-strength behavior. The effects of fabric anisotropy are greater in coarse-grained soils with elongated grains than in those with more spherical grains (Mitchell, 1993). Fabric anisotropy produces differences in volume change (dilatancy) tendencies for different directions of loading, which results in differences in stress-deformation and strength behavior. This effect is illustrated in Fig. 6A.53 for specimens of crushed basalt tested in direct shear (Mahmood and Mitchell, 1974). At relative densities less than about 90%, specimens sheared along the plane of preferred orientation (along the long axis of most particles) were weaker than

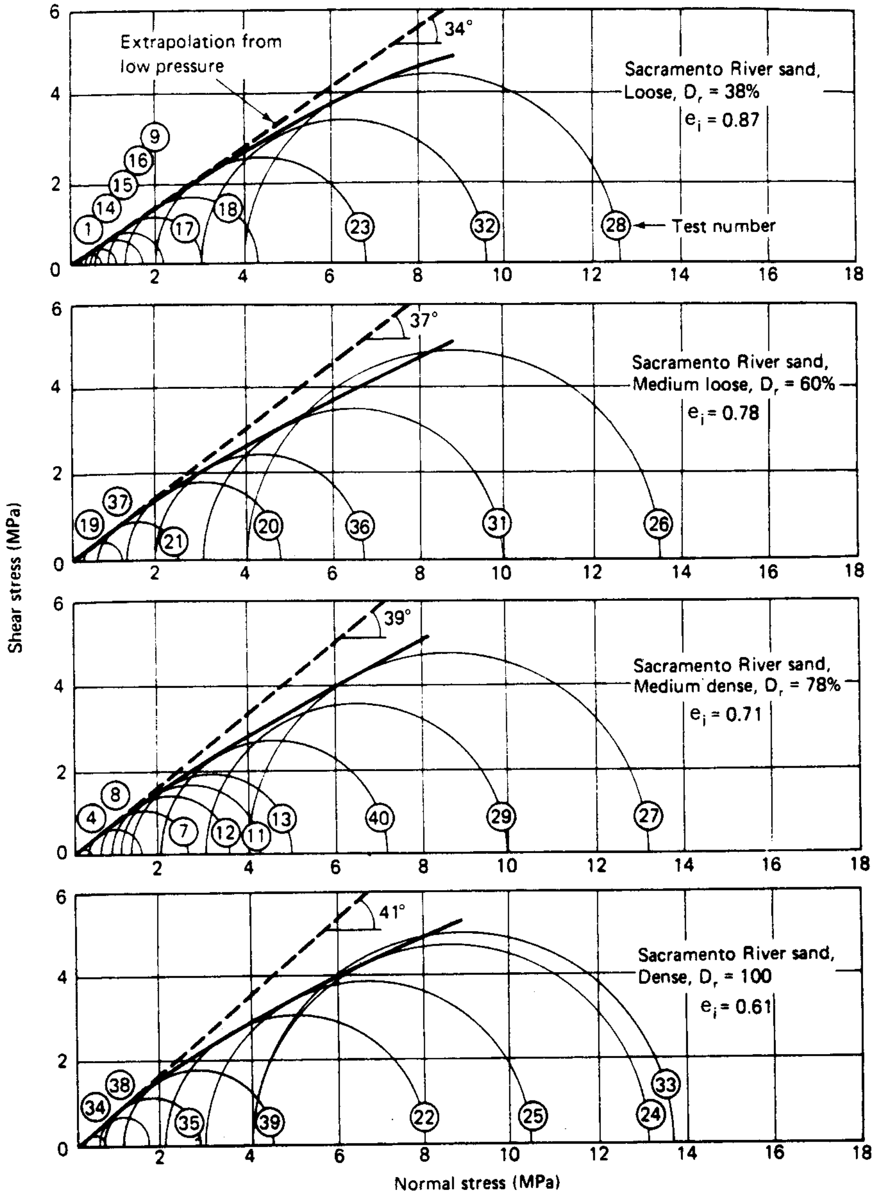


FIGURE 6A.52 Influence of relative density on the drained triaxial shear strength of Sacramento River sand (from Holtz and Kovacs, 1981; data from Lee, 1965 and Lee and Seed, 1967*b*).

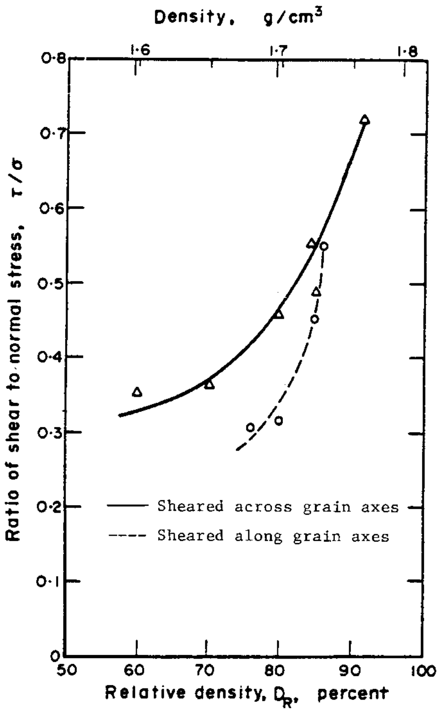


FIGURE 6A.53 Effect of shear direction on the strength of specimens of crushed basalt prepared by pouring into a shear box (from Mahmood & Mitchell 1974).

comparable specimens sheared across the plane of preferred orientation. This difference in strength decreased with increasing density, and the strengths were about the same for relative densities above 90%. These results were consistent with the finding that the intensity of preferred orientation decreased as relative density increased.

The influence of laboratory method of compaction on the triaxial stress-strain and volume-change behavior of a uniform sand with rounded to subrounded grains and a mean axial length ratio of 1.45 is shown in Fig. 6A.54. In these tests, oven-dried soil was poured into a cylindrical mold and then compacted to a dense void ratio of 0.64 by two methods: (1) plunging a hand-held tamper into the sand, and (2) tapping the side of the mold with the hand tamper (vibration). The method of compaction had a significant influence on both the volume-change and stress-deformation characteristics. The specimen compacted by tapping dilated more and was substantially stiffer than the specimen compacted by tamping. These differences in behavior were attributed to differences in fabric—the specimens prepared by tapping tended toward some preferred orientation of long axes parallel to the horizontal plane, and the intensity of orientation increased slightly during deformation; specimens prepared by tamping initially had weak preferred orientation in the vertical direction, and this disappeared with deformation. These results illustrate the importance of using a method of compaction in the laboratory that closely simulates the method to be used in the field, as discussed in Sec. 6A.3.3.

The influence of fabric anisotropy on stress-strain-strength properties of compacted granular soils can be summarized as follows (Mitchell, 1993):

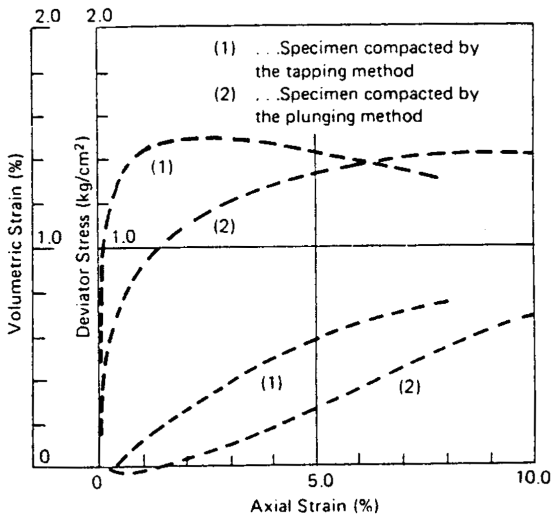


FIGURE 6A.54 Influence of compaction method on the triaxial stress-strain and volume change behavior of a uniform sand (from Mitchell, 1993; data from Oda, 1972).

1. Anisotropic fabric and anisotropic mechanical properties are likely to occur in compacted soils.
2. The magnitude of strength and modulus anisotropy depends on density and the extent to which particles are platy and elongated. When axial ratios of particles are 1.6 or greater, differences in peak strength on the order of 10 to 15% may exist. Stress-strain moduli in different directions may vary by a factor of two or three.

Effect of Moisture on Compressibility and Strength

Clean coarse-grained soils Moisture has little effect on the static stress-strain-strength behavior of most coarse-grained soils, although clean, very fine sands are generally slightly stiffer and stronger in the partially saturated condition than in either the dry or saturated condition owing to matric suction (all other factors being the same). The compressibility and strength of some coarse-grained soils, however, are substantially affected by moisture condition. For example, Bishop and Eldin (1953) showed that the angle of internal friction of a fine to medium clean sand tested in drained triaxial compression was consistently higher in the dry condition than in the saturated condition—about 5° higher for dense condition and 2° higher for loose condition. Triaxial compression tests conducted by Lee and Seed (1967*c*) on a fine uniform sand indicated that the friction angle, stress-strain behavior, and creep behavior were substantially affected by moisture condition. This sand was stronger, less compressible, and exhibited less creep in the oven-dried condition than the saturated condition, with air-dried behavior intermediate between the two extreme states. Increased moisture also caused a decrease in dilatant volume change behavior and an increase in particle crushing during shearing. The moisture sensitivity was therefore attributed to the presence of cracks in some of the particles, into which water infiltrated and weakened those particles so that they crushed during shearing. Moisture sensitivity has also been found in some gravels and rockfills (e.g., Zeller and Wulliman, 1957).

Based on evidence found in the literature and their own work, Lee and Seed (1967*c*) came to the following conclusions regarding moisture sensitivity of granular soils:

From the available information it would appear that moisture sensitivity is likely to be greatest in granular soils whose particles either contain cracks or are susceptible to the formation of fine cracks as a result of loading. Such soils would be those derived from weathered rock, and angular material such as broken rock in which low contact areas would lead to high contact stresses and the formation of minute cracks at the contact points. Furthermore, it would also appear that water sensitivity may be greater at high pressures since fracturing of particles will be facilitated under these conditions.

In light of the available evidence, therefore, it would seem to be desirable to check the nature of the component particles in investigating the strength of cohesionless soils and ascertain their sensitivity to moisture changes.

Granular soils with fines Any granular soil containing particles that are cracked or that develop cracks when loaded may be susceptible to loss of strength and stiffness with increasing moisture content, as discussed in the previous section. Granular soils with a substantial nonplastic fines fraction may also be susceptible to loss of strength and stiffness with increasing moisture owing to a reduction in suction. The total suction in an unsaturated soil can be subdivided into the osmotic suction and the matric suction. Osmotic suction arises from differences in ionic concentrations at different locations in the soil water within a soil. *Osmotic suction* is of significance primarily in cohesive soils where the cations in the soil water are electrochemically attracted to the clay particles, with the result that the concentration of cations is greatest near the surface of the clay particles and decreases with distance from the particles. *Matric suction* is the difference between the pore air pressure and the pore water pressure ($u_a - u_w$) in a partially saturated soil and reflects both the capillary forces and the electrochemical forces between the soil particles and the water molecules in the soil water (adsorptive forces).

In granular soils, the adsorptive forces are small, and thus the matric suction is predominately associated with the capillary menisci that occur in the partially saturated condition owing to *surface tension* that develops along the air-water interface (also called the *contractile skin*). Surface tension is a tensile force per unit length that is tangential to the contractile skin and results from a difference in intermolecular attractions between water molecules along the air-water interface and those within the interior of the water. Within the interior of the water, the water molecules are attracted to neighboring water molecules, and there is no net force acting on any of the water molecules because the molecules are distributed equally in all directions (see Fig. 6A.55). The water molecules along the

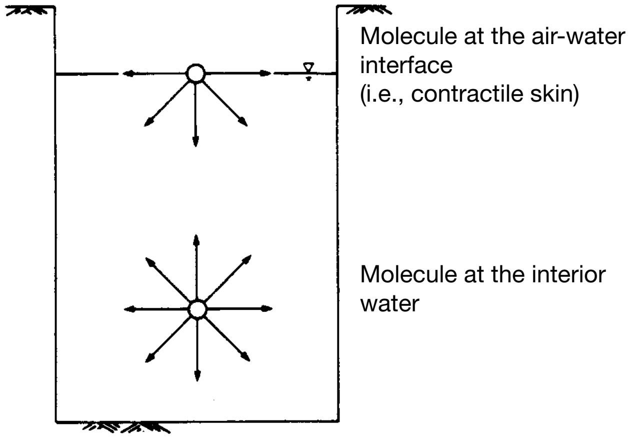


FIGURE 6A.55 Intermolecular forces along the air-water interface and within the interior of water (from Fredlund and Rahardjo, 1993).

air-water interface are attracted to neighboring molecules within the interior and those along the interface, which results in a net attraction into the bulk of the water. Thus, the surface layer of water molecules is pulled toward the interior until the contractile skin assumes a spherical shape, which is the most stable configuration from an energy standpoint because the surface area is minimal (Davis et al., 1984). Surface tension develops, therefore, to overcome the net inward intermolecular force and to provide equilibrium to the contractile skin.

In partially saturated granular soils, u_a is usually zero (atmospheric) or compressive (positive), and u_w is tensile (negative). Kelvin's capillary model equation can be used to relate the matric suction to the curvature of a spherical contractile skin and the surface tension:

$$(u_a - u_w) = \frac{2T_s}{r_m} \tag{6A.16}$$

where T_s = surface tension
r_m = radius of the meniscus (contractile skin)

Theoretical values of $(u_a - u_w)$ from Eq. (6A.16) for a contact angle of zero are shown in Fig. 6A.56 for saturated soils as a function of pore radius, such as might occur at the top of a saturated capillary zone or in a saturated soil as it dries.

The influence of moisture condition on matric suction is illustrated in Fig. 6A.57 using the capillary model. In drier soils (lower degree of saturation), the capillary menisci have smaller radii and higher values of $(u_a - u_w)$. Note, though, that the contact area between the pore water and the soil particles is less for drier soils, as indicated by the parameter χ in the following equation for effective stress in unsaturated soils proposed by Bishop (1959):

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \tag{6A.17}$$

χ relates the portion of $(u_a - u_w)$ that is transmitted to the soil in terms of a change in effective stress. $\chi = 0$ for a completely dry soil and $\chi = 1$ for a saturated soil, and the relationship between χ and de-

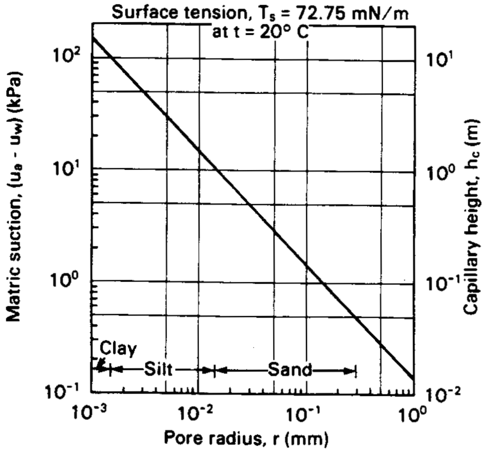


FIGURE 6A.56 Relationships among pore radius, matric suction, and capillary height for saturated soils (from Fredlund and Rahardjo, 1993).

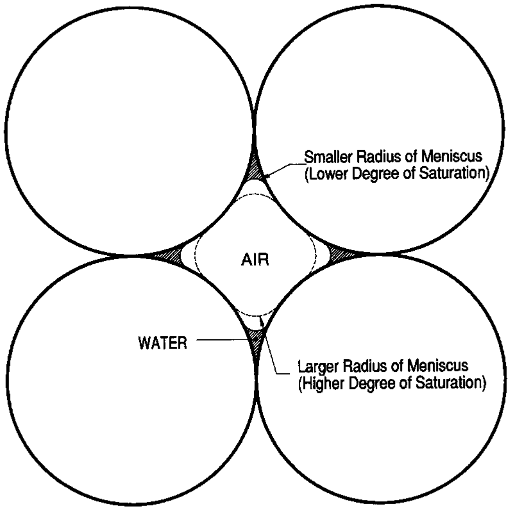


FIGURE 6A.57 Influence of moisture condition on the capillary menisci in an unsaturated soil according to the capillary model.

gree of saturation for a silt is shown in Fig. 6A.58. In granular soils, χ depends primarily on the ratio of area of pore water in contact with the soil particles to the total surface area of the particles but also on factors such as the fabric and stress history of the soil. χ has also been found to differ when determined for volume change behavior and shear strength for the same soil under otherwise identical conditions (Morgenstern 1979).

The net effect of a lower water content may be either to decrease or increase the effective stress in a soil depending upon the combined effect of increased suction and decreased contact area—that is, how the combined factor $\chi(u_a - u_w)$ varies as a function of water content. The relation between matric suction and change in effective stress is shown in Fig. 6A.59 for a clayey sand. This concept is also illustrated for three types of soils in Fig. 6A.60, which shows the unconfined compressive strength as a function of water content. It is clear from Fig. 6A.60 that even perfectly cohesionless soils (e.g., very fine clean sand) can have an unconfined compressive strength greater than zero owing to matric suction, which provides some effective stress in the soil even at essentially zero total stress. The effective stress induced by matric suction in cohesionless soils is sometimes referred to as *apparent cohesion*.

Because the strength of finer-grained soils is not negligible at very low water contents (silty sand and clay in Fig. 6A.60), the capillary model obviously cannot be used by itself to describe the changes in matric suction that result from changes in moisture condition. Thus, the influence of adsorptive forces on matric suction is significant for cohesive soils and will be discussed in more detail in Sec. 6A.3.4.2.

In most granular soils the values of matric suction are small, and the stress-strain-strength behavior of these soils is generally little affected by changes in moisture condition. In current "standard" geotechnical engineering practice, the effect of partial saturation on granular soils is generally ignored for three primary reasons:

1. Ignoring the effect is simpler.
2. The effect is nearly always conservative.
3. There is always a chance the soil will become saturated at some later point.

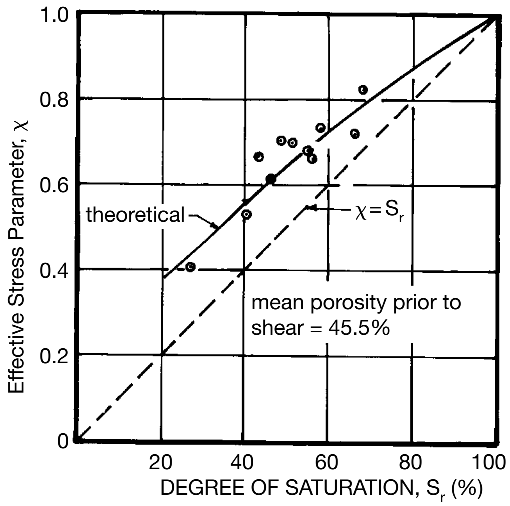


FIGURE 6A.58 Relationship between effective stress parameter χ and degree of saturation for Braehead silt (from Bishop et al., 1960).

Techniques for including the effect of partial saturation are considered to be too difficult to implement in practice.

Thus, most laboratory tests on granular soils are conducted on either dry or saturated specimens, and stress-strain-strength analyses are performed on the basis of total stresses, which are assumed to be equal to effective stresses. In a similar manner, the results of field tests are used to determine appropriate stress-strain-strength parameters based on total stresses, assuming that the total stresses are equal to the effective stresses.

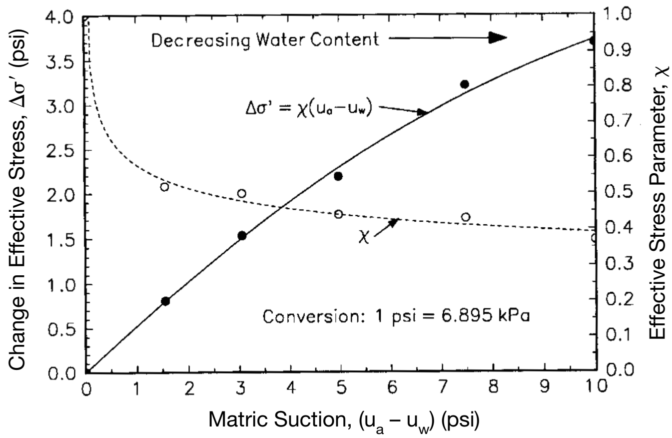


FIGURE 6A.59 Relationship between matric suction and change in effective stress for a clayey sand (data from Blight, 1965).

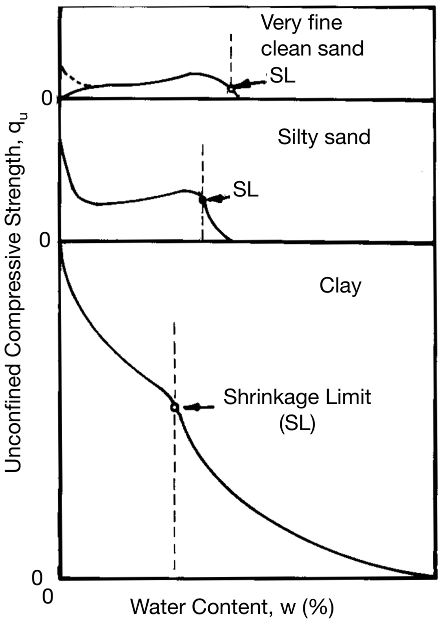


FIGURE 6A.60 Unconfined compressive strength as a function of water content for three types of soils (from Terzaghi and Peck, 1967).

Although ignoring the effects of partial saturation usually has no practical consequence in granular soils, in some instances doing so is uneconomical. Bishop's effective stress equation for unsaturated soils—Eq. (6A.17)—has proved to have little practical use primarily because of difficulties in obtaining values of χ that adequately predict field behavior. These difficulties stem from the fact that stress-strain-strength behavior cannot be uniquely defined in terms of a single effective stress variable (Morgenstern, 1979). Thus, modern methods of predicting the stress-strain-strength behavior of unsaturated soils use two independent stress variables, usually $(\sigma - u_a)$ and $(u_a - u_w)$. Although the use of these more sophisticated techniques is slowly becoming more common in geotechnical engineering practice, most geotechnical engineers prefer to use traditional total stress analyses. Because these techniques have limited application in granular soils, they will not be discussed any further here but will be discussed in more detail in Sec. 6A.3.4.2.

Permeability. Although the terms *permeability* and *hydraulic conductivity* are often used interchangeably in engineering practice, they have different meanings. The *intrinsic permeability* of a soil is independent of the fluid which may flow through it, and the relationship between intrinsic permeability (K) and coefficient of permeability (k) can be described by the following equation (Das, 1983):

$$k = \frac{K \cdot \gamma_f}{\mu_f} \tag{6A.18}$$

where γ_f = weight density (unit weight) of the fluid
μ_f = absolute viscosity of the fluid

Thus, the coefficient of permeability depends on the density and viscosity of the fluid flowing through the soil. The terms *hydraulic conductivity* and *water permeability* describe the same characteristic of a soil—the relative ease with which water can flow through the voids in the soil. In soils, the two most common fluids that flow through the soil in foundation engineering contexts are water and air. For simplicity, the term *permeability* will be used to mean water permeability or hydraulic conductivity, and the permeability with respect to other types of fluids will be differentiated as required.

The coefficient of permeability of clean, coarse-grained soils increases as void ratio (e) increases. Several correlations have been proposed to describe this relationship, including the following (Das, 1983):

$$k \propto \frac{e^3}{1+e} \tag{6A.19a}$$

$$k \propto \frac{e^2}{1+e} \tag{6A.19b}$$

$$k \propto e^2 \tag{6A.19c}$$

Permeability is important in granular soils with respect to their drainage characteristics. It is advantageous in most instances for granular bearing soils to be free-draining, that is, any significant accumulation or introduction of moisture should drain rapidly through the granular layer. Although compaction results in somewhat lower permeability, the permeability of even dense coarse-grained soils is generally still high enough to permit free drainage under most conditions.

k decreases with decreasing grain size, as illustrated by the typical values for various types of granular soils given in Table 6A.5. Casagrande (1938) has established the boundary between good drainage and poor drainage as 1×10^{-4} cm/s (3×10^{-1} ft/day). Thus, granular soils with a large percentage of fines may not be free-draining soils and may require a perimeter or interior drainage system to prevent unwanted buildup of moisture in situations where moisture accumulation may have a deleterious effect on the load-carrying capacity of the foundation soil.

Wetting-Induced Volume Changes

Clean coarse-grained soils Coarse-grained soils will not swell (increase in volume) when wetted because they contain no expansive clay minerals. However, all compacted soils are susceptible to *wetting-induced collapse* or *hydrodensification* (decrease in volume when wetted) under some conditions (Lawton et al., 1992). The magnitude of collapse depends on the state of the soil at the *time of wetting*; in general, collapse increases with increasing mean normal total stress, decreasing water content or degree of saturation, and decreasing dry density (Lawton et al., 1989, 1991). Therefore, one must consider the potential changes in these properties that may occur between the end of

TABLE 6A.5 Typical Values of Coefficient of Permeability for Granular Soils

	k	
Type of soil	cm/s	ft/day
Clean gravel	1×10^0 to 1×10^2	3×10^3 to 3×10^5
Clean sand	1×10^{-3} to 1×10^0	3×10^0 to 3×10^3
Silty sand/sandy silt	1×10^{-4} to 1×10^{-2}	3×10^{-1} to 3×10^1
Silt	1×10^{-6} to 1×10^{-4}	3×10^{-3} to 3×10^{-1}

compaction and the time when wetting may occur. Examples of changes that may occur include the placement of additional fill or structures above a given lift, which increases the stresses acting on that lift and densifies the soil (thereby changing both the state of stress and density); drying of the exposed surfaces of the soil from evaporation or transpiration (which changes the moisture condition); and redistribution of moisture throughout the partially saturated soil as it tries to achieve hydraulic, thermal, chemical, and electrical equilibrium.

The addition of water necessary for collapse in compacted soil may occur in a variety of ways, including precipitation, irrigation, regional ground-water buildup, broken water pipes, moisture buildup beneath covered areas, ponding, inadequate or ineffective drainage, and flooding (Lawton et al., 1992). Some of these methods are difficult to predict or foresee, so it is prudent to consider the wetting-induced collapse potential for all compacted soils.

Although some notable exceptions have occurred, compacted coarse-grained soils will not collapse significantly upon wetting under most circumstances. Some examples of clean coarse-grained soils that have collapsed are provided below to illustrate the potential problems.

Jaky (1948) reported that apartment houses and other structures along the banks of the Danube suffered extensive damage and cracking when the level of the Danube rose 2 m (6.2 ft) above the flood stage. The results from one-dimensional collapse tests on specimens made from six different granular soils indicated that the collapse potential could be high for all the soils under certain conditions. Grain-size distribution properties and USCS classification for the six soils studied are summarized in Table 6A.6, and the effect of relative density on measured wetting-induced collapse strains is shown in Fig. 6A.61. As would be expected, the collapse potential decreased as the relative density of each soil increased. The optimal or critical relative density (the minimum relative density for which the collapse was zero) varied from 69 to 97%.

Collapse settlements have also been observed in rockfill dams. Sowers and colleagues (1965) reported that the Dix River dam was accidentally flooded during construction, with a wetting-induced vertical strain of about 0.5%. They also conducted one-dimensional wetting tests on specimens of freshly broken rocks of various types and found that additional (collapse) settlements occurred rapidly when the dry specimens were wetted. These collapse settlements were attributed to water entering microfissures in the particles that were produced at highly stressed contact points, with the water causing local increases in stress and additional crushing of particles at the contact points. Collapse settlements were also observed during filling of the El Infiernillo dam (Marsal and Ramizeq de Arellano, 1967).

The results of triaxial collapse tests on specimens of Antioch sand and Ottawa sand were reported by Lee and Seed (1967c). In these tests, oven-dried or air-dried dense (D_r = 100%) specimens of both soil types were first loaded to an axisymmetric state of stress with the sustained deviator stress ($\sigma_1 - \sigma_3$) equal to 76 or 83% of the maximum deviator stress for Antioch sand and 90% of the maximum deviator stress for Ottawa sand. To ensure that the pore water pressures

TABLE 6A.6 Grain Size Properties and Classification of Six Granular Soils Investigated by Jaky (1948)

Soil	Grain size distribution by weight, %*			D_{50}, mm	D_{10}, mm	USCS Classification*	
	Gravel	Sand	Silt			Symbol	Descriptive name
Fine sand	0	97	3	0.14	0.10	SP	Poorly graded sand
Coarse sand	0	100	0	0.28	0.15	SP	Poorly graded sand
Danube gravel 1	43	57	0	3.0	0.19	SW	Well-graded sand with gravel
Danube gravel 2	31	52	17	2.2	0.041	SM	Silty sand with gravel
Pea gravel	20	80	0	1.6	0.29	SW	Well-graded sand with gravel
Slag	N.A.	N.A.	N.A.	N.A.	N.A.	—	—

*ASTM D-2487
N.A. = not available

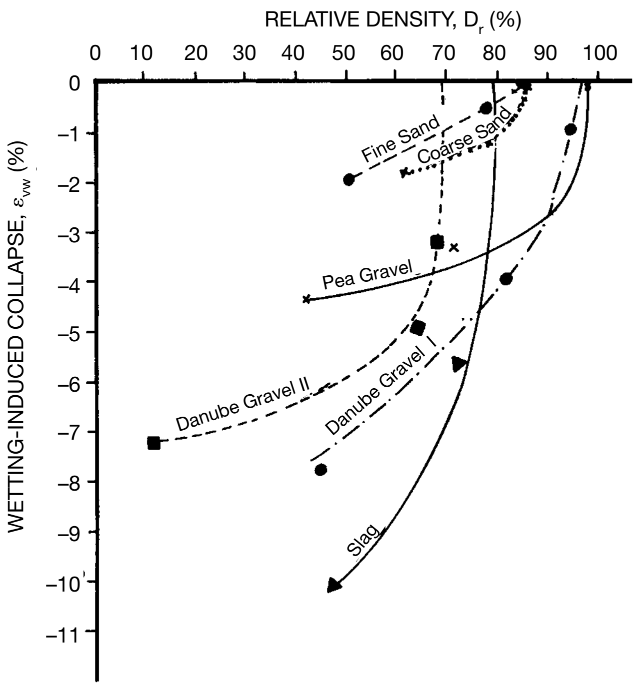


FIGURE 6A.61 Influence of relative density on the one-dimensional collapse of six granular soils (from Jaky, 1948).

were maintained at a very low level during wetting, water was introduced into the specimens by gravity flow through the bottom drainage line under a head of less than 1.0 in (25 mm). Upon wetting, the Antioch sand specimens first underwent substantial collapse strains before culminating in a sudden large deformation along a well-developed shear plane. In contrast, the addition of water to the Ottawa sand specimens produced virtually no collapse. The differences in collapse behavior for these two types of soils were attributed to differences in the soundness of the particles—a significant number of the Antioch sand particles contained small cracks, whereas the Ottawa sand grains were essentially sound.

Hellweg conducted more than 300 one-dimensional collapse tests on specimens of various sands from northern Germany with different densities, moisture contents, and applied loads (Rizkallah and Hellweg, 1980). The results from these tests indicated that uniform fine sands can only be compacted with extraordinary difficulties and that such soils collapse about 8% when wetted. Based on these results, it was concluded that compacting uniform fine sands to a high relative compaction based on the standard Proctor maximum dry density does not necessarily ensure low collapse potential.

It has been recognized for a long time that fill materials containing soft rock fragments—such as shales, mudstones, siltstones, chalks, and badly weathered igneous and metamorphic rocks—can deform significantly upon wetting when used in embankment dams and should be avoided where possible (Sherard et al. 1963). That excessive wetting-induced settlement can also occur in buildings founded on compacted fills containing soft rock fragments has been reported by Wimberley and coworkers (1994). In this project, the excessive settlement (>12 in = 30 cm) of a one-story building founded on a 50-ft (15-m) deep compacted fill occurred. The compacted fill consisted of fine-grained fill with rock fragments (siltstone, limestone, claystone, and sandy shale) and some rocky layers. Much of the settlement was believed to relate to the introduction of moisture into the fill by precipitation and watering of plants, which caused softening of highly stressed contact points between coarser particles and degradable coarse particles (e.g., poorer shales).

In summary, the collapse potential of compacted clean coarse-grained soils under most conditions is insignificant. However, substantial collapse potentials can exist if one or more of the following factors are present in the compacted soil: (1) The compaction is poor or insufficient; (2) defects exist within some of the particles; (3) some of the particles are water soluble or degradable; or (4) the soil is subjected to very high stress levels. The types of granular soils that have defects sufficient to cause potentially high wetting-induced collapse potentials include those derived from highly weathered rock and blasting. Angular particles with low contact areas and high applied stresses seem to be more susceptible to the formation of minute cracks at contact points. Compacted soils containing substantial amounts of soft rock fragments may also be susceptible to wetting-induced collapse. Structures founded on deep fills are especially susceptible to damage resulting from collapse settlements because of the high stresses generated at lower portions of the fill from the weight of the overlying material.

Because potential problems from wetting-induced collapse can be detected by performing one-dimensional collapse tests on representative compacted specimens, it is recommended that collapse tests be conducted in all situations where collapse is possible. Granular soils containing medium and fine sand particles only can be tested in standard consolidometers; soils containing larger particles require special testing equipment that can be found in some universities and other research institutes. For example, San Diego State University has the facilities to test compacted specimens 4 ft (1.2 m) in diameter (Noorany, 1987).

Granular soils with fines There are several cases described in the literature of damage resulting from wetting-induced collapse of compacted granular soils containing fines. Examples include severe cracking of two earth dams in California (Leonards and Narain, 1963), and deformation, cracking, and failure of numerous highway embankments in South Africa (Booth, 1977). The earth dams were constructed of sand/silt mixtures, and silty sands and wetting occurred during filling of the reservoirs. The highway embankments were composed of silty sands, with wetting generally resulting from infiltration of water following periods of heavy rainfall.

Collapse settlements in compacted granular soils generally occur because the soils are too dry, not dense enough, or both. For Woodcrest dam (Leonards and Narain, 1963), which suffered severe cracking, the average compaction water content was 1.7% below standard Proctor optimum, the average degree of saturation was 30% below optimum saturation, and the average standard Proctor relative compaction was 94%. The highway embankments reported by Booth (1977), which suffered extensive damage from collapse settlements, were loosely compacted (modified Proctor relative compaction, R_m = 80%) and were very dry (estimated degree of saturation, S_r < 10% in some instances). It was not known whether the embankments were compacted dry (no compaction water content records were available) or whether substantial moisture loss occurred between compaction and wetting.

Recent research (Alwail et al. 1992) has shown that the amount of silt and shape of the silt particles has an important influence on the collapse potential of granular soils. These effects are shown in Fig. 6A.62 for compacted mixtures of uniform silica sand and two different silts compacted to R_m = 90% at water contents 3% dry of modified Proctor optimum water content (w_{om}). For both silts, the maximum one-dimensional collapse potential for applied vertical stresses ranging from 0.52 to 33.4 ksf (25 to 1600 kPa) increased as the percentage of silt was increased. It is also evident that the group 2 soils had substantially higher collapse potentials than the group 1 soils. Scanning electron micrographs taken on samples of the two groups of soils showed that the group 1 silt grains were rounded to subrounded, whereas the group 2 silt grains were flaky and angular. Thus, the greater collapse potential for the group 2 soils was attributed to a more open structure resulting from the angular and flaky shapes of the silt grains.

Shrinkage. Shrinkage (decrease in volume induced by drying) is not a problem in granular soils because by definition granular soils are cohesionless, and thus they contain no appreciable amount of expansive minerals.

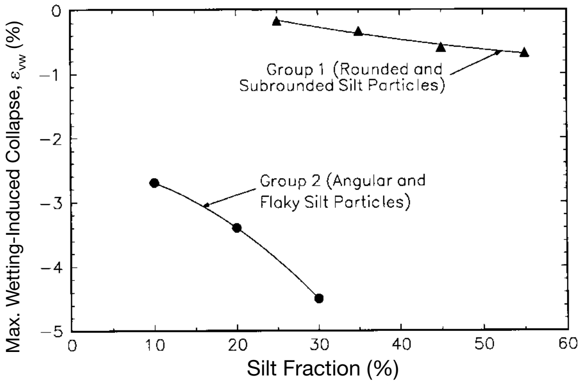


FIGURE 6A.62 Effect of silt content and shape on one-dimensional collapse of sand-silt mixtures (from Alwail et al., 1992).

Liquefaction Potential. Liquefaction occurs in a saturated sandy soil when deformation during undrained or partially drained conditions produces a buildup in pore water pressure to a level that approaches or equals the total confining pressure. At this point the effective confining pressure is zero or near zero, and the soil will continue to deform until enough water has been squeezed from the soil to dissipate a substantial portion of the excess pore water pressure. Liquefaction may occur from either statical strains induced from monotonic loading (e.g., Casagrande 1936*a*) or from cyclic strains induced by dynamic or vibratory loading (e.g., Lee and Seed, 1967*a*). Damage to buildings caused by static liquefaction is rare, so only liquefaction caused by cyclic loadings (also called *cyclic mobility*) will be considered herein.

Extensive damage can occur to buildings and other engineering structures during earthquakes owing to liquefaction in saturated sandy soils beneath the structures. For example, hundreds of buildings were severely damaged as a result of liquefaction during the 1964 earthquake in Nigata, Japan (Seed and Idriss, 1967). Many structures settled more than 3 ft (0.9 m), and the settlement was frequently accompanied by severe tilting (up to 80°). Lateral ground displacements generated during earthquakes by liquefaction-induced lateral spreads and flows have also caused severe damage to structures and their appurtenances (Youd, 1993). Furthermore, liquefaction can occur from lower levels of excitation produced by other phenomena, including blasting, pile driving, construction equipment, road and train traffic, and dynamic compaction (Carter and Seed, 1988). Thus all structures underlain by saturated sand deposits may be susceptible to liquefaction-induced damage, and it is imperative that liquefaction potential be considered for these cases even when the site is not located in a seismically active region.

Three definitions differentiate the possible states of liquefaction (Lee and Seed, 1967a):

Complete liquefaction—when a soil exhibits no resistance or negligible resistance over a wide range of strains (e.g., a double amplitude of 20%).

Partial liquefaction—when a soil exhibits no resistance to deformation over a strain range less than that considered to constitute failure.

Initial liquefaction—when a soil first exhibits any degree of partial liquefaction during cyclic loading.

The following characteristics are known with respect to liquefaction in saturated sandy soils (Lee and Seed, 1967*a*; Seed, 1986):

1. Application of cyclic stresses or strains will induce liquefaction or partial liquefaction over a considerable range of densities, even in moderately dense sands.
2. If the generated pore pressures do not exceed about 60% of the confining pressure, liquefaction will not be triggered in the soil, and there is no serious deformation problem.
3. The smaller the cyclic stress or strain to which the soil is subjected, the higher is the number of stress cycles required to induce liquefaction (Fig. 6A.63).
4. The higher the confining pressure acting on a soil, the higher the cyclic stresses, strains, or number of cycles required to induce liquefaction (Fig. 6A.64).
5. The higher the density of a soil, the higher the cyclic stresses, strains, or number of cycles required to induce liquefaction (Figs. 6A.63 and 6A.64).
6. When loose sandy soils liquefy under cyclic stresses of constant amplitude, deformations immediately become very large (complete liquefaction occurs quickly—see Fig. 6A.65).
7. When dense sandy soils liquefy under cyclic stresses of constant amplitude, deformations are initially small (partial liquefaction occurs) and gradually increase with increasing number of cycles (Fig. 6A.66). Even when the pore water pressure becomes equal to the confining pressure in dense sands, it is often of no practical significance because the strains required to eliminate the condition are very small. Thus, dense cohesionless soils do not normally present problems with regard to the design of foundations.

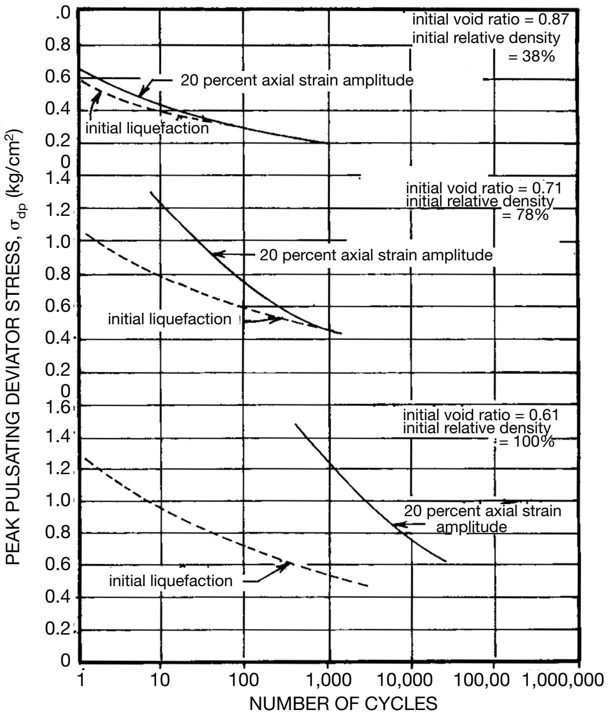


FIGURE 6A.63 Effect of density, number of cycles, and failure criterion on the cyclic stress required to cause liquefaction of Sacramento River sand at σ_3 = 1.0 kg/cm^2 (from Lee and Seed, 1967*a*).

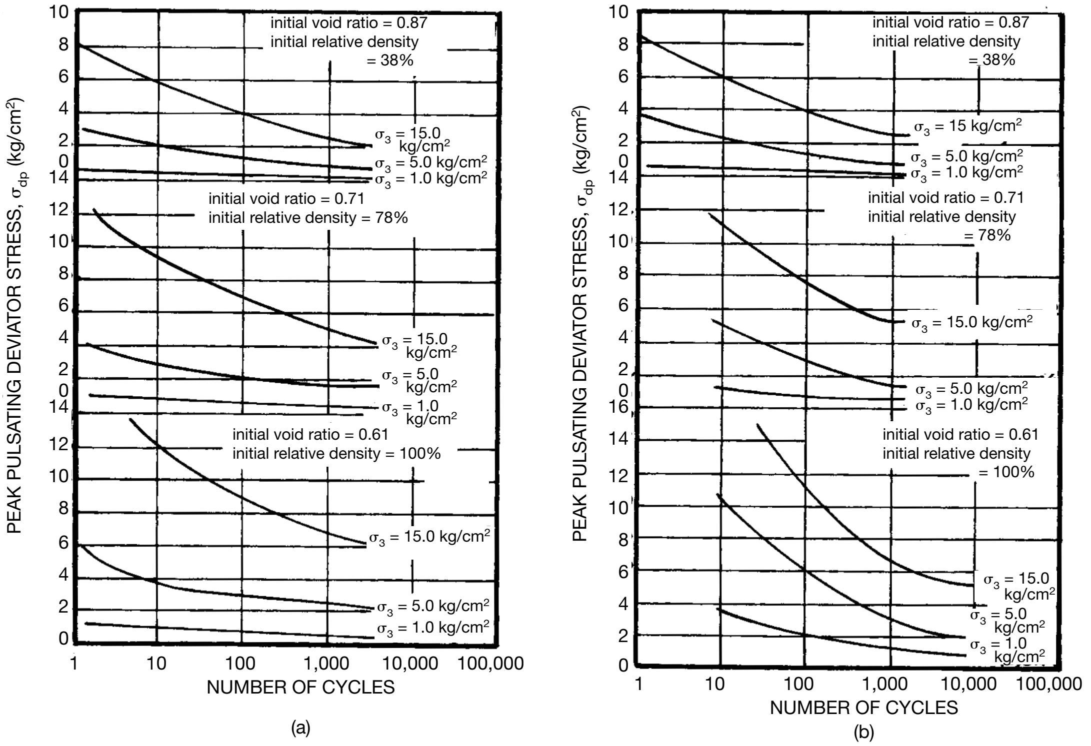


FIGURE 6A.64 Influence of confining pressure, density, and number of cycles on the cyclic stress required to cause (*a*) initial liquefaction and (*b*) 20% strain in Sacramento River sand (from Lee and Seed, 1967*a*).

8. Numerous structures built on liquefiable soils have stood for hundreds of years without liquefaction occurring, simply because there has been no triggering mechanism strong enough to induce liquefaction. Therefore, liquefaction can be prevented by designing to keep the pore pressures well below the confining stresses.

It is evident from the preceding information that the liquefaction potential of any saturated sandy soil depends on both the properties of the soil (density, permeability, and lengths of drainage paths) and the characteristics of the cyclic loading (magnitude of stress and strain, duration). Three possible methods have been used to various extents for estimating liquefaction potential (Youd, 1993): (1) analytical methods, (2) physical modeling, and (3) empirical procedures. Because it is difficult to model the soil conditions at liquefiable sites either analytically or physically, empirical procedures are commonly used in routine engineering practice. A complete discussion of methods for evaluating liquefaction potential for all types of cyclic loading is beyond the scope of this presentation, so details are given below only for earthquake-induced liquefaction. Methods for evaluating liquefaction for cyclic loads from other sources (blasting, pile driving, construction equipment, road and train traffic, and dynamic compaction) are given in Carter and Seed (1988).

The factor of safety against the occurrence of earthquake-induced liquefaction is commonly defined as the available soil resistance to liquefaction (expressed in terms of the cyclic stresses required to cause liquefaction) divided by the cyclic stresses generated by the design event (Youd, 1993). Both factors are usually normalized with respect to the preevent effective overburden stress at the depth being analyzed. In equation form, the factor of safety is defined as

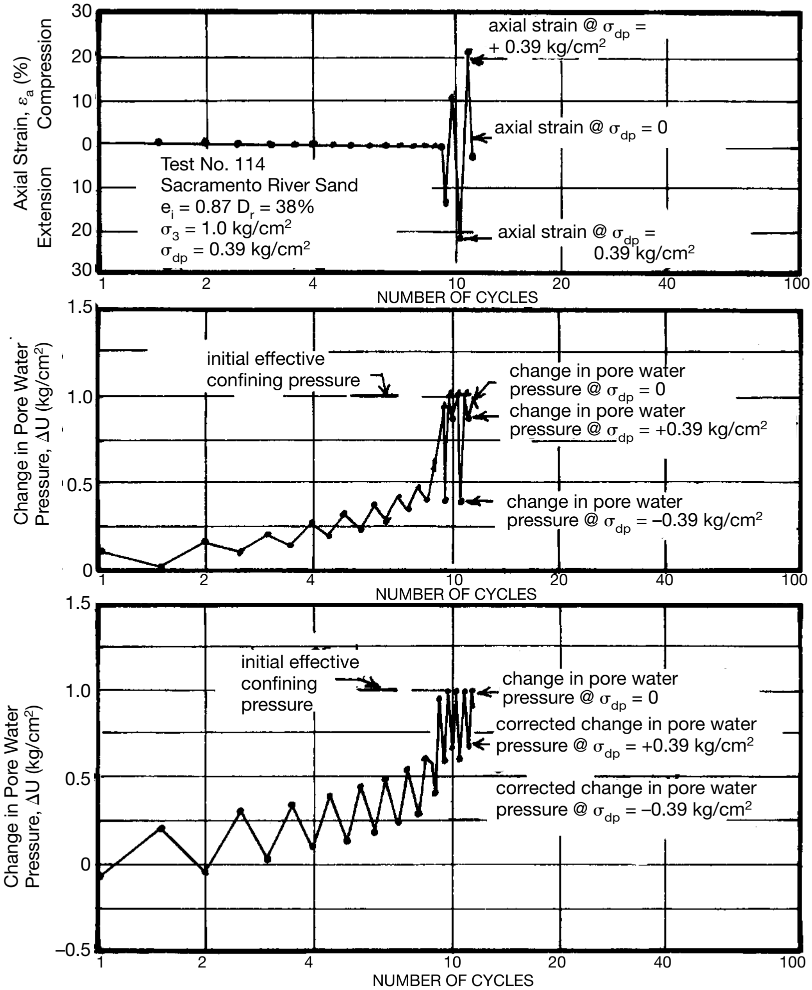


FIGURE 6A.65 Results from pulsating load test on loose Sacramento River sand (from Seed and Lee, 1966).

$$F_s = \frac{\text{CSRL}}{\text{CSRE}} \tag{6A.20}$$

where CSRL = cyclic stress ratio required to generate liquefaction
CSRE = cyclic stress ratio generated by the design earthquake = τ_{av}/σ'_{v0}
τ_{av} = average earthquake-induced cyclic shear stress
σ'_{v0} = preearthquake effective overburden stress at the depth under consideration

CSRE can evaluated either by using a computer code (such as SHAKE or DESRA) to estimate τ_{av} or by estimating it directly from the following equation (Seed & Idriss 1971):

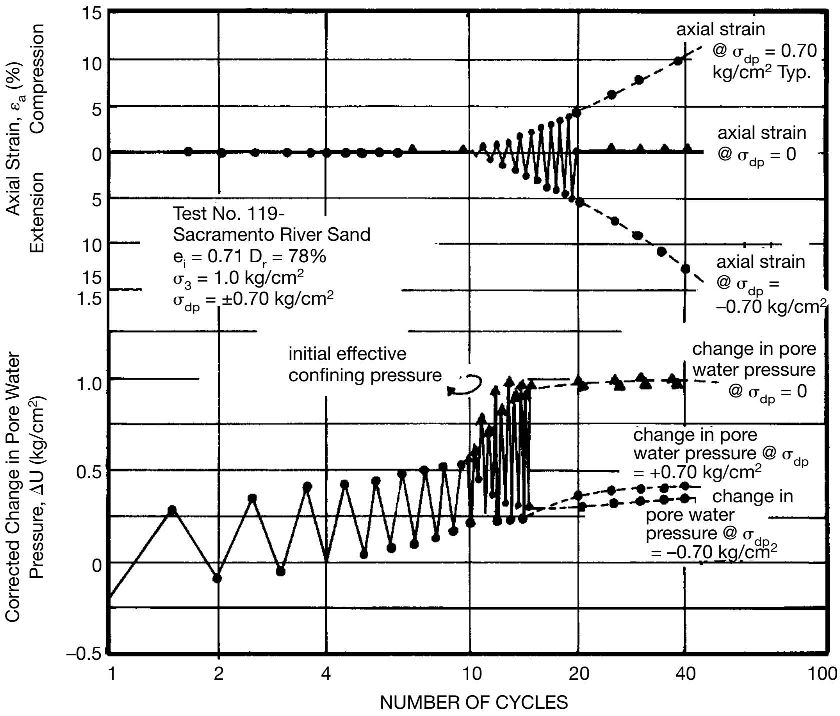


FIGURE 6A.66 Results from pulsating load test on dense Sacramento River sand (from Seed and Lee, 1966).

$$\text{CSRE} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot r_d \tag{6A.21}$$

where $a_{\max}$ = maximum acceleration at the ground surface that would occur in the absence of liquefaction

g = acceleration caused by gravity

σ_{v0} = total overburden stress at the depth under consideration

r_d = depth-related stress reduction factor that varies with depth (z) from the ground surface

r_d can be estimated from the following equation for noncritical projects (NCEER 1997):

$$r_d = 1.0 - 0.00765\,z \qquad \text{for } z \leq 9.15 \text{ m} \tag{6A.21a}$$

$$r_d = 1.174 - 0.0267\,z \qquad \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \tag{6A.21b}$$

$$r_d = 0.744 - 0.008\,z \qquad \text{for } 23 \text{ m} < z \leq 30 \text{ m} \tag{6A.21c}$$

$$r_d = 0.50 \qquad \text{for } z > 30 \text{ m} \tag{6A.21d}$$

where z is the depth below the ground surface in meters.

Estimated values for a_{max} are usually determined using one of the following three methods (Youd, 1993): (1) standard peak acceleration attenuation curves that are valid for comparable soil conditions; (2) standard peak acceleration attenuation curves for bedrock sites, with correction for local site effects using standard site amplification curves or computerized site response analysis; and (3) probabilistic maps of a_{max}, with or without correction for site attenuation or amplification depending on the rock or soil conditions used to generate the map.

CSRL is most commonly estimated from empirical correlations with corrected SPT blowcount $[(N_1)_{60}]$ such as those shown in Fig. 6A.67, which are valid for magnitude 7.5 earthquakes and sands and silty sands with up to 35% fines. Open symbols represent sites where surface liquefaction did not develop, and filled symbols are from sites where surface liquefaction did occur. Curves are given in the figure for varying percentage of fines that represent the boundary between the liquefaction and no liquefaction zones. Youd (1993) recommends using a factor of safety of 1.2 for engineering design based on this chart because it is possible that liquefaction may have occurred at some sites

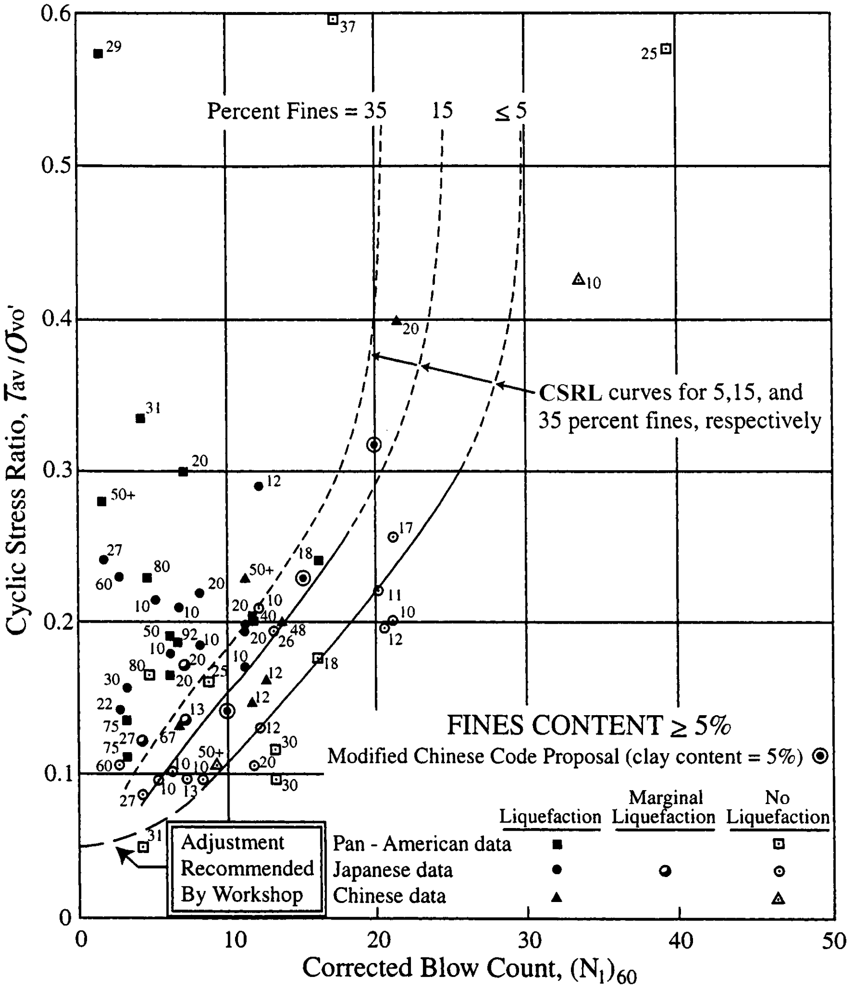


FIGURE 6A.67 Relationship between CSRL and $(N_1)_{60}$ values for silty sands and magnitude 7.5 earthquakes (from NCEER 1997, modified from Seed et al. 1985).

but was not detected at the ground surface. One should exercise care when using the curves in Fig. 6A.67 for soils containing substantial gravel fractions because verified corrections for gravel content have not been established (Youd, 1993).

Correlations between CSRL and a number of other parameters besides SPT blowcount have recently been developed and are summarized in NCEER (1997). These other parameters include tip resistance (q_c) from the Cone Penetration Test, shear wave velocity, and blowcount from the Becker Penetration Test. Details of these correlations are beyond the scope of this handbook and can be found in NCEER (1997).

$(N_1)_{60}$ is the field SPT blowcount corrected for overburden pressure (reference stress = 1 tsf = 95.8 kPa), energy efficiency, length of drill rod, and characteristics of sampling barrel (Seed et al., 1985) and can be represented by the following equation:

$$(N_1)_{60} = N_{\text{field}} \cdot C_E \cdot C_N \cdot C_L \cdot C_S \cdot C_B$$

where N_{field} = SPT blowcount measured in the field
C_E = correction factor for energy ratio
C_N = correction factor for overburden pressure
C_L = correction factor for length of drill rod
C_S = correction factor for sampling barrel
C_B = correction factor for borehole diameter

$(N_1)_{60}$ is standardized to an energy ratio (actual energy divided by theoretical free-fall energy) of 60%, so the energy ratio correction can be calculated as

$$C_E = \frac{E_r}{60} \tag{6A.23}$$

where E_r = energy ratio for the equipment being used

E_r can be either measured during use of the actual equipment (preferred) or estimated from the type of equipment (e.g., Table 6A.7). C_N can be obtained from Fig. 6A.68 or the following equation suggested by Liao and Whitman (NRC, 1985):

$$C_N = \left(\frac{2}{\sigma'_{v0}}\right)^{1/2} \leq 2 \text{ for } \sigma'_{v0} \text{ in units of ksf} \tag{6A.24a}$$

$$C_N = \left(\frac{95.76}{\sigma'_{v0}}\right)^{1/2} \leq 2 \text{ for } \sigma'_{v0} \text{ in units of kPa} \tag{6A.24b}$$

C_L is 0.75 for drill rod shorter than 13 ft (4 m); 0.85 for drill rod lengths of 13 to 20 ft (4 to 6 m); 0.95 for drill rod lengths of 20 to 33 ft (6 to 10 m); 1.0 for drill rod lengths of 33 to 100 ft (10 to 30 m) and > 1.0 for drill rod lengths greater than 100 ft (30 m). C_S is 1.0 for a constant-barrel diameter (that is, with a liner); if a liner is not used, C_S is 1.1 for loose sands and 1.25 to 1.30 for dense sands. C_B is 1.0 for borehole diameters less than 4.5 in. (115 mm) or if the test is conducted through the stem of a hollow-stem auger; 1.05 for a 6 in. (150 mm) diameter borehole; and 1.15 for an 8 in. (200 mm) diameter borehole.

The following SPT procedure is recommended by Seed and colleagues (1985) for use in liquefaction correlations:

1. *Borehole*. 4- to 5-in (102- to 127-mm) diameter rotary borehole with bentonitic drilling mud to stabilize borehole.
2. *Drill bit*. Upward deflection of drilling mud (tricone or baffled drag bit) to prevent erosion of soil below the cutting edge of the bit.

TABLE 6A.7 Estimated Energy Ratios for SPT Procedures (from Seed et al., 1985)

Country	Hammer type	Hammer release	Estimated rod energy, %	Correction factor for 60% rod energy
Japan	Donut	Free-fall	78	1.30
	Donut	Rope and pulley with special throw release	67	1.12
United States	Safety	Rope and pulley	60	1.00
	Donut	Rope and pulley	45	0.75
Argentina	Donut	Rope and pulley	45	0.75
China	Donut	Free-fall	60	1.00
	Donut	Rope and pulley	50	0.83

3. *Sampler*. 2.00-in (50.8-mm) outer diameter, 1.38-in (35.0-mm) constant inner diameter.
4. *Drill rods*. A or AW for depths less than 50 ft (15 m); N or NW for greater depths.
5. *Energy ratio*. 60% of theoretical free-fall energy.
6. *Blowcount rate*. 30 to 40 blows per minute.
7. *Penetration resistance count*. Measured over depth of 6 to 18 in (152 to 457 mm) below the bottom of the borehole.

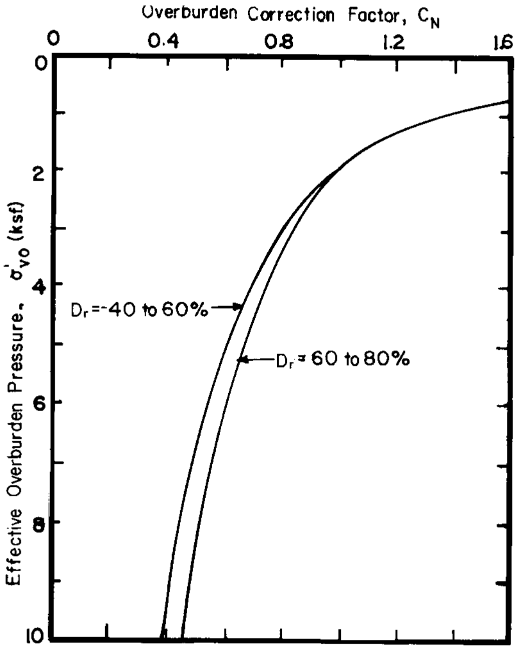


FIGURE 6A.68 Chart for values of overburden correction factor, C_N (from Seed et al., 1985).

If the design earthquake has a magnitude other than 7.5, the value of CSRL obtained from Fig. 6A.67 must be multiplied by a magnitude scaling factor (MSF) to obtain the adjusted CSRL. Values of MSF provided by Seed and Idriss (1982) are commonly used and are summarized in Table 6A.8. A panel of experts convened to discuss liquefaction (NCEER 1997) determined that Seed and Idriss' values are probably too conservative for magnitudes less than 7.5 and probably not sufficiently conservative for magnitudes greater than 7.5. The panel's recommended values are also given in Table 6A.8. It is preferable to use moment magnitude in liquefaction analyses, but surface-wave magnitude or local (Richter) magnitude may also be used for magnitudes less than 7.5 (Youd, 1993).

The previous discussions clearly establish that density plays an important role in the liquefaction potential of a saturated sandy soil. Therefore, the liquefaction potential of a compacted granular fill that may become saturated can be reasonably controlled in many instances by ensuring that the fill soil is highly densified. Reducing the liquefaction potential of an existing saturated granular soil deposit can be more difficult. Near-surface compaction is not a viable alternative in many cases because it is effective only to a maximum depth of about 5 to 10 ft (1.5 to 3.0 m) from the ground surface, and many potentially liquefiable deposits extend to much greater depths. In these instances, several types of deep compaction techniques have been successfully used. Liquefaction potential can also be controlled by shortening the drainage path within the liquefiable deposit, which prevents the excess pore pressure from building to a level where liquefaction can occur. This can be accomplished by the inclusion of granular columns (usually gravel or stone columns) with much higher permeability than the existing deposit.

Effects of Cementation. Many soils contain carbonates, iron oxide, alumina, and organic matter that may precipitate at interparticle contacts and act as cementing agents (Mitchell, 1993). The effect of cementation is to increase the stiffness and strength of the soil while reducing the permeability and liquefaction potential somewhat. For highly cemented coarse-grained soils, the increase in stiffness and strength and the decrease in liquefaction potential can be significant, whereas the reduction in permeability is generally small, so that the overall effect is beneficial for bearing soils. Cemented granular soils that are densified in place by near-surface or dynamic compaction may lose most of or all their natural cementation during the compaction process. The net effect may be either an increase or a decrease in stiffness and strength depending on which mechanism predominates—the increase produced by densification or the decrease resulting from loss of cementation. Therefore, compaction should be used with caution in naturally cemented soils.

Any cementation of borrow materials will also likely be substantially destroyed during the excavation and compaction procedures. However, borrow soils that were cemented before excavation

TABLE 6A.8 Correction Factors for Influence of Earthquake Magnitude on Liquefaction Resistance

	Magnitude Scaling Factor, MSF	
Earthquake magnitude, M	Seed & Idriss (1982)	Recommended by NCEER (1997)
5.5	1.43	2.20 to 2.80
6.0	1.32	1.76 to 2.10
6.5	1.19	1.44 to 1.60
7.0	1.08	1.19 to 1.25
7.5	1.00	1.00
8.0	0.94	0.84
8.5	0.89	0.72

contain minerals that will cement, and therefore it is likely they will develop new cementation with time after compaction. The effect of this postcompaction cementation on the engineering properties of compacted soils is commonly ignored in foundation engineering practice primarily because it is difficult or impractical to determine both the rate at which the cementation will occur and what effect this cementation will have at critical stages of construction and loading of the structure being built. In addition, since cementation has a positive effect on compressibility, strength, and liquefaction potential, and little effect on the permeability, it is conservative to use the as-compacted properties in design and analysis.

In some instances neglecting the postcompaction cementation of fills may be highly uneconomical. For example, in southeastern Florida a common borrow material is carbonate sand that either is a residual soil from the oolitic limestone found near the ground surface or is excavated and crushed from rock deposits of the same material. It is common knowledge among engineers in the area that recementing of the carbonate sand occurs after compaction so that the resulting fill behaves more like a soft rock than a soil. A significant improvement in the postcompaction stress-strain-strength characteristics of the compacted fills often occurs, so that the stiffness and strength of the fills commonly continue to increase during their service life. This improvement is sometimes included in a qualitative way in the design of the foundations for structures constructed on fills composed of these carbonate materials, but more in-depth studies of this phenomenon would surely result in more economical foundation designs. The effects of aging on the engineering properties and behavior of compacted soils are discussed in more detail in Sec. 6A.3.4.3.

6A.3.4.2 Cohesive Soils

The engineering characteristics of compacted cohesive soils are strongly influenced by the as-compacted structure, moisture condition, and density of the soil; by the method used to compact the soil; and by any postcompaction changes in structure, moisture condition, and density that may occur.

Fabric and Structure. Although the terms *fabric* and *structure* are sometimes used interchangeably, it is preferable to define *fabric* as the arrangement of particles, particle groups, and pore spaces within a soil, and to use *structure* to refer to the combined effects of fabric, composition, and interparticle forces. The structure of cohesive soils is extremely complex, and the reader is referred to Mitchell (1993) for a comprehensive discussion. Therefore, only fabric will be discussed here. The fabric of a soil can be considered to consist of the following three components (Mitchell, 1993):

1. *Microfabric.* The microfabric consists of the regular aggregations of particles and the very small pores between them. Typical fabric units are up to a few tens of micrometers across.
2. *Minifabric.* The minifabric contains the aggregations of the microfabric and the interassemblage pores between them. Minifabric units may be a few hundred micrometers in size.
3. *Macrofabric.* The macrofabric may contain cracks, fissures, root holes, laminations, and so on.

For a given cohesive borrow material, the fabric of the as-compacted soil can vary widely depending on the compaction water content and the method of compaction. Conventional descriptions of the fabric of compacted cohesive soils assume that the soil is composed entirely of plateshaped clay particles and are based on the microfabric of the soil, that is, how the individual clay particles are arranged (Lambe, 1958). If most of the particles are oriented parallel to each other, the fabric is said to be *dispersed* or *oriented* (Fig. 6A.69). In clays where most of the particles are arranged perpendicular to each other, the fabric is considered to be *flocculated* or *random*. Compaction dry of optimum or with low effort tends to produce a more flocculated fabric, whereas compaction wet of optimum or with high effort generally yields a more dispersed fabric (Fig. 6A.70).

Although the engineering behavior of pure clay samples compacted in the laboratory can be adequately explained on the basis of the microfabric, it is difficult to justify its use for field compacted

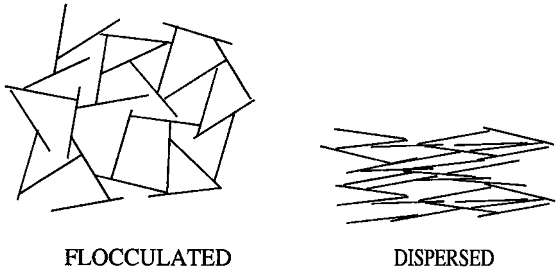


FIGURE 6A.69 Microfabrics for individual particle interaction in pure clays.

cohesive soils for two reasons: (1) Most of the soils contain a variety of particles, including both coarse-grained and fine-grained particles; and (2) the minifabric and macrofabric have a predominant influence in many cases on the engineering characteristics. Excellent reviews of models for compacted soil behavior can be found in Hausmann (1990) and Hilf (1991). A practical theory that considers both microfabric and minifabric is the *clod model*, a slightly modified version of the model proposed by Hodek and Lovell (1979), which built upon the basic concepts developed by others (e.g., Barden and Sides, 1970; Mitchell et al., 1965; Olsen, 1962). In this model, the borrow soil consists of aggregations of particles held together by cohesive (clay) particles and separate granular particles. These aggregations (called *clods*), may contain particles as coarse as gravel-sized, but their behavior during compaction is dominated by the clay particles. The granular particles are generally brittle, but the clods may be either brittle or plastic depending on their moisture condition. The total void space within the soil consists of *intraclod pores* (voids within the clods) and *interclod pores* (void spaces between bulky units, either clods or separate granular particles). The interclod pores are considerably larger than the intraclod pores.

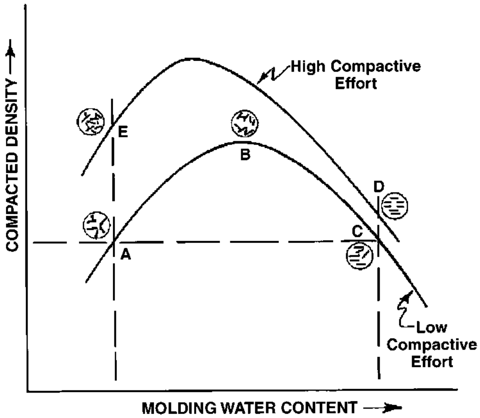


FIGURE 6A.70 Effects of molding water content and compactive effort on the microfabric of pure clays (from Lambe, 1958).

Densification is achieved during compaction by one or more of the following three mechanisms: (1) Pushing the clods and granular particles closer together; (2) breaking brittle clods; and (3) deforming plastic clods. The moisture condition of the clods during compaction is the key factor that determines the density, fabric, and behavior of the as-compacted soil. The granular particles are generally little affected by moisture within the soil. At relatively low compaction water contents, the capillary (matric) suction within the clods is very large, and the clods are dry, strong, and brittle and behave as discrete units. During compaction these clods may be either broken or moved as a unit, but they deform very little, and thus behave essentially as granular particles. Compaction reduces the interclod space but does not affect the intraclod space, which is relatively small because the clods are shrunken (Fig. 6A.71). Thus, the as-compacted fabric of dry cohesive soil essentially consists of discrete, hard clods. Sheepsfoot and tamping foot rollers are generally more effective in densifying dry cohesive soils because the higher contact pressures break down more of the hard clods.

At relatively high water contents, the suction is small, and the clods are plastic and may be remolded into almost any shape. As the clods are deformed during compaction, the interclod space can be nearly eliminated without affecting the intraclod space, which is relatively large because the aggregates are swollen. Depending on the amount of remolding of clods that occurs during compaction, the as-compacted fabric may vary. Owing to their high contact pressures and large induced shearing strains, sheepsfoot rollers tend to remold the clods into a relatively homogeneous fabric. Smooth-drum rollers remold the clods primarily by compressive forces, resulting in a fabric that retains to some extent the identity of the individual clods.

The engineering behavior and properties of compacted cohesive soils often vary with time as changes occur in moisture condition, stress state, and fabric. The as-compacted characteristics of cohesive soils, and how these properties may change as other parameters change, is discussed in the following sections.

Stress-Strain-Strength Behavior. The discussion in this section on the stress-strain-strength be-

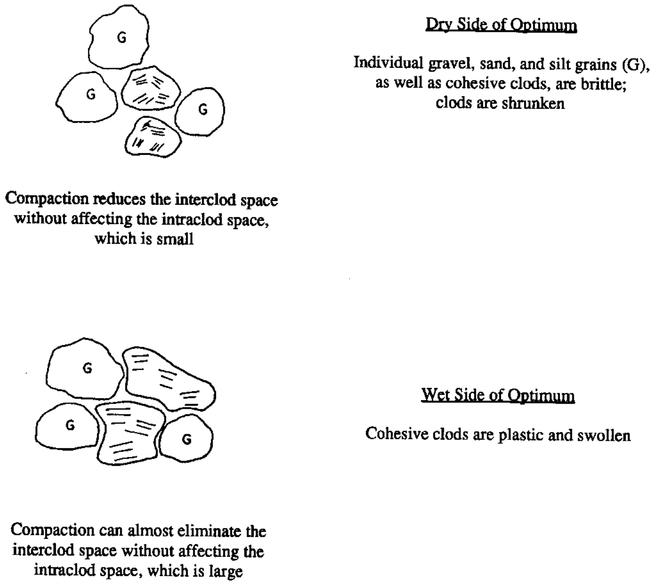


FIGURE 6A.71 Schematics of compaction according to the clod model (modified from Hodek and Lovell, 1979).

havior of compacted cohesive soils is divided into two primary classifications based on the moisture condition of the soil at the time of loading: (1) Soils that remain at their as-compacted water content up to and throughout the loading, and (2) soils that become soaked prior to loading. These two moisture conditions represent the limiting cases usually considered for compacted cohesive soils. Although some compacted cohesive soils dry out during their service life, the drying generally results in a stiffer and stronger soil (unless substantial cracking occurs), so it is usually conservative to ignore the effects of drying and use the compressibility and strength characteristics of the soil at its as-compacted moisture content. However, drying of the soil may result in shrinkage and settlement of any structures founded on that soil. In addition, the loss of moisture increases the potential for subsequent wetting-induced volume changes (swelling or collapse), and this increased potential for volume changes should also be considered (see Sec. 6A.3.4.2).

The term "as-compacted" will be used throughout the next section to refer to a compacted cohesive soil that has remained at its compaction water content up to the time of loading. Thus, *as-compacted* refers to no change in moisture content but does not preclude changes in void ratio, for example, owing to compression under the weight of fill placed on top of the soil prior to the inducement of shearing stresses by a structure constructed on or within the fill. This definition of *as-compacted* is somewhat different than that given by others (for example, Seed et al., 1960), who have defined as-compacted soils as those in which no change in either moisture condition or void ratio occurs prior to or during loading.

As-compacted moisture content

Compactive Prestress The as-compacted compressibility and strength of compacted cohesive soils are influenced by the moisture condition and density of the soil, as well as method and effort of compaction. The cyclic application and removal of mechanical energy during compaction prestresses the soil, and the stress-strain behavior of an as-compacted cohesive soil is similar to that of an overconsolidated, saturated clay. A typical strain-stress plot for one-dimensional compression of an as-compacted cohesive soil is shown in log $\sigma_v - \varepsilon_{VL}$ space in Fig. 6A.72, in which the recompression and virgin compression portions of the loading curve are evident. The vertical stress at which the behavior changes from recompression to virgin compression is called the *compactive prestress* (σ_{vc}) and is analogous to the effective preconsolidation pressure (σ_p') in saturated clays. Casagrande's (1936b) graphical procedure for estimating σ_p' of saturated clays can also be used to approximate σ_{vc} for compacted cohesive soils. σ_{vc} for a particular soil increases with increasing dry density and decreasing water content (Lin and Lovell, 1983). Method of compaction also has a significant influence on σ_{vc}, as indicated in Fig. 6A.73 for specimens of a clayey sand compacted at two water contents to the same dry density using kneading, static, and impact compaction. For both compaction water contents, σ_v for static compaction is greater than for kneading compaction because static compaction is much less efficient in cohesive soils than kneading compaction. Thus, for the same compaction water content, higher pressures are needed in static compaction to produce the same density. σ_v for impact compaction is less than for static compaction at both water contents but is greater than for kneading compaction at w = 10% and less than for kneading compaction at w = 16%. This suggests that impact compaction is less efficient than kneading compaction for compaction below optimum moisture condition (as-compacted $S_r < S_{\text{opt}}$) but more efficient above optimum moisture condition (as-compacted $S_r > S_{\text{opt}}$) The apparent anomalies in the behavior of impact-compacted cohesive soils will be discussed in more detail in a subsequent section.

Influence of Moisture Condition Moisture condition has a dramatic influence on the compressibility and strength of compacted cohesive soils owing to changes in suction. As discussed previously (Sec. 6A.3.4.1), the total suction in an unsaturated soil consists of two components—osmotic suction and matric suction. Values of total, matric, and osmotic suction are shown in Fig. 6A.74 for specimens of a glacial till compacted with modified Proctor effort at various water contents. Matric suction is the largest component of suction in unsaturated soils. Because osmotic suction varies lit-

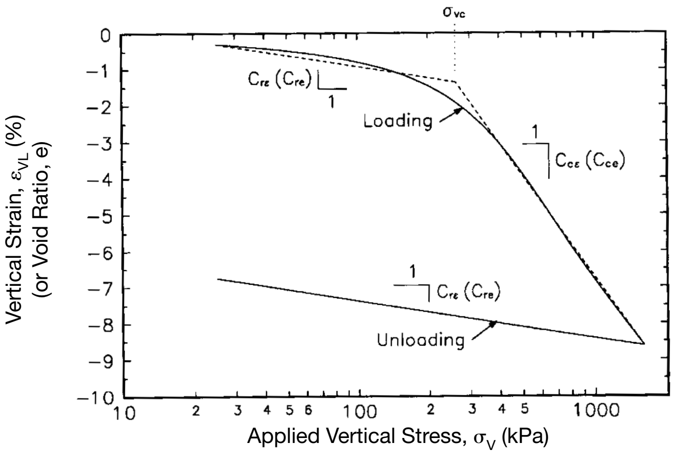


FIGURE 6A.72 Typical strain-stress curve for one-dimensional loading-induced compression of a compacted cohesive soil.

tle for the normal variation in water contents for most compacted soils, changes in suction are primarily related to changes in matric suction, which can be substantial. For example, for the glacial till shown in Fig. 6A.74, the matric suction decreased from about 900 kPa (131 psi) at $w = 11\%$ to about 200 kPa (29 psi) at $w = 17\%$, a 78% reduction in matric suction for a 6% increase in water content.

The effect of moisture condition on the one-dimensional compressibility of a compacted clayey sand is illustrated in Fig. 6A.75. At low water contents the clods are hard and the compacted soil is relatively stiff. Well-compacted, highly plastic, dry soils can be nearly incompressible under stress levels encountered in some projects. For the same method of compaction and as-compacted density, the compressibility of the soil increases as the water content increases, owing to the reduction in suction described previously. Similar trends are seen in Fig. 6A.76 for triaxial stress-strain-strength behavior. An increase in compaction moisture content at constant energy level results in a weaker, more compressible soil. Note also the reduction in brittleness as water content is increased.

The shear strength of an unsaturated soil is commonly formulated in terms of the stress-state variables net normal stress $(\sigma - u_a)$ and matric suction $(u_a - u_w)$. The resulting shear strength equation, which takes the form of a three-dimensional extension of the Mohr-Coulomb failure criterion (see Fig. 6A.77), is as follows (Fredlund et al., 1978):

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \cdot \tan \phi' + (u_a - u_w)_f \cdot \tan \phi^b \tag{6A.25}$$

where τ_{ff} = shear stress on the failure plane at failure (shear strength)

c' = intercept of the extended Mohr-Coulomb failure envelope on the shear stress axis where the net normal stress and the matric suction at failure are equal to zero (commonly referred to as the *effective cohesion intercept*)

$(\sigma_f - u_a)_f$ = net normal stress state on the failure plane at failure

u_{af} = pore air pressure on the failure plane at failure

ϕ' = angle of internal friction associated with the net normal stress variable

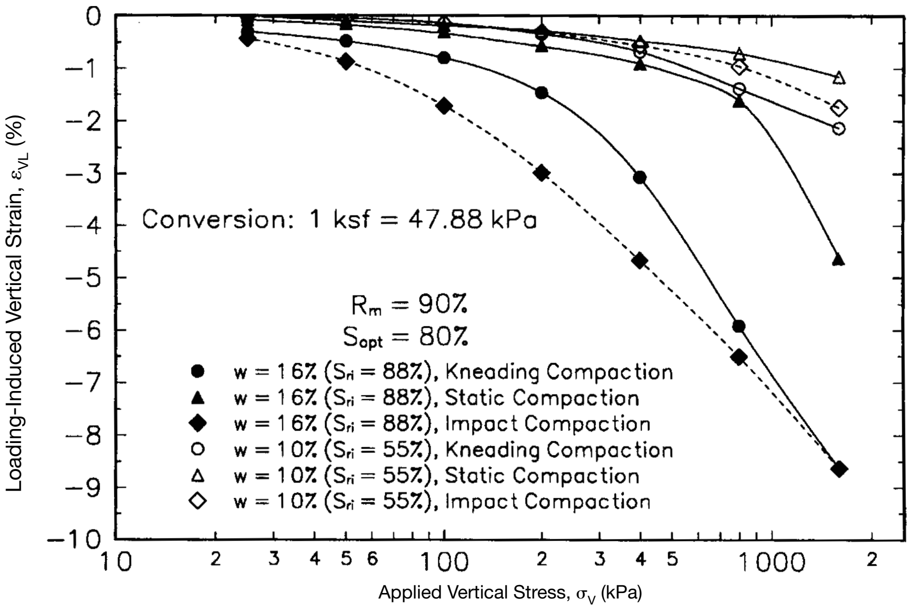


FIGURE 6A.73 Effect of compaction method on one-dimensional compressibility of compacted clayey sand (data from Lawton, 1986).

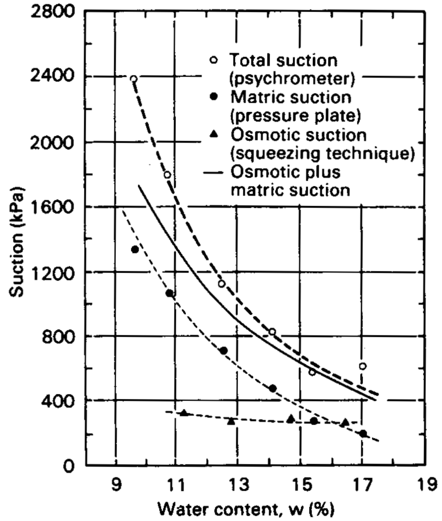


FIGURE 6A.74 Total, matric, and osmotic suctions for glacial till (Fredlund and Rahardjo, 1993, data from Krahn and Fredlund, 1972).

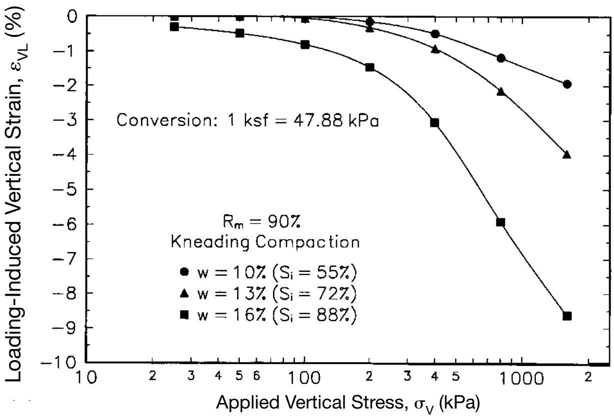


FIGURE 6A.75 Influence of moisture condition on one-dimensional compressibilily of compacted clayey sand (data from Lawton, 1986).

$(u_a - u_w)_f$ = matric suction on the failure plane at failure
ϕ^b = angle indicating the rate of increase in shear strength relative to the matric suction, $(u_a - u_w)_f$

The effect of changes in matric suction on the shear strength of an unsaturated soil is illustrated in Fig. 6A.78(*a*). The projections of the three-dimensional failure envelope on the shear stress-net normal stress plane are shown in Fig. 6A.78(*b*). Thus, the effect of wetting an unsaturated cohesive soil is to reduce its shear strength owing to a reduction in matric suction. This loss of shear strength upon wetting can be considered to result from a reduction in cohesion intercept, where the cohesion intercept is defined as

$$c = c' + (u_a - u_w)_f \cdot \tan \phi^b$$

Values of ϕ' and c' can be measured using saturated soil specimens and conventional triaxial and direct shear testing equipment. Tests on unsaturated specimens using specialized triaxial or direct shear testing apparatuses are needed to obtain values of ϕ^b and the reader is referred to Fredlund and Rahardjo (1993) for details of these tests and equipment. Experimentally determined values of ϕ^b have ranged from about 7 to 26°, with most values in the range of about 13 to 18°.

Effect of Density In general, an increase in density of a compacted cohesive soil for the same method of compaction and water content results in an increase in stiffness and strength. The decrease in as-compacted one-dimensional compressibility of a compacted clayey sand with increasing density at the same water content is shown in Fig. 6A.79. Note also that the compactive prestress increases with increasing density, as would be expected because more energy is required, at the same compaction water content, to achieve a greater density.

The effects of density on the unconsolidated, undrained triaxial compressive strength of two cohesive soils at small and large strains are shown in Figs. 6A.80 and 6A.81. In these graphs, the strength is defined as the maximum stress that the sample sustained to reach the designated value of strain and thus is not necessarily the point on the stress-strain graph corresponding to that value of

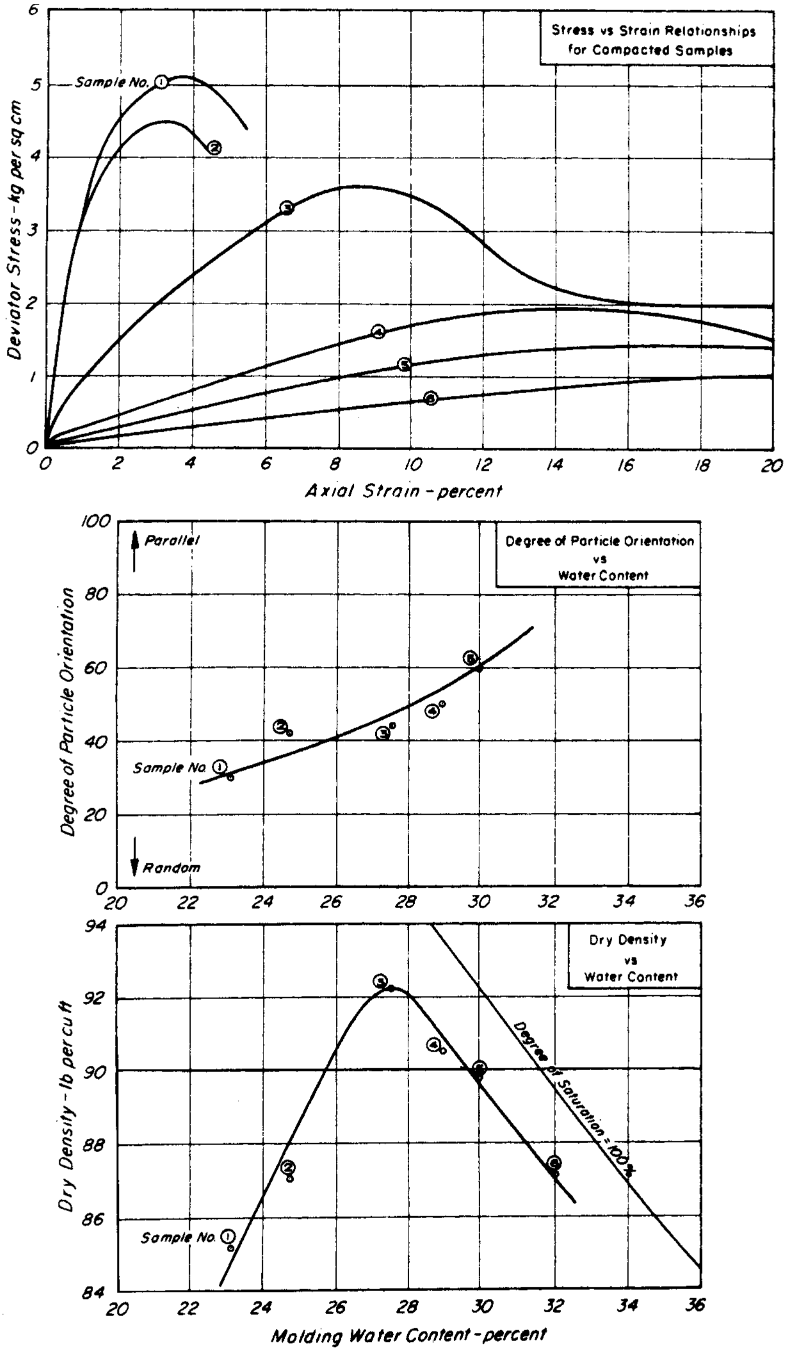


FIGURE 6A.76 Influence of molding water content on structure and triaxial stress–strain relationship for as-compacted samples of Kaolinite (from Seed and Chan, 1959*a*).

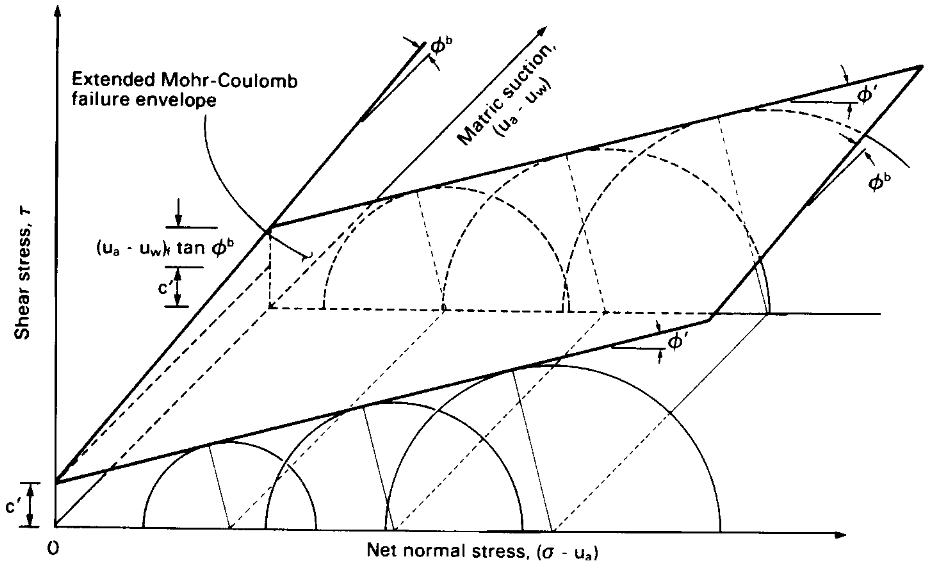


FIGURE 6A.77 Extended Mohr-Coulomb failure envelope for unsaturated soils (from Fredlund and Rahardjo, 1993).

strain. The strength at large strain (20 or 25%) is applicable to situations where large deformations are tolerable, such as a flexible (wet) core of a high earth dam. The strength at small strain (5%) is appropriate for bearing soils where only small deformations are tolerable, such as for a building founded on a compacted fill, or where the soil is brittle (dry) and may crack at small strains.

At large strains, the strength of the sandy clay and the silty clay either increased or remained constant as the dry density increased. At low strains, the strength of both soils increased with increased density at low water contents, but at high water contents the strength peaks at a given density and then decreases with increased density beyond the peak. This loss of strength at high water contents is especially pronounced in the silty clay; for example, at a water content of 16% the strength peaks at about 5.2 kg/cm^2 (10.7 ksf = 510 kPa) for γ_d = 112.5 pcf (17.7 kN/m^3) but drops to 2.0 kg/cm^2 (4.1 ksf = 200 kPa) for γ_d = 114.9 pcf (18.0 kN/m^3), a reduction of 62%. The rapid decrease in strength at low strains near optimum water content can be seen in the left side of Fig. 6A.81 for each of the three compactive energy levels. At the same water content of 16%, the sample compacted with lower energy [γ_d = 112.5 pcf (17.7 kN/m^3)] is dry of optimum ($S_r < S_{opt}$) and therefore stronger than the sample compacted with higher energy [γ_d = 114.9 pcf (18.0 kN/m^3)] that is denser but wet of optimum ($S_r > S_{opt}$). The same trend can be seen in the right side of Fig. 6A.81, where the author has added the line of optimums. Seed and coworkers (1960) attributed this marked decrease in strength near optimum moisture condition to differences in fabric—higher pore water pressures develop in undrained cohesive soils compacted wet of optimum owing to a greater degree of dispersion of the microfabric. An alternative interpretation of this phenomenon can be given in terms of the air/water phases and their pressures. For $S_r < S_{opt}$, the air phase is continuous, and the matric suction likely remains positive even under undrained loading conditions because the air is somewhat compressible, and therefore only small values of positive air pressure are likely to develop compared to the negative water pressures. This suction adds to the strength and stiffness of the soil. As S_r approaches S_{opt}, there is a transition from a continuous to an occluded air phase, which changes the pore water pressures from negative to positive during undrained loading, with a corresponding reduction in strength and stiffness of the soil.

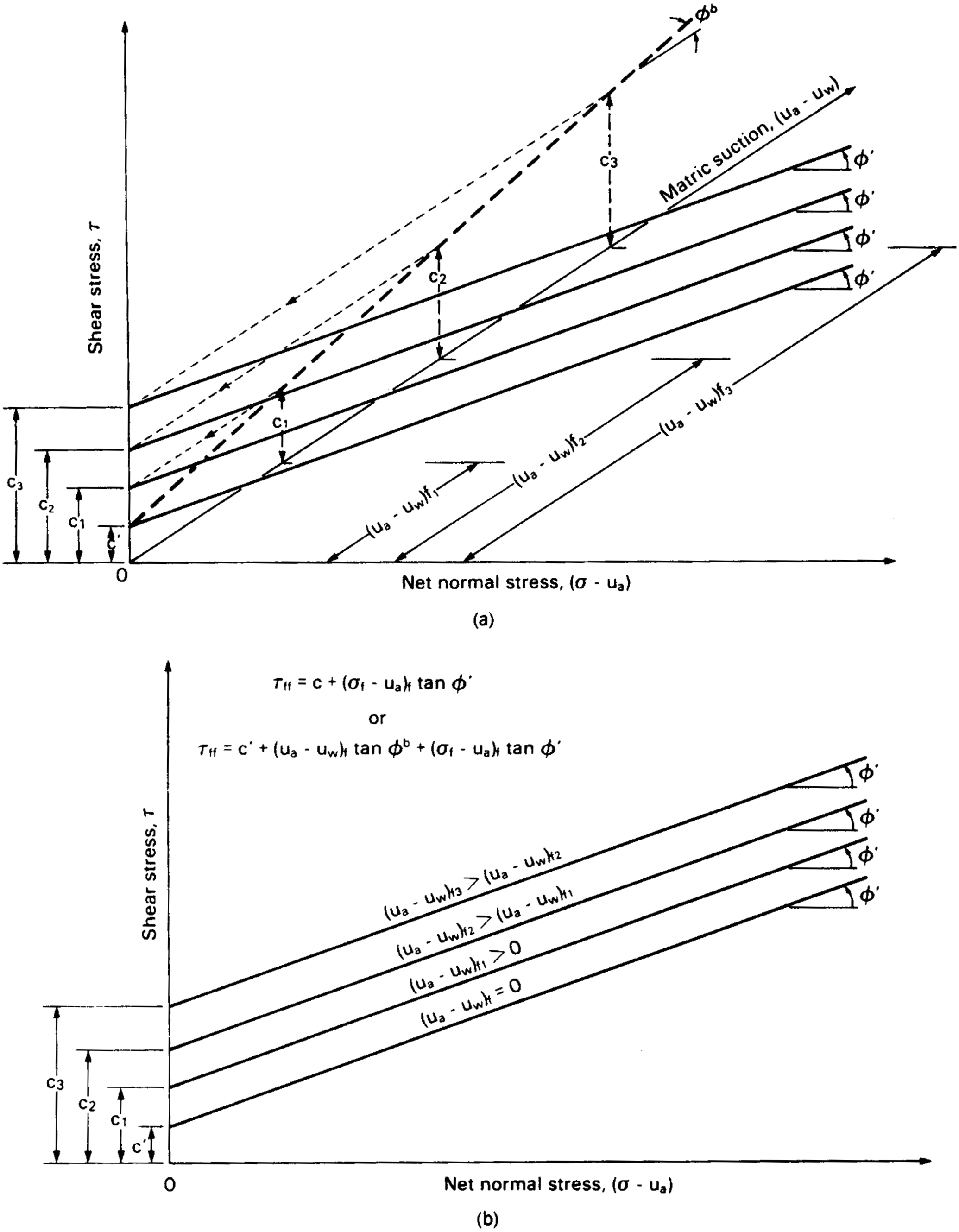


FIGURE 6A.78 Horizontal projection of the failure envelope onto the τ versus $(\sigma - u_a)$ plane, viewed parallel to the $(u_a - u_w)$ axis (from Fredlund and Rahardjo, 1993): (*a*) failure envelope projections onto the τ versus $(\sigma - u_a)$ plane; and (*b*) contour lines of the failure envelope onto the τ versus $(\sigma - u_a)$ plane.

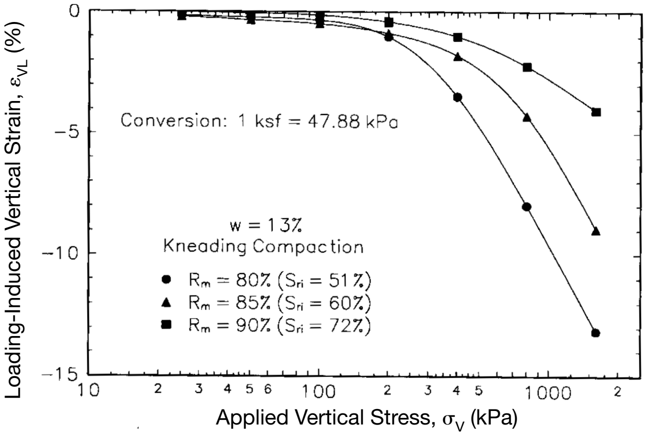


FIGURE 6A.79 Effect of density on as-compacted one-dimensional compressibility of a compacted clayey sand (data from Lawton, 1986).

Method of Compaction Method of compaction can have an important influence on the fabric of as-compacted cohesive soils, which in turn can have a substantial effect on their stress-strain-strength characteristics. For dry cohesive borrow materials, the clods are hard, and densification occurs either by rearrangement of the clods, which retain their basic shape, or by breaking of the clods into smaller (but still hard) pieces. During static compaction (smooth-drum roller), the induced stresses are primarily compressive and of relatively low magnitude compared to other types of rollers of comparable weight, so little breakage of dry clods occurs, and densification results mainly from rearrangement of clods. In contrast, kneading compaction (sheepsfoot and padfoot rollers) induces both shear stresses and compressive stresses in the soil that are of relatively high magnitude owing to the smaller contact areas of the feet, which results in densification from both rearrangement and breakage of particles. Thus, the microfabric of dry clods for both types of compaction is the same because it is essentially unaffected by the compaction, but the minifabric of the compacted soil is different—larger clods and larger intraclod pores for statically compacted soils, and smaller clods and smaller intraclod pores for soils compacted by kneading compaction. If the clods are very hard or the compactors are light, little breakage of clods will occur even for kneading compactors, and the method of compaction used will have little effect on the fabric of the compacted soil. Whether breakage occurs during compaction has little effect on the strength and stiffness of the as-compacted soil for the applied stress levels normally encountered in engineering practice because both characteristics depend on the strength and stiffness of the clods, which are independent of their size. At very high applied stress levels where some breakage might occur, a soil with a fabric consisting of small, hard clods might be stronger and stiffer than one at the same density but with larger clods because of the higher number of interclod contacts, which would result in less force being carried per clod and therefore less breakage of particles. This tendency toward greater stiffness may be offset to some degree by the possibility of more cracks being induced in the smaller clods during compaction, which would make the smaller clods more susceptible to breakage for the same applied force.

Wet cohesive borrow materials are soft, and clods are easily remolded during compaction. Static compaction tends to flatten the clods from the applied compressive stresses, but little change occurs to the microfabric of the clods. The resulting compacted soil mass tends to consist of relatively large interclod pore spaces and clods with flocculated microfabrics that retain to some extent their individual identity. Kneading compaction results in a more homogeneous fabric owing

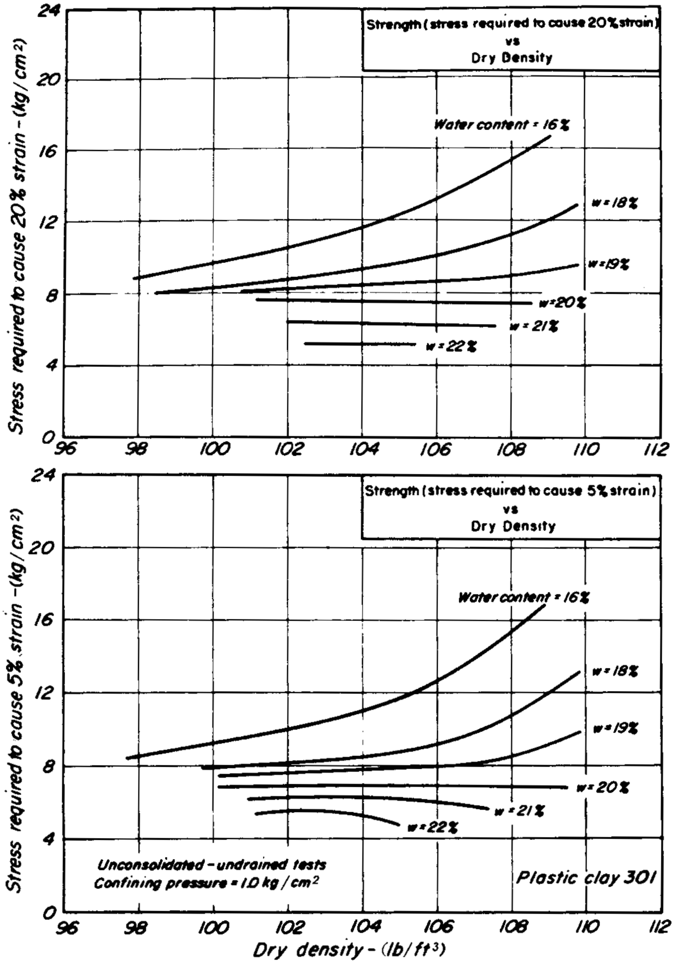


FIGURE 6A.80 Relationships among dry density, water content, and triaxial strength for as-compacted specimens of highly plastic clay prepared by kneading compaction (from Seed et al., 1960).

to the shearing stresses induced during kneading, and the microfabric tends to be more dispersed than for static compaction. For the same moisture content and density, a cohesive soil with a flocculated fabric is less compressible and stronger than the same soil with a dispersed fabric. Thus, static compaction at high water contents produces a stronger and stiffer soil than does kneading compaction. The reasons for this difference in strength and stiffness for differing fabric are quite complex (see Lambe, 1958 for details). Simplified explanations for differences in strength and stiffness for the same soil, water content, and density but different methods of compaction can be given as follows:

Compression. Compression from an applied load tends to align the particles in a parallel array perpendicular to the direction of the applied load. Applying a load to particles that are already parallel merely brings them closer together. Applying the same load to particles that are random-

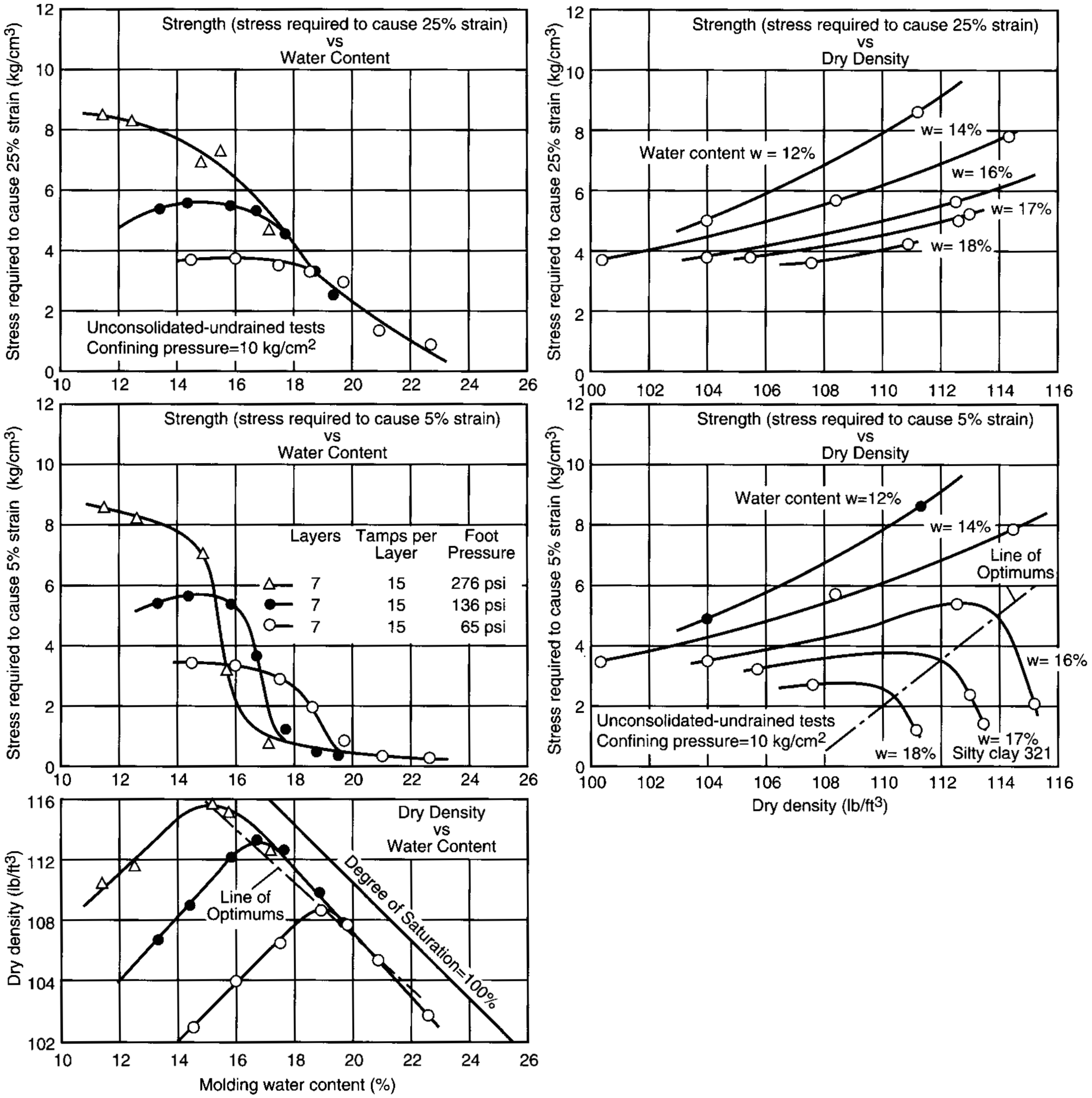


FIGURE 6A.81 Relationships among dry density, water content, and triaxial strength for as-compacted specimens of silty clay prepared by kneading compaction (from Seed et al., 1960).

ly oriented requires that a portion of the load be spent reorienting the nonaligned particles to a more parallel arrangement before any reduction in average void ratio occurs.

Shear Strength. For the same water content and average spacing between clay particles (same void ratio), increasing the angle between particles increases the net attractive force between the particles, and hence increases the shear stress required to cause sliding of the particles relative to one another.

It is unlikely that field compaction would be undertaken on a cohesive borrow material so dry that no remolding would occur during compaction. Therefore, it is reasonable that method of compaction will have some effect on the compressibility and strength of all field-compacted cohesive soils.

Typical results from a series of laboratory one-dimensional compression tests conducted on specimens of a clayey sand compacted directly into the confining rings using static, impact, and kneading methods are shown in Fig. 6A.73. Two sets of results are provided—those for $w = 16\%$ and $R_m = 90\%$ corresponding to a wet of optimum moisture condition (as-compacted degree of saturation, S_{ri}, greater than optimum saturation, S_{opt}), and those for $w = 10\%$ and $R_m = 90\%$ corresponding to a dry of optimum moisture condition ($S_{ri} < S_{opt}$). In both cases, the statically compacted specimens are less compressible than the kneading-compacted specimens. For the dry of optimum specimens at low stresses [$\sigma_v < 200$ kPa (4.2 ksf)], there is little discernible difference in the results. Isograms of loading-induced vertical strain (ε_{VL}) for kneading and statically compacted specimens loaded to $\sigma_v = 400$ kPa (8.4 ksf) are shown in dry density-water content space in Fig. 6A.82. Note that ε_{VL} is greater for kneading compaction than for static compaction for the entire ranges of dry densities and water contents studied. It is interesting that the isograms of ε_{VL} for kneading compaction and S_i greater than about 70% are approximately horizontal, indicating that the compressibility of the specimens at these higher values of S_r is essentially independent of the water content and suggesting that the air phases become occluded in this soil after kneading compaction at about $S_r = 70$ to 80% for $R_m = 80$ to 90%. In contrast, the isograms ε_{VL} for the statically compacted specimens in the same region indicate that compressibility is a function of both water content and density, which suggests that occlusion of the air phases has not occurred.

One-dimensional compression results are also shown in Fig. 6A.73 for specimens compacted directly into oedometer rings using a specially designed impact hammer. At both water contents, the impact-compacted soil was more compressible than the statically compacted soil. However, the comparison between the effects of kneading and impact compaction is not as straightforward—im-

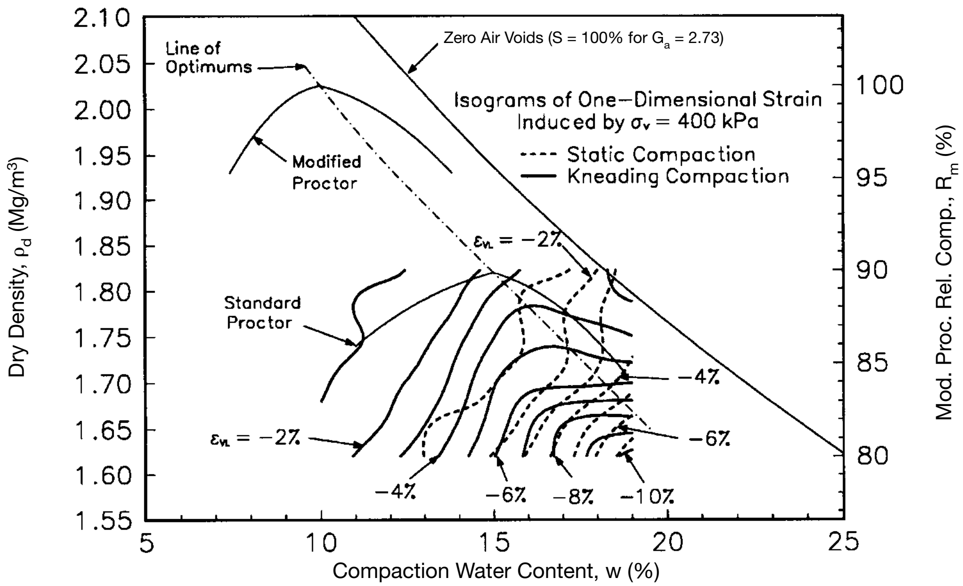


FIGURE 6A.82 Isograms of one-dimensional as-compacted compressibility of clayey sand for specimens prepared by static and kneading compaction (data from Lawton, 1986).

pact compaction produced a less compressible soil at w = 10% but a more compressible soil at w = 16%. To provide additional insight into the differences in compressibility produced by impact compaction compared to static and kneading compaction, isograms of ε_{VL} at σ_v = 400 kPa for the same clayey sand are given in Fig. 6A.83(a) for static and impact compactlon, and Fig. 6A.83(b) for kneading and impact compaction. It can be seen in Fig. 6A.83(a) that static compaction produced a stiffer soil than kneading compaction at all values of ρ_d and w studied. In Fig. 6A.83(b), the equivalent isograms for kneading and impact compaction intersect, with the kneading-compacted soil tending to be more compressible than the impact-compacted soil at low densities and less compressible at high densities. Also note that the isograms for the impact-compacted soil are nearly vertical, suggesting that the as-compacted compressibility of the impact-compacted soil is highly dependent on water content and nearly independent of density for the ranges studied.

Similar results have been found with respect to the effect of method of compaction on the unconsolidated-undrained shear strength of a silty clay (Seed and Chan, 1959a). Values of relative unconsolidated, undrained triaxial strength at 5% strain, based on the strength for kneading compaction, are shown in Fig. 6A.84 for specimens prepared to nominally identical values of (γ_d, w) at various points along the standard Proctor moisture-density curve. The method of compaction had little influence on the strength dry of optimum but had a substantial influence on the strength wet of optimum. Note that the strength of the statically compacted soil was equal to or greater than the strength of the kneading-compacted and impact-compacted soil at all water contents. Impact compaction produced a weaker soil than kneading compaction at low water contents (<18.5%) and a stronger soil at high water contents (>18.5%). For purposes of further analysis, the strength results shown in Fig. 6A.84 for the silty clay are subdivided into three regions:

1. At the low end of the compaction water contents (13 to 14%), the soil was very dry, and the strengths of the soil for both impact and static compaction were about the same.
2. Around optimum water content (15.5 to 17.5%), the specimens were dense (standard Proctor relative compaction, $R_s \cong$ 99 to 100%), and impact-compacted specimens were about 80 to 90% as strong as the kneading-compacted specimens.
3. At the upper end of the compaction water content range for the silty clay (19.5 to 20.5%), the densities are lower ($R_s \cong$ 94 to 96%), and impact compaction produced soil that was about 1.4 to 1.5 times stronger than the soil prepared by kneading compaction.

The reasons for the differences in stress-strain-strength behavior of impact-compacted and kneading-compacted cohesive soils are not completely understood, and further research is needed in this area. A preliminary explanation for these differences can be given as follows (modified from Seed and Chan, 1959a):

1. For cohesive borrow materials that are very dry, no method of compaction produces any appreciable shear deformation, and consequently the method of compaction has no appreciable effect on the fabric and hence the stress-strain-strength characteristics of the as-compacted soil.
2. For the water contents at which cohesive borrow soils are usually compacted, impact compaction causes somewhat less shearing strain during compaction than does kneading compaction. At the same density, the as-compacted fabric for impact compaction tends to have larger but fewer intraclod voids and less dispersion within the clods than for kneading compaction. The larger intraclod voids tend to decrease the strength and increase the compressibility of the impact-compacted soil, but the lesser degree of dispersion tends to increase the strength and decrease the compressibility. Whether the impact-compacted soil is stronger and stiffer than the kneading-compacted soil or vice versa depends on which characteristic dominates the stress-strain-strength behavior—larger intraclod voids or decreased dispersion. At high densities, the fabric of impact-compacted cohesive soil apparently has a degree of dispersion that is close to that of kneading-compacted soil but still has larger intraclod voids. In this case, the larger intraclod voids dominate, resulting in a more compressible soil. At low densities, the increased stiffness produced by the more flocculated fabric of the impact-compacted soil is greater than the reduction in stiffness resulting from the larger intraclod voids.

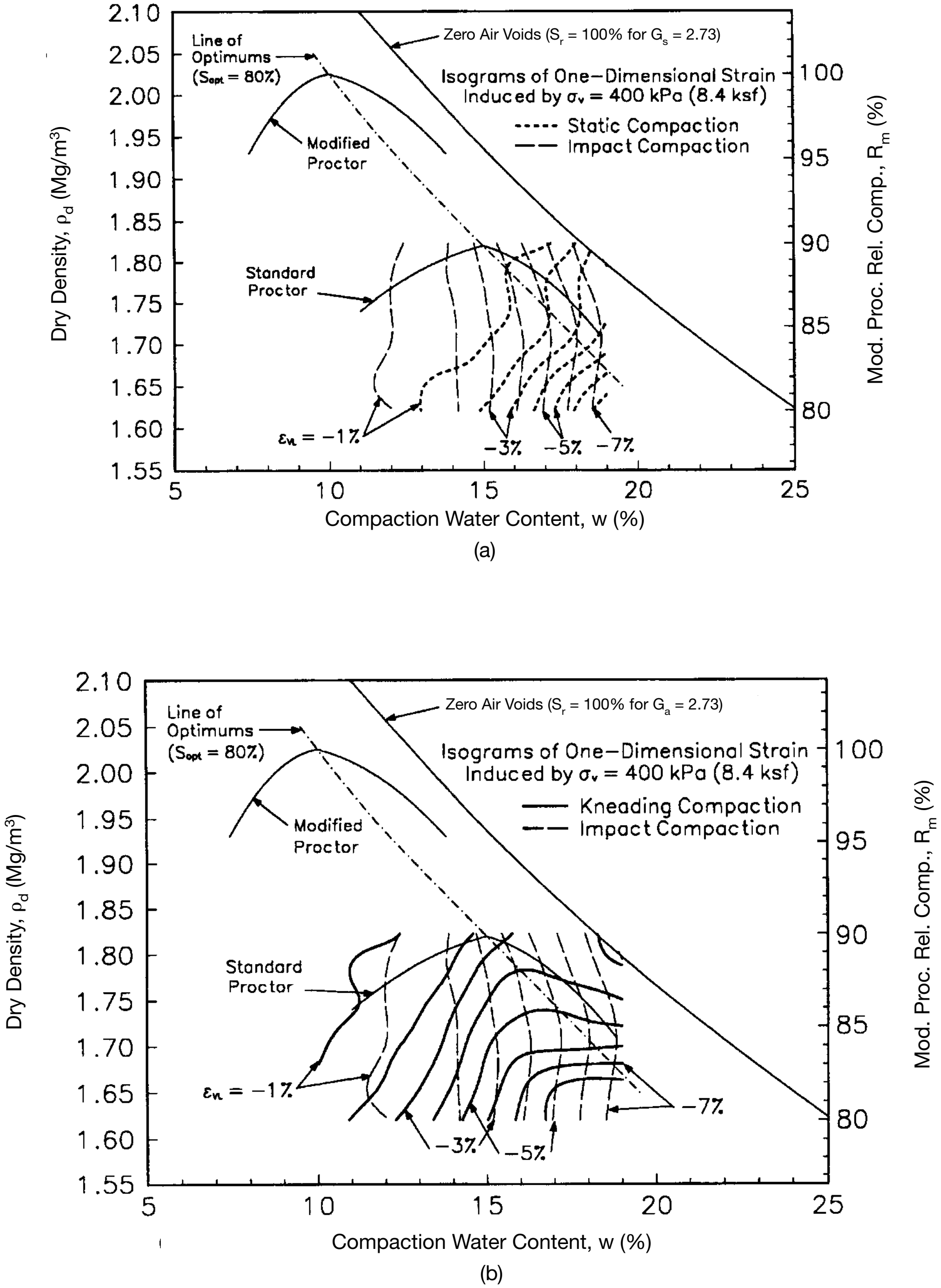


FIGURE 6A.83 Isograms of one-dimensional as-compacted compressibility of clayey sand for specimens prepared by (*a*) static and impact compaction; and (*b*) kneading and impact compaction (data from Lawton, 1986).

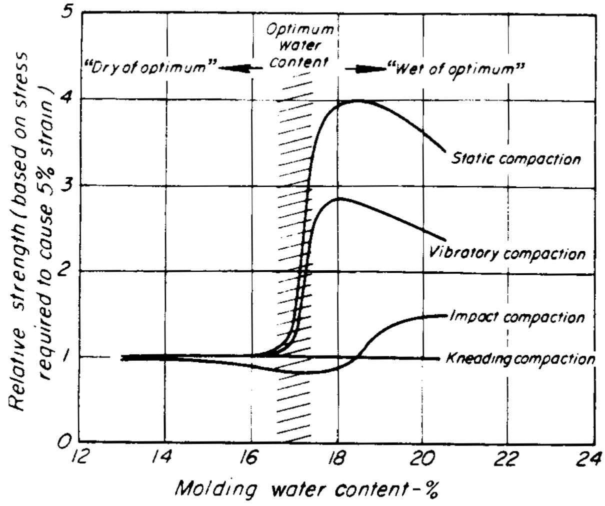


FIGURE 6A.84 Influence of compaction method on the triaxial strength of silty clay (from Seed and Chan, 1959a).

The results presented in this section provide salient evidence to substantiate that the methods and procedures used to prepare laboratory-compacted specimens for testing should simulate as closely possible the methods and procedures used in the field compaction process. It is also apparent that specimens obtained for stress-strain-strength testing by trimming Proctor samples—a routine practice in some testing laboratories—are unlikely to have stress-strain-strength characteristics representative of the soil as compacted in the field using most types of rollers. The following procedure is the best way to ensure that the fabrics of cohesive specimens to be tested in the laboratory are similar to those of the field-compacted soil:

1. Construct several lifts of the borrow soil in a test pad in which the actual compaction procedure is simulated, that is, using the same compaction roller, lift thicknesses, and construction techniques that will be employed in the field compaction process.
2. Obtain samples of the compacted soil from the test pad that are several times larger than the sizes of the specimens to be tested in the laboratory.
3. Carefully trim laboratory specimens to the appropriate sizes for testing from within the interior portions of the larger samples obtained from the field.

Unfortunately, this procedure is not economically feasible for many projects, and thus test specimens are commonly prepared using laboratory compaction procedures.

Time Rate of Settlement As with saturated naturally deposited cohesive soils, the settlement of compacted cohesive soils can be considered to consist of three parts: immediate settlement, primary compression settlement, and secondary compression settlement. *Primary compression* is defined herein for compacted cohesive soils as the loading-induced compression that occurs until either the matric suction reaches a steady-state value under the applied load (unsaturated soil where the air phase is continuous), or the pore water pressure reaches a steady-state value under the applied load (soil is unsaturated but the air phase is occluded or becomes occluded during compression). Primary compression in unsaturated compacted cohesive soils generally occurs much more quickly than does primary consolidation in the same cohesive soil that is saturated (all other factors being the same), owing to differences in water permeability and air permeability. Coefficients of air permeability (k_a) and water permeability (k_w) for a compacted cohesive soil (10% clay) as a function of water content are shown in Fig. 6A.85 (Barden and Pavlakis, 1971). At degrees of saturation less than about S_{opt}, the air phase is continuous, and loading-induced compression expels air from the voids rather than water. For $S_r << S_{opt}$, the portion of the void through which air can flow (the area not occupied by water) is large and primary compression occurs very quickly because of the high value of k_a. As S_r increases, k_a decreases owing to a decrease in the size of the air phase, with a concomitant decrease in the rate of primary compression. For $S_r < S_{opt}$, the air phase becomes occluded, water rather than air is expelled during compression, and the rate of primary compression (consolidation) is controlled by k_w.

The effect of moisture condition on the time rate of one-dimensional compression in a compacted clayey sand is illustrated in Fig. 6A.86. The total compressive strains are shown in Fig. 6A.86(*a*), and the time-dependent strains (after the first reading at 0.25 min) are plotted in Fig. 6A.86(*b*). In these tests, oedometer specimens were compacted at different water contents to the same nominal dry density (R_m = 80%) and then loaded incrementally in a standard one-dimensional compression test. The results shown in Fig. 6A.86 are for the loading increment from 200 to 400 kPa (4.2 to 8.4 ksf). During the previous loading increments, the magnitudes of the strains produced in the specimens were different, so the void ratios of the specimens at the beginning of the 200 to 400 kPa loading increment (e_{200}) varied somewhat, but the influence of these small differences in void ratio on the loading-induced compression was negligible compared to the effect produced by the differences in degree of saturation (S_{r-200}) It can be seen in Fig. 6A.86(*b*) that the magnitude of the time-dependent strains increased substantially with increasing degree of saturation. It is also apparent that the strain-log time plot for the wettest sample ($S_{r-200} > S_{opt}$) has the classical "backward S" shape associated with saturated cohesive soils; in contrast, the plot for the driest sample ($S_{r-200} << S_{opt}$) is essentially linear, suggesting that primary compression was completed prior to the first reading (0.25 min).

Differentiating between primary and secondary compression in compacted cohesive soils is not easy unless tests are conducted in which the pore air and pore water pressures are simultaneously measured during compression, which requires special testing equipment. Tests of this type are rarely conducted in geotechnical engineering practice. Details of equipment and methods that can be used to measure or control u_a and u_w in unsaturated samples are given in Fredlund and Rahardjo (1993). Simultaneous measurement of u_a and u_w is further complicated by the fact that the air phase in the

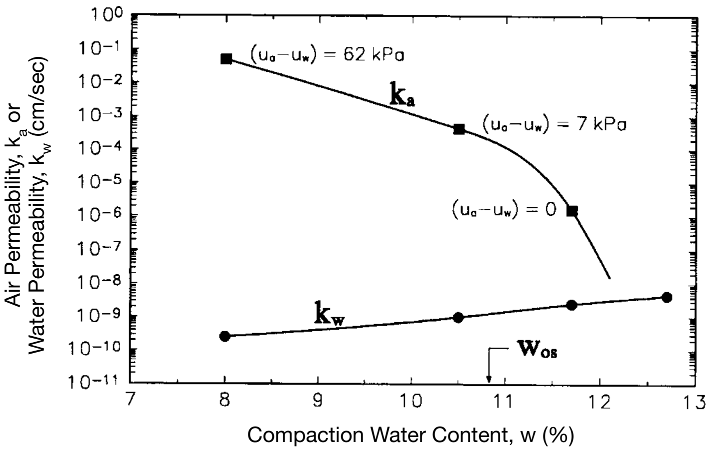
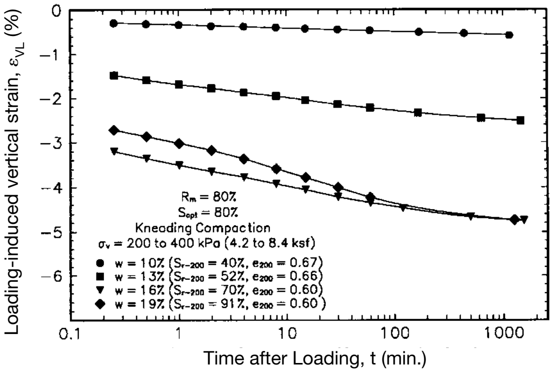
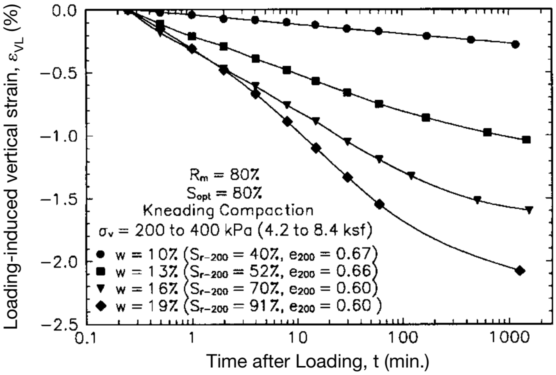


FIGURE 6A.85 Effect of compaction water content on air and water permeability of Westwater soil (from Barden and Pavlakis, 1971).



(a)



(b)

FIGURE 6A.86 Effect of moisture condition on the time rate of one-dimensional loading-induced compression of a compacted clayey sand (data from Lawton, 1986): (*a*) total vertical strain; and (*b*) time-dependent vertical strain.

compacted specimen may change from continuous to occluded during the compression. For example, the specimen with w = 16% in Fig. 6A.86 was loaded incrementally from the as-compacted state ($\sigma_v \cong 0$) to σ_v = 1600 kPa (33.4 ksf), which increased S_r from 63% (17% below S_{opt}) to nearly 100%, and it is likely that the air phase changed from continuous to occluded. [Note that S_r increased from 70 to 81% during the loading increment from 200 to 400 kPa (4.2 to 8.4 ksf) shown in Fig. 6A.86.] Although the air phase was continuous, the pore air pressures were either positive or zero during loading, whereas the pore water pressures were negative [$u_w \cong -80$ kPa (–1.7 ksf)] in the as-compacted state determined from suction measurements. After the air phase became occluded, the pore water pressures were positive (perhaps as much as 800 kPa during the 800–1600-kPa loading increment) or zero, and the pore air pressures in the occluded bubbles could not have been measured but probably had little effect on the behavior during compression. To have measured both the pore air and pore water pressures throughout this test would have required the following setup:

1. A sealed sample with a low air entry porous disk at the top of the sample (for measuring or controlling u_a) and a high air entry porous disk at the bottom of the sample (for measuring u_w).
2. A pressure transducer for measuring positive air pressures up to perhaps 800 kPa (16.7 ksf = 116 psi).
3. A pressure transducer for measuring u_w from about –80 kPa (–1.7 ksf = –12 psi) to perhaps + 800 kPa (16.7 ksf = 116 psi).

Because high air entry ceramics usually have low permeabilities, the response time is generally slower than desired for rapidly changing pore water pressures so that there may be somewhat of a difference between measured and actual pore water pressures at any time.

Owing to the difficulty and expense associated with measuring u_a and u_w during compression tests on compacted cohesive soils, the delineation between primary and secondary compression is usually accomplished by visually examining the strain-log time curves in the same manner as for saturated cohesive soils. The following guidelines are provided for conducting an immediate/primary/secondary compression analysis for a compacted cohesive soil:

1. The values for immediate strain can be calculated from the first readings taken after applying any loading increment (usually 6 or 15 s). From the data in Fig. 6A.86(*a*), the immediate strains for the 200–400-kPa (4.2–8.4-ksf) loading increment would be 0.29, 1.5, 3.2, and 2.7% for w = 10, 13, 16, and 19%, respectively.

2. Primary and secondary (creep) compression occur during the phase called *primary compression*, but there is no method currently available to separate the two components other than to assume that the secondary compression that occurs during primary compression is the same as (or similar to) that measured after primary compression has ended. In typical geotechnical engineering practice, the secondary compression component is not differentiated from the primary compression component, and the two components together are considered to constitute primary compression.

3. When the strain-log time plot has the classical backward S shape [e.g., w = 19% in Fig. 6A.86(*b*)], the time to the end of primary compression is estimated to occur at the point of intersection of a straight line extended forward from the central (steepest) portion of the backward S (some judgment involved) and a best-fit straight line extended backward from the tail of the backward S. Secondary compression is assumed to begin when primary compression ends. The secondary compression index, $C_{\alpha\varepsilon}$ (or $C_{\alpha e}$ from void ratio-log time plot), can be determined from the slope of the best-fit straight line through the tail, and settlement estimated using the following equations:

$$C_{\alpha\varepsilon} = -\frac{d\varepsilon_v}{d(\log t)} \tag{6A.27a}$$

$$C_{\alpha e} = -\frac{de}{d(\log t)} \tag{6A.27b}$$

$$S_s = \int_{z=0}^{z=H} \int_{t=t_p}^{t=t_{\text{design}}} C_{\alpha\varepsilon} \cdot d(\log t) \cdot dz \tag{6A.28a}$$

$$S_s = \int_{z=0}^{z=H} \int_{t=t_p}^{t=t_{\text{design}}} \frac{C_{\alpha e}}{1 + e_p} \cdot d(\log t) \cdot dz \tag{6A.28a}$$

where ε_v = vertical strain
e = void ratio
z = depth from bearing level

H = height of the compacted soil layer
e_p = void ratio at the end of primary compression
t_{design} = design life of the structure

If $C_{\alpha\varepsilon}$ (or $C_{\alpha e}$) is assumed to be independent of time and stress level, and e_p is assumed to be approximately constant with depth, Eq. (6A.28) reduces to the form more commonly seen in the literature:

$$S_s = C_{\alpha\varepsilon} \cdot H_0 \cdot \log \frac{t_{\text{design}}}{t_p} \tag{6A.29a}$$

$$S_s = \frac{C_{\alpha e}}{1 + e_p} \cdot H_p \cdot \log \frac{t_{\text{design}}}{t_p} \tag{6A.29b}$$

The height of the compacted soil layer is shown as H_0 in Eq. (6A.29*a*) because strain is generally plotted as engineering strain ($\Delta H/H_0$). In Eq. (6A.29*b*) e_p and H_p are used because secondary compression is assumed to begin when primary compression ends (t_p). However, for one-dimensional compression the ratio $H/(1 + e)$ is a constant, so e_0 with H_0 can also be used, which is the form more commonly seen in the literature, but the equation as shown is the strictly correct form. If the soil is highly layered or if $C_{\alpha\varepsilon}$ (or $C_{\alpha e}$) varies significantly as a function of stress level, the depth integral in Eq. (6A.28) can be approximated by a summation:

$$S_s = \sum_{i=1}^{i=n} C_{\alpha\varepsilon n} \cdot \log \frac{t_{\text{design}}}{t_p} \cdot \Delta z_n \tag{6A.30a}$$

$$S_s = \sum_{i=1}^{i=n} \frac{C_{\alpha en}}{1 + e_{pn}} \cdot \log \frac{t_{\text{design}}}{t_p} \cdot \Delta z_n \tag{6A.30b}$$

The coefficient of primary compression, c_v, can be calculated from the same techniques used for saturated cohesive soils. A more direct method for extrapolating one-dimensional laboratory results to field time scale is to assume that the time rate of compression is proportional to H_{dp}^2 where H_{dp} is the height of the longest drainage path. For example, ε_{VL} after 1 h in a 1-in (25-mm) high, doubly drained laboratory specimen can be extrapolated to field time scale for a 20-ft (6.1-m) thick, singly drained compacted fill in the following manner:

$$H_{dp\text{–lab}} = 0.5 \text{ in (13 mm)}$$

$$H_{dp\text{–field}} = 20 \text{ ft (6.1 m)} = 240 \text{ in}$$

$$t_{\text{field}} = t_{\text{lab}} \cdot \frac{H^2_{dp\text{–field}}}{H^2_{dp\text{–lab}}} = (1\ h) \cdot \frac{(240 \text{ in})^2}{(0.5 \text{ in})^2} = 230{,}400 \text{ h} = 9600 \text{ day} = 26.3 \text{ yr}$$

Although this method is crude, it is similar in concept to the methods commonly used for saturated cohesive soils and is simple enough to use in routine practice. More sophisticated methods are available for estimating the time rate of compression for unsaturated soils (see Fredlund and Rahardjo, 1993), but these techniques are beyond the scope of this handbook. An alternative method for establishing the effect of H_{dp} on the time rate of compression is to conduct a series of one-dimensional compression tests on specimens compacted at the same water content to the same density but of varying thicknesses, and then comparing the values of strain for the same loading increments and time after loading.

4. If the strain-log time plot can be approximated quite closely with a straight line [e.g., plot for $w = 10\%$ in Fig. 6A.86(*b*)], it can be assumed that primary compression was complete and secondary compression began before the first reading was taken. Thus, the strain at the first reading can be conservatively used as the immediate plus primary compression strain. In this case, the approach for estimating settlement is similar to that used for sands wherein the primary compression settlement occurs so quickly that it cannot be separated from the immediate (distortion) settlement, so both components are included in a single component called *immediate settlement*.

5. For time-rate plots that are neither straight lines nor backward S shapes [e.g., $w = 13$ and 16% in Fig. 6A.86(*b*)], the time to the end of primary compression can be estimated as the point of intersection between a straight line extended forward for the initial (steeper) portion of the plot and a straight line extended backward from the tail (flatter) portion of the plot. Some judgment is commonly required to draw these straight lines, especially the line drawn through the initial portion of the plot, because this part is generally curved.

6. For loading and field conditions that cannot be reasonably approximated by one-dimensional compression tests, more sophisticated laboratory testing techniques—such as the stress path method (Lambe and Whitman, 1979)—are needed to model the field compression in a reasonable way.

Total Settlement It is difficult to differentiate between immediate compression and secondary compression in the laboratory and even more difficult to extrapolate this difference to field time scale. Using a traditional dead-weight consolidometer setup, the first reading commonly taken in a one-dimensional compression test is either 6 s or 15 s. Using the method given in the previous section, these two laboratory times are extrapolated to field scale for a 1.0-in (25-mm) high, doubly drained laboratory specimen and various singly drained fill thicknesses in the following table:

Singly drained field thickness		$H_{dp\text{–field}}$	t_{field} (days)	
ft	m	$H_{dp\text{–lab}}$	$t_{lab} = 6$ s	$t_{lab} = 15$ s
1	0.3	24	0.04	0.1
5	1.5	120	1	2.5
10	3.0	240	4	10
20	6.1	480	16	40
50	15.2	1200	100	250
100	30.5	2400	400	1000
200	61.0	4800	1600	4000

Thus, it is clear that extrapolating laboratory immediate settlement to field time scale depends on the relative values of $H_{dp\text{–lab}}$ and $H_{dp\text{–field}}$ and is not easy to do for large values of $H_{dp\text{–lab}}/H_{dp\text{–field}}$. Because of these difficulties, it is common in geotechnical engineering practice to perform a combined settlement analysis for the immediate and primary compression components, S_{ic}. It should be recognized, though, that usually a significant portion of the settlement that occurs at the first reading is immediate settlement that would occur regardless of when the first reading is taken. Thus, the author recommends taking readings as quickly as possible and assigning the value of strain at the first reading to immediate strain, with the time-dependent strains assumed to occur after the first reading.

The results from one-dimensional compression tests are commonly plotted as either a log $\sigma_v - \varepsilon_v$ or log $\sigma_v - e$ plot, as shown in Fig. 6A.72. In either case, the plot can be approximated by two straight lines (dashed lines in Fig. 6A.72) as is commonly done for saturated clays, with the slopes of the recompression and virgin compression lines designated $C_{r\varepsilon}$ and $C_{c\varepsilon}$, respectively (or C_{re} and C_{ce} if plotted in log $\sigma_v - e$ space). Because the initial portion of the strain-stress plot is curved, some judgment is needed to establish C_{re}. The method preferred by the author is to use the average slope for the rebound curve obtained by unloading the specimen after the loading portion is completed, as shown in Fig. 6A.72. $C_{c\varepsilon}$ is obtained by approximating the virgin compression curve with a straight line. The intersection of the two straight lines representing initial recompression and virgin compression establishes the value of σ_{vc}. With the calculated values for $C_{r\varepsilon}$, $C_{c\varepsilon}$, and C_{vc}, one-dimensional settlement for a shallow foundation can be estimated from the following equation:

$$S_{ic} = \int_{z=0}^{z=\infty} \varepsilon_{VL} dz \tag{6A.31}$$

where $z = 0$ corresponds to the bearing level, with z increasing with depth below the bearing level.

The symbol for settlement in Eq. (6A.31) is designated S_{ic} because the results from one-dimensional compression tests include both the immediate and primary compression components of the settlement. The loading-induced, one-dimensional vertical strain for any point within a compacted soil mass can be expressed by one of the following equations:

For $\sigma_{v1} \le \sigma_{vc}$:

$$\varepsilon_{VL} = C_{r\varepsilon} \cdot \log \frac{\sigma_{v1}}{\sigma_{v0}} \tag{6A.32a}$$

For $\sigma_{v1} > \sigma_{vc}$:

$$\varepsilon_{VL} = C_{r\varepsilon} \cdot \log \frac{\sigma_{vc}}{\sigma_{v0}} + C_{c\varepsilon} \cdot \log \frac{\sigma_{v1}}{\sigma_{vc}} \tag{6A.32b}$$

where σ_{v0} = initial total vertical stress
σ_{v1} = total vertical stress after loading

If true one-dimensional compression is closely approximated—such as a wide foundation bearing on a relatively thin and homogeneous compacted soil layer overlying a thick, incompressible layer—and $\sigma_{v1} \le \sigma_{vc}$ at all depths, Eq. (6A.31) can be used directly to estimate settlement:

$$S_{ic} = \int_{z=0}^{z=H} C_{r\varepsilon} \cdot \log \frac{\sigma_{v1}}{\sigma_{v0}} \cdot dz \tag{6A.33}$$

where H = the height from the bearing level to the bottom of the compacted soil layer
$\sigma_{v0} = \gamma D + z$
$\sigma_{v1} = \sigma_{v0} + q_0$
γ = average unit weight of the compacted soil
D = depth of embedment of the foundation
q_0 = applied uniform, centric bearing stress

Assuming that $C_{r\varepsilon}$ is constant throughout H, $C_{r\varepsilon}$ can be pulled out of the integrand, and the solution for S_{ic} is as follows:

$$S_{ic} = C_{r\varepsilon}\left\{ \frac{\gamma(H+D)+q_0}{\gamma} \cdot \log[(\gamma(H+D)+q_0)] - (H+D)\cdot \log[(\gamma(H+D)] - \frac{\gamma D + q_0}{\gamma} \cdot \log(\gamma D + q_0) + D \cdot \log(\gamma D) \right\} \tag{6A.34}$$

A similar equation can be derived for $\sigma_{v1} > \sigma_{vc}$. In Eqs. (6A.33) and (6A.34), the settlement term S_{ic} is used because $C_{c\varepsilon}$ and $C_{r\varepsilon}$ are commonly calculated based on laboratory data that include both immediate and primary compression strains. If the time rate of settlement is an important consideration, a preferred alternative is to separate the immediate and primary compression components, with values of $C_{c\varepsilon}$ and $C_{r\varepsilon}$ determined separately for immediate compression and primary compression (that is, $C_{c\varepsilon i}$ and $C_{r\varepsilon i}$, for immediate compression and $C_{c\varepsilon c}$, and $C_{r\varepsilon c}$ for primary compression).

Equation (6A.34) is valid if the excavation is backfilled after construction of the foundation, and if no rebound occurs from the time of excavation to the completion of backfilling (which depends on time rate of rebound for the soil versus the elapsed time between excavation and construction of the foundation). If the foundation is not backfilled, or if the soil rebounds completely before the backfill is placed, the initial stress at the bearing level is zero, and S_{ic} should be calculated based on $\sigma_{v0} = \gamma z$. For this case, Eq. (6A.34) simplifies to

$$S_{ic} = C_{r\varepsilon}\left\{\frac{\gamma(H+D)+q_0}{\gamma}\cdot\log[(\gamma(H+D)+q_0)] - H\cdot\log(\gamma H) - \frac{\gamma D + q_0}{\gamma}\cdot\log(\gamma D + q_0)\right\} \tag{6A.35}$$

It is assumed in Eq. (6A.35) that the total weight of the foundation, the portion of the wall or stem below the ground surface, and the backfilled soil above the foundation is about the same as the weight of the excavated soil within the footprint of the foundation.

In many situations the width of the foundation is small in comparison to H, which means that the equation for σ_{v1} is much more complex than that given in Eq. (6A.33), and the $\log(\sigma_{v1}/\sigma_{v0})$ term in Eq. (6A.33) cannot be easily integrated. For these cases, the integration must be approximated with a summation, and the appropriate equations for S_{ic} using the one-dimensional compression indices are

$$S_{ic} = \sum_{i=1}^{i=n} S_{ici} \tag{6A.36a}$$

For $\sigma_{v1} \le \sigma_{vc}$:

$$S_{ici} = C_{r\varepsilon i}\cdot\log\frac{\sigma_{v1i}}{\sigma_{v0i}}\cdot\Delta z_i \tag{6A.36b}$$

For $\sigma_{v1} > \sigma_{vc}$:

$$S_{ici} = \left(C_{r\varepsilon i}\cdot\log\frac{\sigma_{vci}}{\sigma_{v0i}} + C_{c\varepsilon i}\cdot\log\frac{\sigma_{v1i}}{\sigma_{vci}}\right)\cdot\Delta z_i \tag{6A.36c}$$

where n = the number of sublayers into which the bearing soil is subdivided for purposes of settlement analysis

i = subscript added to each parameter indicating the value of that parameter for the ith sublayer

$\sigma_{v1} = \sigma_{v0} + \Delta\sigma_v$

$\Delta\sigma_v$ = increase in total vertical stress caused by the foundation load

Δz_i = height of the ith layer

The greater the value of n, the more closely the integration is approximated. Values of σ_{v0} and σ_{v1} at the center of each sublayer are used as "average" values. $\Delta\sigma_v$ can be estimated for each sublayer using an appropriate stress distribution method. If the soil within the settlement influence zone is relatively homogeneous, the Boussinesq-type equations can be used for foundations bearing on the ground surface. For foundations embedded within a relatively homogeneous soil, Skopek's (1961) and Nishida's (1966) solutions can be used for rectangular and circular foundations, respectively. For layered soils and other conditions, refer to the discussion in Sec. 6A.2.2. Spreadsheet programs are ideally suited for estimating settlement in this manner.

If one-dimensional strain conditions are closely approximated but the soil within the settlement influence zone is layered, the settlement can be calculated by integrating the strains for each layer using Eq. (6A.33) (or a comparable equation for $\sigma_{v1} > \sigma_{vc}$) and the appropriate value of H for each layer. As an alternative, the settlement can be estimated using Eq. (6A.36) and subdividing each layer into a number of sublayers.

Three-Dimensional Settlement It is important to note that for small-width foundations relative to the thickness of the compressible zone (a common situation), the use of Eqs. (6A.32) to (6A.36) would be inconsistent, because one-dimensional compression indices ($C_{c\varepsilon}$ and $C_{r\varepsilon}$) would be used to estimate settlement for conditions of three-dimensional strain. The influence of principal stress ratio on the compression characteristics of a clayey sand compacted at w_{om} to R_m = 85% is shown in Fig. 6A.87. These results were determined from a series of tests conducted in a triaxial cell in which an initial vertical stress of 25 kPa (3.6 psi) was applied to four identically compacted specimens, and the initial radial (horizontal) stress was varied to provide ratios of σ_r/σ_v = ⅓, ½, ⅔, and 1. The vertical strain induced in each sample by the application of the initial stresses was very small and could not be accurately measured, and thus are shown as zero in Fig. 6A.87. The stresses on the specimens were then increased incrementally, while maintaining the appropriate value of lateral stress coefficient (σ_r/σ_v), up to the controlling capacities of the equipment (a maximum cell pressure of 690 kPa = 100 psi or a maximum vertical stress of 1600 kPa = 232 psi). Therefore, the maximum value of σ_v for each test varied.

The results from these triaxial compression tests are compared in Fig. 6A.87 with those from one-dimensional compression tests conducted in an oedometer on specimens compacted directly into the oedometer ring at the same water content using the same compaction method to the same relative compaction. At low stresses the strains are small and the effect of σ_r/σ_v is small, and it is difficult to determine a consistent relationship. At the higher stresses, it is clear that the stress ratio

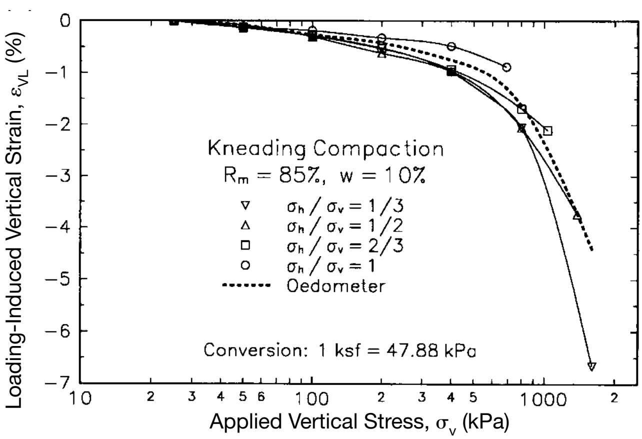


FIGURE 6A.87 Influence of stress ratio on vertical compressibility of compacted clayey sand (data from Lawton 1986).

has a significant influence on the loading-induced strain, with ε_{VL} increasing with decreasing σ_r/σ_v. Thus, the lower the confining pressure, the greater the induced vertical strain. Comparing the position of the curve for the oedometer test with those from the triaxial test indicates that the value of σ_r/σ_v in the oedometer sample was consistently between ⅔ and 1 for stresses below the compactive prestress ($\sigma_{vc} \cong 600$ kPa = 87 psi) but decreased at stresses above σ_{vc} to a value of about 0.5 at $\sigma_v =$ 1600 kPa (232 psi).

These results suggest that the results of oedometer tests can be used to estimate three-dimensional settlements for dry, stiff compacted cohesive soils at low applied stresses but that more sophisticated techniques are needed for higher stresses or wet, soft soils. Some methods that might be used include the following:

1. The Skempton and Bjerrum (1957) method for correcting one-dimensional settlement of saturated cohesive soils for three-dimensional conditions might be used for soils with $S_r > S_{\text{opt}}$ because the induced pore pressures in the compacted soil may be similar to those for saturated soil. However, as noted by Lambe and Whitman (1979), Skempton and Bjerrum's method assumes a discontinuous—and therefore incorrect—stress path and tends to underestimate settlement.
2. The stress-path method (Lambe, 1967) could be employed and would involve conducting stress-path triaxial tests on a number of samples that duplicate the expected stress paths in the soil at various depths beneath the foundation. The major problem with using the stress-path method is that the induced changes in vertical and horizontal stress must be predicted, and the predicted values may vary substantially from the actual changes in stress.
3. A series of plate-load tests using at least three sizes of plates could be conducted in the field-compacted soil at the expected bearing level, and the results could be extrapolated to full field scale.
4. A finite element settlement analysis could be performed (see Sec. 6A.2.3 for recommended procedures).

Ultimate Bearing Capacity Analyses Any appropriate method for estimate ultimate bearing capacity can be used (see Sec. 2B). It is important that the strength parameters (friction angle and cohesion intercept) be consistent with the desired analysis, that is, undrained strength parameters for short-term bearing capacity and drained strength parameters for long-term bearing capacity.

Final Comments One might conclude from the previous discussions that it is best to compact cohesive bearing soils at low water contents to maximize strength and minimize loading-induced settlement. Although dry cohesive soils are stronger and stiffer than wet (all other factors being the same), the potential for wetting-induced and drying-induced volume changes must also be considered, as will be discussed. Thus, a settlement analysis for a foundation bearing on a compacted cohesive soil must consider the potential for settlement or heave from loading, wetting, and drying throughout the design life of the structure.

Soaked

In some instances it is highly likely that compacted cohesive soils will become wetter at some time during the design life of the structure founded on these soils. Wetting may occur from obvious sources such as precipitation (rainfall and snowmelt) and changes in regional or local groundwater conditions (rising of the ground-water table, diversion of runoff, and so on). Wetting may also result from less obvious sources, including broken or leaking water pipes, landscape irrigation, and clogged drainage systems. Therefore, it is nearly impossible to ensure that wetting will not occur in cohesive bearing soils, which prompts many engineers to estimate settlement and ultimate bearing capacity based on soaked compressibility and strength characteristics of the soil.

Compressibility Wetting of a compacted cohesive soil generally results in some volume change (swelling or collapse, as will be discussed), which must be considered in the overall settlement/heave analysis, and which also changes the compressibility and strength of the soil. The changes in compressibility and strength depend on the prewetting moisture condition and the magnitude and sign of the wetting-induced volume change (if any). For example, a soil that swells when wetted becomes more compressible owing to both the reduction in suction and the increase in void ratio. The significant increase in one-dimensional compressibility caused by wetting a high-density (R_m = 90%), dry (S_r = 55%) compacted clayey sand is illustrated in Fig. 6A.88. After inundation at a very low applied stress (σ_v = 1 kPa = 0.02 ksf), the specimen swelled 14.4%, which changed the void ratio from 0.51 in the as-compacted condition (e_i) to 0.66 at the beginning of the loading process (e_L). In addition, the matric suction in the specimen was reduced by soaking from about 800 kPa (16.7 ksf) to essentially zero. At an applied stress of 1600 kPa (33.4 ksf), the soaked sample compressed 14.5%—about 7 times as much as the as-compacted sample (2.1%). Soaking the specimen eliminated the prestressing induced during compaction, as indicated by the essentially linear strain-log stress relationship for the soaked sample.

Cohesive soils compacted to high degrees of saturation will undergo little or no wetting-induced volume change and their compressibility will not be appreciably affected by soaking. This is illustrated in Fig. 6A.89 for a compacted clayey sand with $S_r \cong S_{opt}$ that swelled only 0.4% after soaking. The compressibility of this soaked specimen did not differ significantly from that of the as-compacted specimen. In addition, the soaking had no noticeable effect on the compactive prestress.

Soaking can also induce a decrease in void ratio (collapse) in the soil. This decrease in void ratio tends to reduce the compressibility, but in most instances this reduction is more than offset by the increase in compressibility produced by the reduction in matric suction.

Strength The soaked strength of compacted cohesive soils depends on many factors, including the following:

1. Fabric (microfabric, minifabric, and macrofabric)
2. Method of compaction
3. Compaction water content
4. Soaked water content and degree of saturation
5. Volume change induced by the soaking

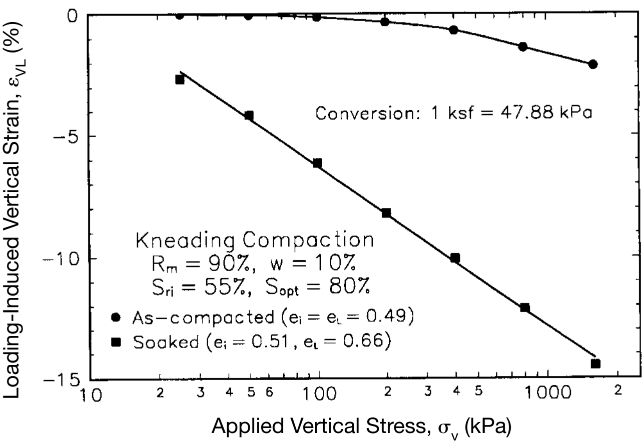


FIGURE 6A.88 Effect of soaking on one-dimensional compressibility of dry compacted clayey sand (data from Lawton, 1986).

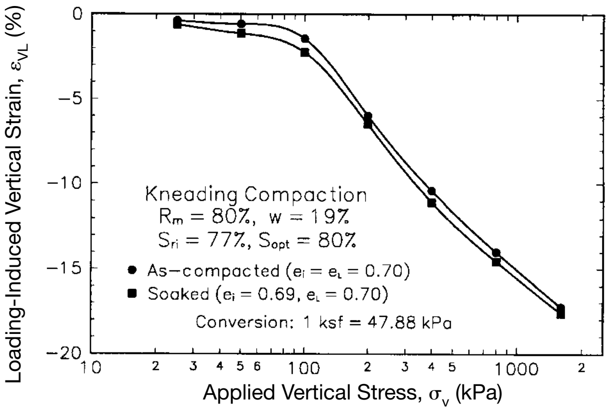


FIGURE 6A.89 Influence of soaking on one-dimensional compressibility of wet compacted clayey sand (data from Lawton, 1986).

6. Dry density or void ratio after soaking
7. Magnitude and sign of the pore water pressures that develop during loading

Most of these factors are interdependent. Because these interrelationships are quite complicated, only general discussions will be given below. Further details can be found in Seed and Chan (1959*b*) and Yi (1991).

Effect of fabric The fabric of the soaked soil at the initiation of loading can have a substantial effect on the strength. It is generally assumed that the fabric of the soaked soil is essentially the same as that of the as-compacted soil, but some changes in fabric may occur because of soaking, and further study is needed of this issue. There is no doubt, however, that the soaked fabric depends to some extent on the as-compacted fabric.

The fabric affects the undrained strength of soaked soil primarily in terms of the developed pore pressures. The results from consolidated-undrained triaxial compression tests on two specimens of compacted silty clay are given in Fig. 6A.90. The two specimens were prepared by kneading compaction to different as-compacted states—one dry of optimum and one wet of optimum, but after soaking the specimens had the same composition (γ_d, w). The dry-of-optimum specimen had an initially flocculated microfabric and developed a high strength at low strain (≅ 2%) and thereafter showed only a small increase in strength with additional deformation. The wet-of-optimum specimen had an initially dispersed microfabric, and at strains less than about 12.5%, appreciably larger pore water pressures were developed than in the dry-of-optimum specimen, with the result that the wet-of-optimum specimen was weaker. These differences in developed pore water pressures result from the different shapes of the pore spaces in the microfabric. The magnitude of developed pore water pressure in any void space is inversely related to the permeability of that void space, and the permeability of a void space depends to a great extent on its smallest dimension. Thus, for two void spaces with the same volume and identical loading conditions, greater pore water pressures will be induced in a void space with a long, narrow shape (dispersed microfabric) than in a more equidimensionally shaped void space (flocculated microfabric). At strains greater than 12.5%, the differences in pore water pressures were small and the strengths of the two specimens were nearly the same, suggesting that the microfabric in the zones of the failure planes for the two specimens were essentially the same (dispersed). The effective principal stress ratios for both specimens were nearly

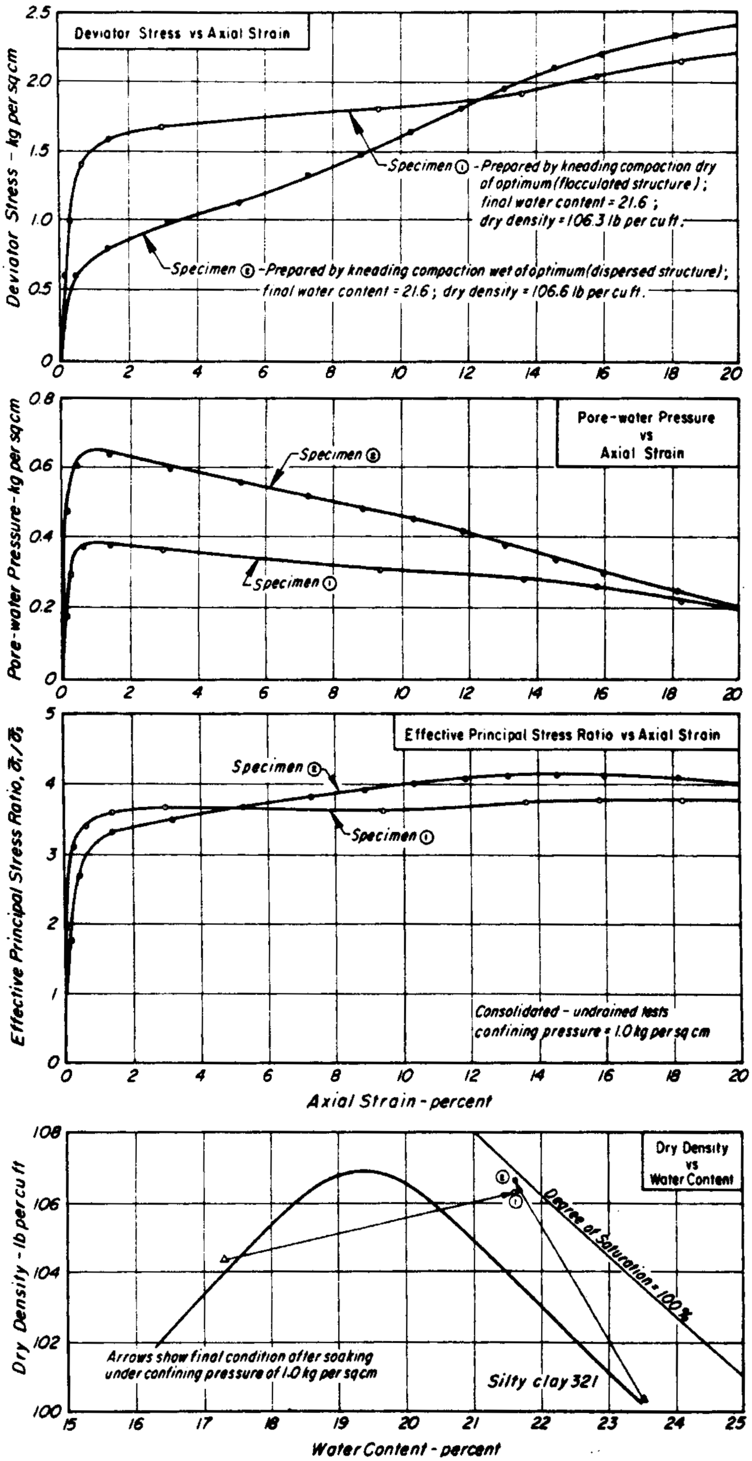


FIGURE 6A.90 Effect of microfabric on pore water pressure and strength from triaxial consolidated-undrained tests on a silty clay (from Seed et al., 1960).

the same at all strains, suggesting that the effective stress strengths were similar for the two specimens so that the differences in total stress strengths (load-carrying capacity) were primarily the result of differences in developed pore water pressures.

Yi (1991) conducted a similar study on soils that collapsed when wetted. The results of consolidated-undrained simple shear tests conducted on specimens of a clayey sand that were compacted to two different as-compacted states and then soaked to the same γ_d and w are shown in Fig. 6A.91. The dry-of-optimum specimens collapsed on wetting, whereas the wet-of-optimum specimens underwent no volume change. The results clearly show that at all three vertical confining pressures, the dry-of-optimum specimens developed higher pore water pressures and were weaker than the wet-of-optimum specimens. Similar results were obtained for three other soils. These results cannot be explained in terms of flocculated versus dispersed microfabric and are opposite to those found by Seed and Chan (1959*b*) for the silty clay discussed above. It is not known if these differences in results are related to differences in types of soils, type of wetting-induced volume change (swelling versus collapse), or perhaps differences in sample preparation (although kneading compaction was used in both studies). Scanning electron micrographs of the dry-of-optimum clayey sand specimens showed that the effect of collapse on fabric was to reduce the size of the interclod pores without any apparent change to the microfabric. Unfortunately, no micrographs of the wet-of-optimum specimens were taken, so the characteristics of the fabric for these specimens are not known. Yi assumed both that the soaked fabrics of the wet- and dry-of-optimum specimens were similar, with the wet-of-optimum specimens containing larger interclod pores and therefore denser microfabric, and that the strengths of these specimens were governed by the density of their microfabric. The author believes that the soaked fabrics of the wet- and dry-of-optimum specimens were probably not as similar as assumed by Yi, so that the explanation for the differences in strength is probably quite complex and cannot be determined from the available information.

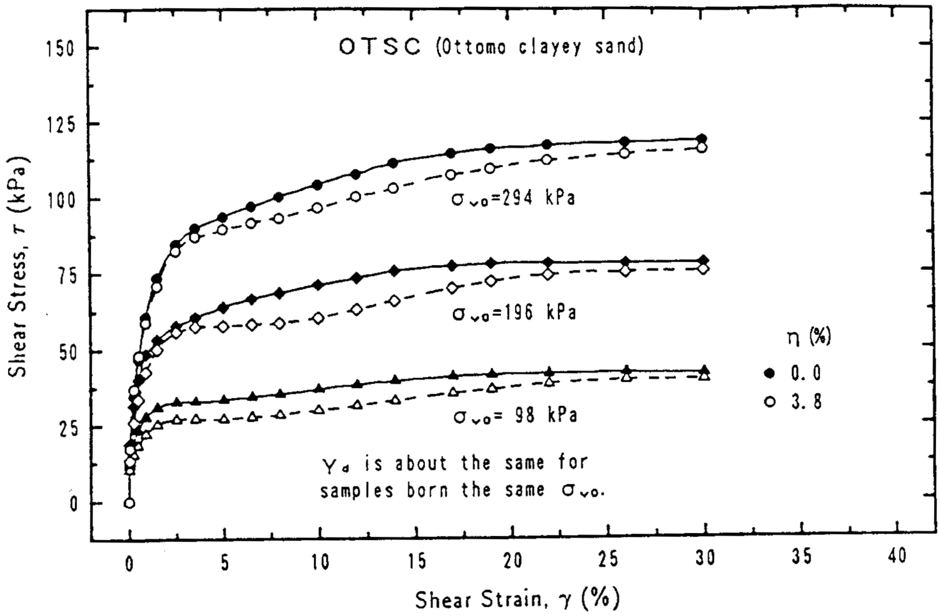
Clearly, fabric can play an important role in the soaked strength of compacted cohesive soils. However, with our present knowledge and the uncertainty regarding the relative importance of microfabric and minifabric, additional research is needed before a reasonable explanation can be given for the influence of fabric on soaked strength.

Influence of wetting-induced volume change The volume changes that occur from soaking a compacted cohesive soil can be summarized as (*a*) no change in volume, (h) an increase in volume (swelling), and (*c*) a decrease in volume (collapse). Compacted cohesive soils that undergo no volume change after soaking are a special case and may be of two types—nonexpansive soils that are soaked under a low surcharge pressure and expansive soils soaked under a surcharge pressure sufficiently high to prevent swelling.

The undrained strength of a compacted cohesive soil that has been soaked depends on the characteristics of the soil in the soaked condition—dry density, water content, and fabric. These soaked characteristics depend on the as-compacted characteristics of the soil (γ_d, w, and fabric) as well as the stress state of the soil at the time of soaking, as discussed previously.

The influence of soaking at constant volume on the unconsolidated-undrained (UU) triaxial compressive strength at low strain (ε_{VL} = 5%) of a silty clay prepared by kneading compaction is shown in Fig. 6A.92. At all densities and water contents, the soaked soil is weaker than the comparable as-compacted soil, with the differences decreasing with increasing compaction water content.

Isograms of triaxial *UU* triaxial compressive strength at low strain are shown in Fig. 6A.93 for kneading-compacted specimens of sandy clay soaked under two different surcharge loads. Under the low surcharge pressure (1 psi = 7 kPa), the soaked strength depended significantly on the compaction water content owing to the differences in swelling that occurred upon soaking. At lower water contents the soil swelled more than at higher water contents, and thus for the same as-compacted dry density, the dry density after soaking was less and the water content was more for the initially dryer soil. Thus, for the same dry density, the soaked strength increased significantly with increasing compaction water content. Note that this relationship is the opposite of that for as-compacted strength, which decreases with increasing compaction water content at the same dry density (see Fig. 6A.92).



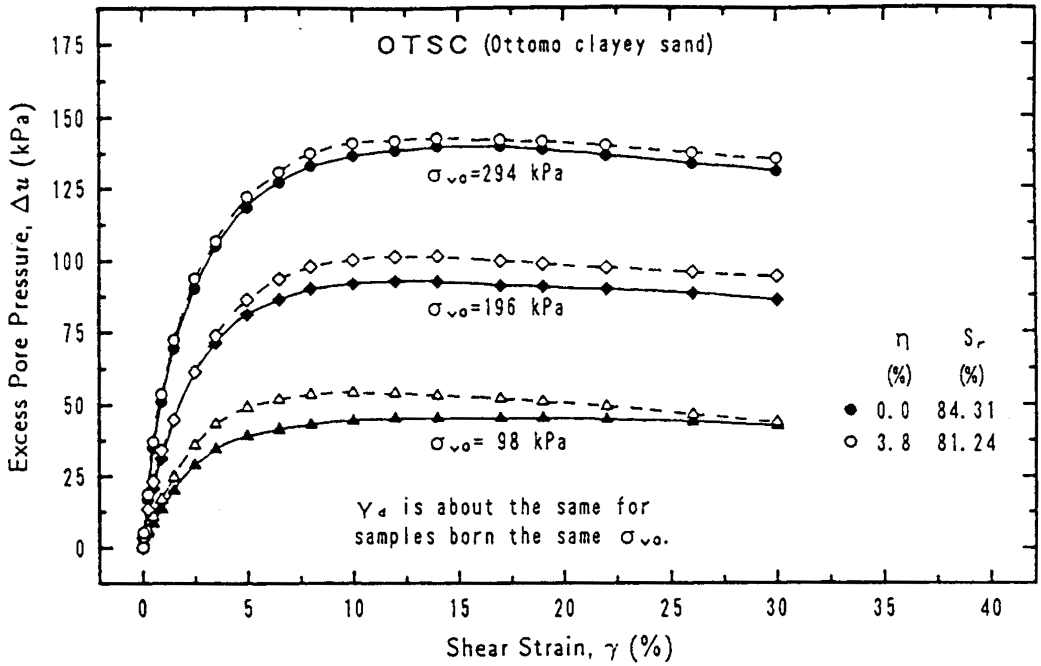


FIGURE 6A.91 Results from consolidated-undrained simple shear tests on specimens of a clayey sand which underwent different amounts of collapse (from Yi, 1991).

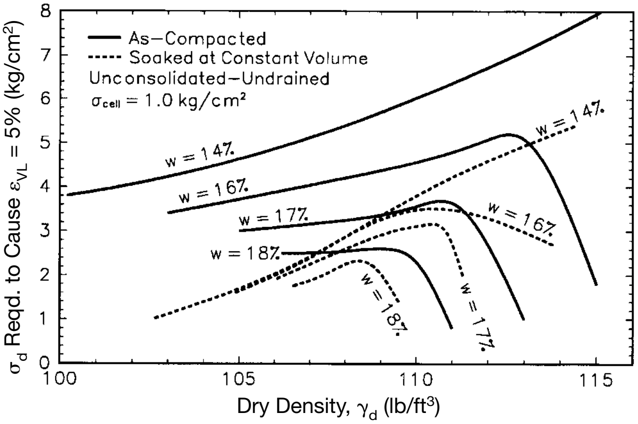


FIGURE 6A.92 Influence of soaking on the triaxial compressive strength of kneading-compacted silty clay (data from Seed and Chan, 1959*a,b*).

The high-surcharge specimens underwent little swelling, and the corresponding soaked-strength isograms are nearly horizontal, indicating that the strength depended primarily on as-compacted density and was nearly independent of compaction water content. Dry of optimum ($S_r < S_{opt}$) the strength at constant as-compacted dry density increases slightly with increasing compaction water content owing to slightly more swelling at the lower compaction water contents. Note, though, that the strength decreases slightly with increasing water content wet of optimum ($S_r > S_{opt}$) because the microfabric became increasingly more dispersed but no swelling occurred.

There is no information available with regard to the influence of the magnitude of collapse on the soaked strength of compacted cohesive soils. For a given composition and fabric, increasing the

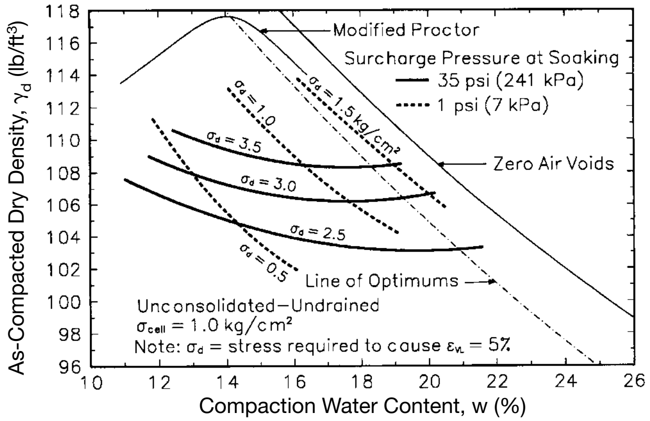


FIGURE 6A.93 Influence of surcharge pressure at soaking on the triaxial compressive strength at low strain for kneading-compacted sandy clay (data from Seed and Chan, 1959a,b).

applied stresses causes greater wetting-induced collapse and thus increases the density of the soil, which likely increases the soaked strength of the soil. However, differences in magnitude of collapse may produce variations in fabric, which in turn may affect the soaked strength of the soil.

Method of compaction For a cohesive soil compacted to the same composition, varying the method of compaction may alter the soaked strength in two ways: (*a*) The as-compacted fabric (and hence the soaked fabric) may be different, and (*b*) the wetting-induced volume change may vary. As noted previously, flocculated microfabrics are stronger than dispersed microfabrics, so, other than very dry of optimum where compaction method has little effect on fabric, static compaction would be expected to produce a stronger soil than kneading compaction. However, the wetting-induced volume change also affects the soaked strength. Static compaction induces a higher swelling potential or a higher collapse potential in the soil than does kneading compaction, depending on the applied stress level (see discussion below). As discussed above, greater swelling reduces the soaked strength of the soil while greater collapse likely increases it. Thus, for conditions where swelling is likely to occur (highly expansive soils and low stresses), static compaction may result in greater or lesser soaked strength than kneading compaction depending on which mechanism controls. For nonexpansive soils or high surcharge pressures, either no volume change or collapse is likely to occur from wetting, and static compaction will probably produce a stronger soaked soil.

Wetting-Induced Volume Changes. *Swelling,* which is an increase in total volume of a soil upon wetting, and *collapse*, which is a decrease in total volume upon wetting, are considered separate and distinct phenomena by many engineers and researchers. Recent research has shown, however, that any unsaturated cohesive soil may either collapse, swell, or not change volume when wetted, depending on the condition of the soil at the time of wetting. Thus, swelling and collapse in unsaturated cohesive soils are associated aspects of the general phenomenon of wetting-induced volume changes.

The magnitude and sign of the volume change that occurs when an unsaturated cohesive soil is wetted are primarily related to changes in matric suction. As suction decreases during wetting, the change in pore water pressure is positive (the pore water pressure becomes less negative), which reduces both the effective stresses acting on the soil and the stiffness of the soil structure. The decrease in effective stress results in an increase in volume (swelling), whereas the reduction in stiffness produces a decrease in volume (collapse). The net change in volume may be positive, negative, or zero depending on the condition of the soil at the time of wetting.

The primary factors that affect the wetting-induced volume change in a compacted cohesive soil are as follows (Lawton et al., 1992, 1993):

1. Gradation of the soil and mineralogy of the clay particles
2. State of stress at the time of wetting
3. Void ratio or dry density at the time of wetting
4. Moisture condition at the time of wetting
5. Degree of wetting
6. Fabric of the soil at the time of wetting
7. Chemistry of the water that is introduced into the soil

Soil type The effect of clay content on the one-dimensional wetting-induced volume change of compacted silt-clay and sand-clay mixtures was studied by El Sohby and Rabbaa (1984), with the results shown in Fig. 6A.94. The clay was highly expansive, with calcium montmorillonite the predominant mineral (45% by weight). For any given soil type (same silt or sand and clay fraction), the net collapse increased or the net swelling decreased with increasing applied vertical stress. For the applied stress levels studied (σ_v = 400, 800, or 1600 kPa or 8.4, 16.7, or 33.4 ksf), the specimens containing low clay fractions collapsed and those containing high clay fractions swelled. The magnitude of swelling at high clay fractions increased with increasing clay content. At low clay contents, the silt-clay collapsed more than the sand-clay; at moderate to high clay contents, the silt-clay col-

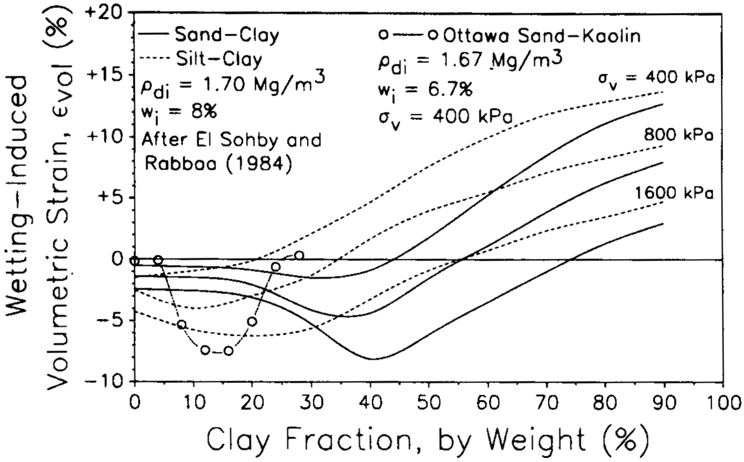


FIGURE 6A.94 Influence of clay fraction on potential for wetting-induced volume change of clay-sand and clay-silt mixtures (from Lawton et al., 1992).

lapsed less or swelled more than the sandy clay. The maximum collapse occurred at clay contents of 10 to 20% for the silt-clay and 30 to 40% for the sand-clay.

A similar study was conducted by the author to determine the influence of clay content on the wetting-induced volume change of mixtures of uniform sand and slightly expansive clay (kaolin). The results from this study are also shown in Fig. 6A.94 and can be summarized as follows:

1. At very low clay contents (<5%), little or no wetting-induced volume change occurred.
2. At clay contents from between 5 and 25%, collapse occurred, reaching a maximum between 12 and 16%.
3. At clay contents greater than 25%, a small amount of swelling occurred.

The effects of soil type on the potential for wetting-induced volume change in cohesive soils can be summarized as follows (Lawton et al., 1992):

1. *Very low clay contents*. The clay acts as a binder between silt and sand particles. When wetted, the clay binders soften and lubricate the intergranular contacts, thereby facilitating collapse at high applied stresses. If the clay is highly expansive, there may be some swelling at low pressures and high densities, but the magnitude will likely be small because the clay content is too low for the soil to have appreciable swelling. Silts tend to collapse more than sands.
2. *Low to moderate clay contents*. With increasing clay content, more clods are present in the borrow material, and for the compaction water contents at which significant collapse and swelling potentials exist, the as-compacted soil retains to some extent the identities of the individual clods. Upon wetting, the clods swell but also soften and distort under the applied load, so that the net volume change depends on the stress state and the expansiveness of the clay minerals. Swelling tends to occur at low stresses and for expansive clay minerals, and collapse tends to occur at high stresses and for nonexpansive clay minerals.
3. *Moderate to high clay contents*. Although some collapse occurs during wetting, swelling generally predominates over collapse, especially for highly expansive clay minerals and low stresses. For highly expansive clay minerals and high clay contents, swelling can occur at overburden pressures as great as 1600 kPa (33.4 ksf).

Density and moisture condition The effects of prewetting density and prewetting moisture condition on the one-dimensional wetting-induced volume change of a compacted clayey sand subjected to an applied total vertical stress (σ_v) of 200 kPa (4.2 ksf) are shown in Fig. 6A.95. The prewetting dry density is represented as a percentage of modified Proctor maximum dry density, R_{mw}, and the prewetting moisture condition is expressed as degree of saturation, S_{rw}. The values of wetting-induced volumetric strain, ε_{vw}, shown in Fig. 6A.95 are for complete inundation with water and therefore represent the maximum potential for wetting-induced volume change. For the same S_{rw}, ε_{vw} is more positive (greater swelling) or less negative (less collapse) as R_{mw} increases because the stiffness of the soil increases with increasing dry density. Thus, for the same moisture condition and total vertical stress, the swelling resulting from the reduction in effective stresses would be the same, although the collapse resulting from the reduction in stiffness would be less, with a net increase in swelling or decrease in collapse. For the same R_{mw}, the matric suction in the soil decreases with increasing moisture, with the result that the net swelling or net collapse decreases as S_{rw} increases up to the value of S_{rw} at which the suction is zero (typically in the range of 70 to 90%). Therefore, for the same dry density and total vertical stress, the net wetting-induced volume change—whether positive or negative—will be less for an initially wetter soil because the magnitude of the dominating mechanism—reduction in effective stress for net swelling or reduction in stiffness for net collapse—will be less for lower values of prewetting suction.

For the same prewetting moisture condition (S_{rw}), an increase in dry density either reduces the collapse potential or increases the swelling potential. Collapse in most compacted cohesive soils seems

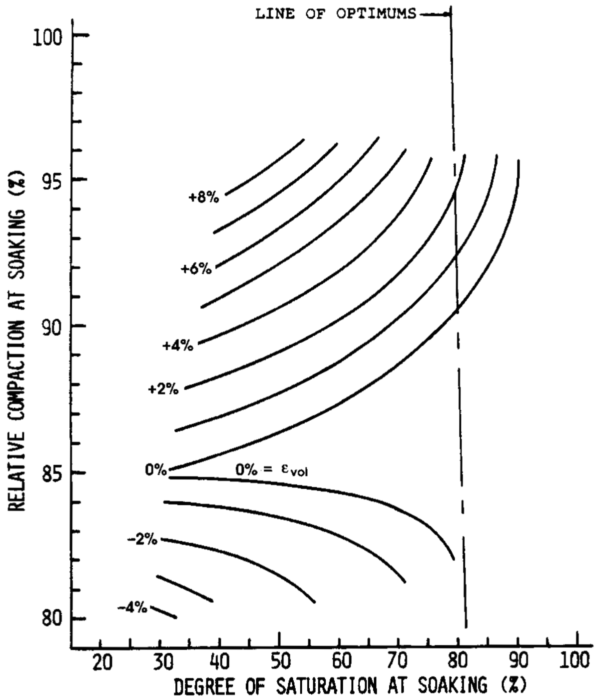


FIGURE 6A.95 Isograms of one-dimensional wetting-induced vertical strain for an applied vertical stress of 200 kPa (4.2 ksf) as a function of prewetting relative compaction and prewetting degree of saturation (from Lawton, 1986).

to occur primarily as a reduction in volume of the interclod voids (Yi, 1991). In a denser soil, the interclod voids are smaller. After wetting, the soil will come to an equilibrium density based on the state of stress. Therefore, less reduction in volume will occur for a denser soil with the same prewetting moisture condition. Swelling is a complex phenomenon that cannot be easily characterized. A greatly simplified explanation for the influence of density on swelling potential can be given as follows:

1. For a given saturated cohesive soil with the same soil fabric and water chemistry, there is a unique relationship between void ratio and state of stress.
2. For the same fabric and moisture condition of the prewetting soil, when soaked the soil will come to an equilibrium void ratio that depends on the state of stress acting on the soil but is independent of the prewetting density. Therefore, the increase in void ratio caused by soaking will be greater for an initially denser soil.

The wetting-induced volume change in compacted cohesive soils can also be zero. In compacted cohesive soils, this can occur in three ways: (*a*) The swelling and collapsing components are equal and therefore offset each other; (*b*) the prewetting suction in the soil is small enough that no significant reduction in effective stress or stiffness will occur upon wetting; or (*c*) cementation of particles is strong enough to preclude any wetting-induced volume change. Cementation of particles may exist in clods prior to compaction and may survive the compaction process to some extent. Cementation may also occur naturally with time after compaction or may be intentionally induced in the soil by chemical stabilization prior to compaction. These latter two possibilities are discussed in more detail in subsequent sections.

For a cohesive soil loaded and wetted shortly after compaction (before any significant post-compaction cementation occurs), there is a range of prewetting dry densities and degrees of saturation for which there is no wetting-induced volume change, as illustrated by the zone in Fig. 6A.95 bounded by the two isograms for $\varepsilon_{vw} = 0$. No wetting-induced volume change occurs at low degrees of saturation because the swelling and collapsing components offset each other, and in Fig. 6A.95 this occurs at $R_{mw} \cong 85\%$. At degrees of saturation equal to or greater than a critical value (S_{rc}), the prewetting suction is small, and $\varepsilon_{vw} \cong 0$. This phenomenon is indicated in Fig. 6A.95 by the increase in distance between the two isograms for $\varepsilon_{vw} = 0$ at higher values of S_{rw}. S_{rc} varies as a function of soil type, prewetting dry density, and prewetting state of stress. For a given soil, the influence of density and state of stress is small, and S_{rc} can be approximated as S_{opt} obtained from Proctor tests or can be determined more accurately by conducting a series of one-dimensional wetting-induced volume change tests.

Another important characteristic illustrated by the isograms of Fig. 6A.95 is that for any soil and any value of σ_v, there is range in values of R_{mw} for which $\varepsilon_{vw} = 0$ regardless of moisture condition. The value of R_{mw} at the middle of this range is called the *critical prewetting relative compaction* (R_{mwc}), and for a given soil R_{mwc} increases as σ_v increases (Fig. 6A.96).

STATE OF STRESS The effect of increased stress for the same prewetting density, moisture condition, and fabric is to reduce the swelling potential or increase the collapse potential. This relationship is illustrated in Fig. 6A.97, where isograms of one-dimensional wetting-induced strain are given for a compacted clayey sand at two levels of overburden pressure. At each point in $R_{mw} - S_{rw}$ space, ε is less positive or more negative at $\sigma_v = 800$ kPa than at $\sigma_v = 400$ kPa.

For three-dimensional strain conditions, the total wetting-induced volume change is uniquely related to the mean normal total stress (Lawton et al. 1991) and thus is independent of the principal stress ratio, as illustrated in Fig. 6A.98 for wetting tests on kneading-compacted specimens of a compacted clayey sand conducted under axisymmetric loading conditions. However, the individual components of wetting-induced volume change depend significantly on the stress ratios. For a given magnitude of stress in a direction of interest, the swelling increases or the collapse decreases in that direction as the perpendicular stresses increase. This concept is illustrated in Fig. 6A.99, where vertical and radial wetting-induced strains are plotted as a function of vertical stress. An increase in lateral stress ratio (σ_r/σ_v) for any given value of vertical stress results in an increase in vertical strain (increase in swelling or decrease in collapse) and a decrease in radial strain (decrease in swelling or increase in collapse).

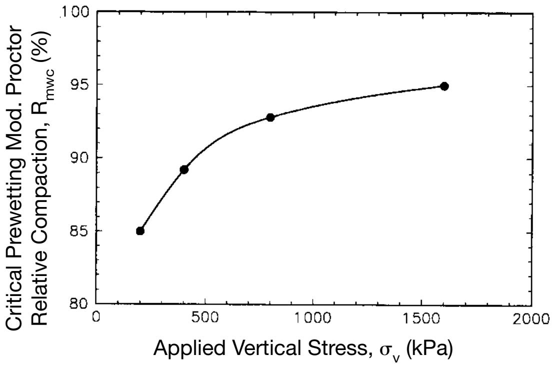


FIGURE 6A.96 Effect of vertical stress on critical prewetting density (data from Lawton, 1986).

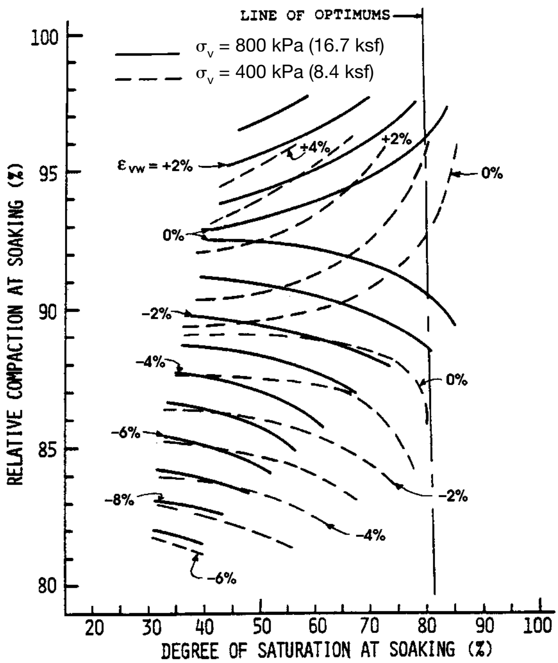


FIGURE 6A.97 Influence of applied vertical stress on isograms of wetting-induced volume change as a function of prewetting relative compaction and prewetting degree of saturation for a compacted clayey sand (data from Lawton, 1986).

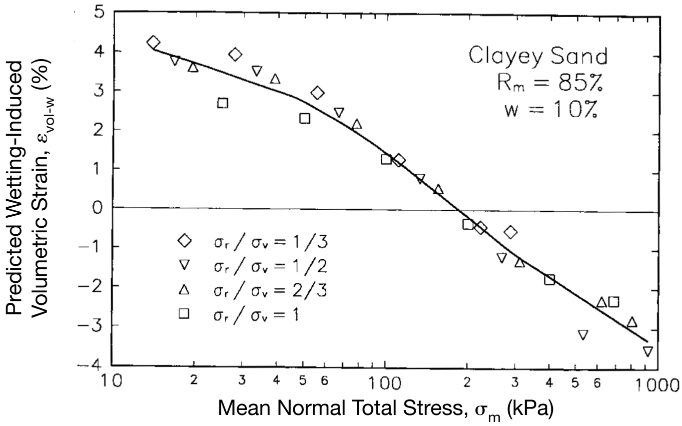
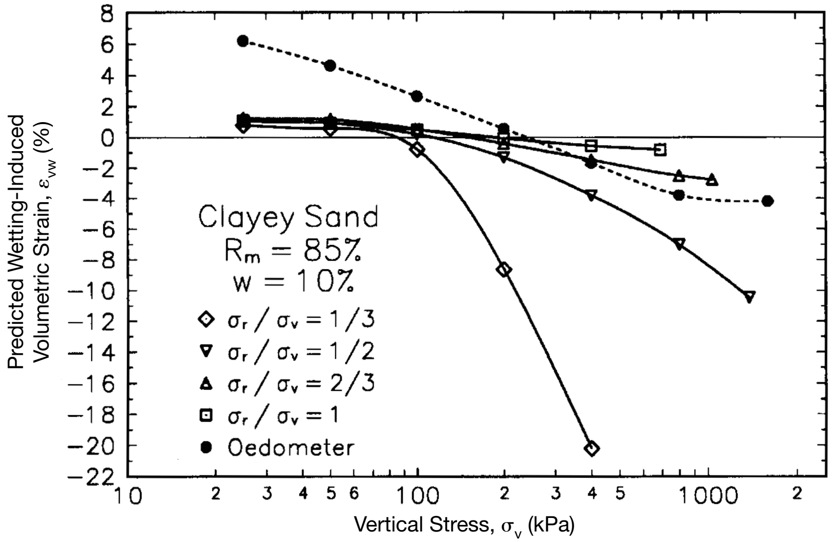


FIGURE 6A.98 Predicted wetting-induced volumetric strain as a function mean normal total stress from double triaxial tests on kneading-compacted specimens of a clayey sand (from Lawton et al., 1991).

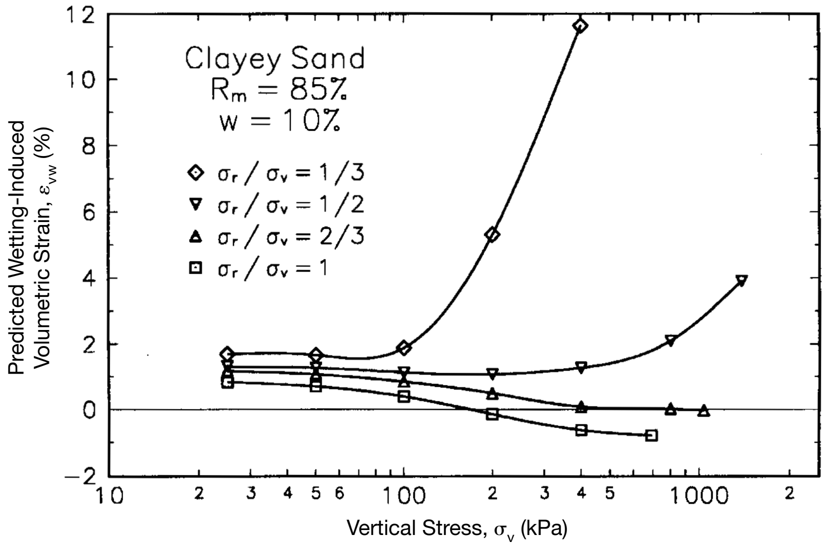
Results from wetting tests on oedometer specimens compacted at the same water content to the same density using the same method of compaction are also shown in Fig. 6A.99(*a*) for comparison with the triaxial results. This comparison clearly indicates that the one-dimensional wetting tests may not give reliable predictions for situations where three-dimensional strains occur. It is also apparent that for the range of lateral stress ratios studied (σ_r/σ_v = ⅓ to 1), collapse was affected more by the stress ratio than was swelling. In addition, the results appear to indicate that the oedometer tests significantly overestimate the swelling for three-dimensional volume change. However, the process of application and removal of loads during compaction can result in significant lateral earth pressures, which can be many times greater than the theoretical at-rest values and may approach limiting passive earth pressure values (Duncan and Seed, 1986). Thus, the values of σ_r/σ_v for as-compacted soils may be much greater than the maximum value of 1.0 shown in Fig. 6A.99. In addition, for a constant vertical stress, the horizontal stresses may vary substantially as the soil undergoes wetting-induced volume change. Preliminary research indicates that the horizontal stress in a compacted soil increases with increasing vertical collapse and decreases with increasing vertical swelling, that the limiting passive horizontal pressure may be approached for large vertical collapse, and that limiting active horizontal pressure may be achieved for large vertical swelling (Lawton et al., 1991). Thus, laboratory tests in which σ_r/σ_v is kept constant do not accurately model the stress path of soils in the field that undergo three-dimensional wetting-induced volume change, and additional research is needed to clarify the interdependence of horizontal stress and wetting-induced volume change of soil.

Method of Compaction The influence of compaction method on the one-dimensional wetting-induced volume change of a clayey sand is shown in Fig. 6A.100 for three different as-compacted compositions and can be summarized as follows:

1. For the same water content, density, and state of stress, the statically compacted specimens swelled more than the kneading-compacted specimens.
2. Within the lower range of stresses at which collapse occurred, the magnitude of collapse was less for the statically compacted specimens than for the kneading-compacted specimens.
3. At stresses near or greater than the stresses at which maximum collapse would occur for any given composition, the statically compacted specimens collapsed more than the kneading-compact-



(a)



(b)

FIGURE 6A.99 Comparison of predicted wetting-induced (a) vertical strain, and (b) radial strain versus vertical stress curves from double triaxial and double oedometer tests on kneading-compacted specimens of a clayey sand (from Lawton et al., 1991).

ed specimens. This trend is not observed for R_m = 90% and w = 10% because the maximum stress on the specimens was less than the stress at which maximum collapse would occur.

Statically compacted cohesive soil can be either more or less collapsible than the comparable kneading-compacted soil because static compaction tends to produce a fabric that is susceptible to both more swelling and more collapse. Recall from previous discussions that both swelling and collapse occur when an expansive soil is wetted. At low stresses, the swelling component clearly dominates, and swelling is greater for static compaction than for kneading compaction. At high stresses, the collapsing component dominates and the statically compacted soil collapses more than the kneading-compacted soil. There is a transition zone for moderate stresses wherein the net wetting-induced volume change is negative (collapse) but the tendency for greater swelling in the statically-compacted soil results in less collapse than for the kneading-compacted soil.

Although method of compaction can have a measurable effect on ε_{vw}, it is apparent from the results in Fig. 6A.100 that w and σ_v influence ε_{vw} to a much greater degree than method of compaction. To illustrate this point, examples of changes in ε_{vw} produced by changes in w, σ_v, R_m, and method of compaction using ε_{vw} = +8.6% (static compaction, R_m = 90%, w = 10%, σ_v = 25 kPa) as the base are as follows:

Change	ε_{vw}, %	$\Delta\varepsilon_{vw}/\varepsilon_{vw\text{–base}}$, %
Increase w to 16%	+1.4	–84
Increase σ_v to 200 kPa	+ 3.1	–64
Decrease R_m to 80%	+4.4	–49
Change to kneading compaction	+7.0	–19

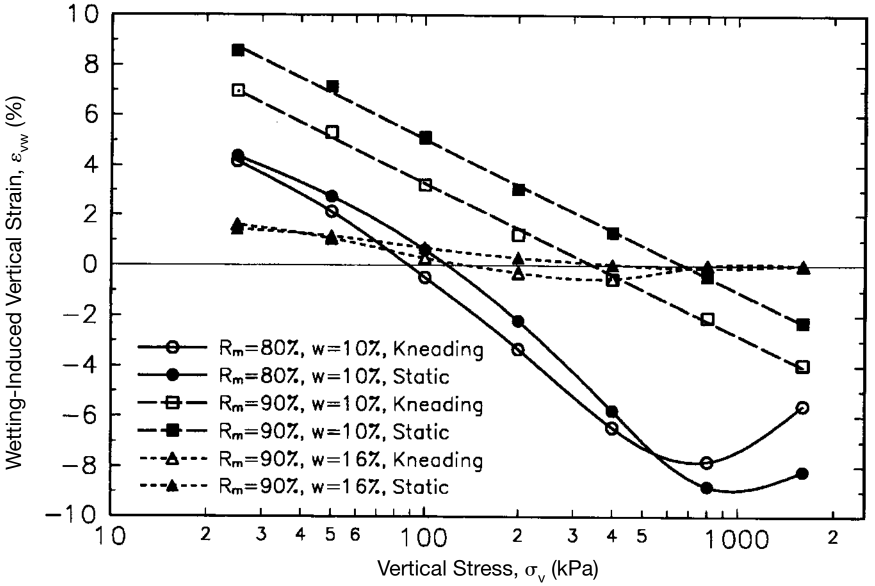


FIGURE 6A.100 Influence of compaction method on one-dimensional wetting-induced strain for a clayey sand (data from Lawton, 1986).

This discussion is not meant to imply that method of compaction is unimportant; rather that with respect to wetting-induced volume change in cohesive soils, it is a relatively minor factor compared to w and σ_v. In addition, it should be borne in mind that specimens which will be tested in the laboratory should always be prepared with a method of compaction that simulates as closely as possible the method of compaction to be used in the field compaction process.

Degree of wetting The degree of wetting has a substantial influence on the wetting-induced volume change; that is, the results of wetting tests in which the specimen is inundated with water represent the maximum wetting-induced volume change for the conditions under which the specimen was tested. Partial wetting reduces the matric suction somewhat but to a lesser degree than full wetting. Thus, the magnitude of wetting-induced volume change, which is significantly influenced by the changes in suction, will be less for partial wetting than for full wetting. This is illustrated in Fig. 6A.101 for collapse of three naturally deposited soils. Note that the collapse for the three soils shown in Fig. 6A.10l is essentially completed at degrees of saturation less than 100%, typically about 80%. Similar trends have been noted by the author for the wetting-induced volume change (both collapse and swelling) of compacted cohesive soils. In general, little or no wetting-induced volume change occurs in compacted cohesive soils for additional wetting above S_{opt}.

Oversize particles When tests are conducted in the laboratory to estimate the potential for wetting-induced volume changes in field-compacted soils, the specimens used for testing are generally small, and the maximum particle size that can be used in a specimen is limited to approximately one-sixth the least dimension of the specimen. In this context, the term *oversize* refers to sizes of particles that are larger than the maximum size that can be used in a specimen for any particular test, and the maximum size will therefore vary depending on the test being performed and the size of the sample being tested. In general, the oversize particles are gravel size and larger. As with other types of tests, the results from laboratory wetting-induced volume change tests should be corrected to account for the effects of the oversize particles that are present in the field-compacted soil but not in the specimens tested in the laboratory.

A common method of correcting for the lack of oversize particles in laboratory wetting tests is based on the following two assumptions:

1. The oversize particles are inert and therefore do not change volume when wetted.
2. The presence of the oversize particles in the borrow material during field compaction does not influence the potential for wetting-induced volume change within the matrix soil (the soil exclusive of the oversize particles).

With these two assumptions, the results from laboratory tests on specimens prepared at the same compaction water content as the field matrix soil to the same dry density as the field matrix soil us-

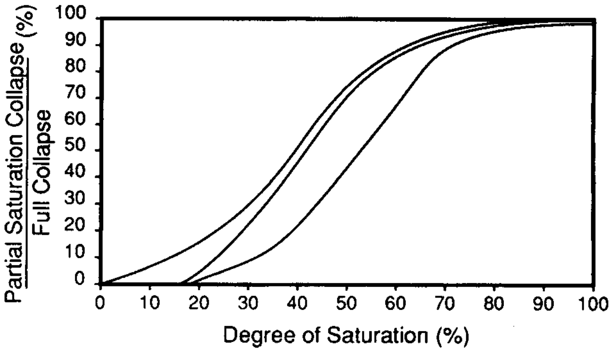


FIGURE 6A.101 Partial collapse curves for three collapsible silts (from Walsh et al., 1993).

ing the same (or similar) method of compaction and loaded to the same state of stress can be used to predict the potential for wetting-induced volume change of the field-compacted soil. The dry density and water content of the field matrix soil (γ_{dm} and w_m) can be calculated from the following equations, which are derived from phase relations:

$$\gamma_{dm} = \frac{(1 - P_{ow}) \cdot G_{so} \cdot \gamma_w \cdot \gamma_d}{G_{so} \cdot \gamma_w - P_{ow} \cdot \gamma_d (1 + G_{so} \cdot w_o)} \tag{6A.37}$$

$$w_m = \frac{w - w_o \cdot P_{ow}}{1 - P_{ow}} \tag{6A.38}$$

where w = water content of the field-compacted soil
γ_d = dry density of the field-compacted soil
P_{ow} = fraction of the soil that is oversized particles, by dry weight, expressed as a decimal
G_{so} = specific gravity of the oversize particle solids
w_o = water content of the oversize particle fraction
γ_w = density of water

w_o can be estimated by removing the oversize particles from a sample of the field-compacted soil and scraping the surfaces of the oversize particles to remove any cohesive material that is adhered to them. This can be a cumbersome task, and a value for w_o is usually estimated in the range of 0.5 to 2%. The following example illustrates the calculation of γ_{dm} and w_m for the preparation of laboratory specimens for wetting tests (after Noorany and Stanley, 1994): Given that w = 11.7%, γ_d = 112.1 pcf (17.61 kN/m^3), P_{ow} = 15%, G_{so} = 2.66, and w_{os} assumed to be 1%, calculate values of γ_{dm} and w_m for the field-compacted soil

$$\gamma_{dm} = \frac{(1 - 0.15)(2.66)(62.43)(112.1)}{(2.66)(62.43) - (0.15)(112.1)[1 + (2.66)(0.01)]} = 106 \text{ pcf} \left(16.7 \frac{\text{kN}}{\text{m}^3}\right)$$

$$w_m = \frac{0.117 - (0.01)(0.15)}{1 - 0.15} \cdot 100 = 13.6\%$$

Some preliminary research has been conducted to determine the validity of this method for predicting wetting-induced volume change of field-compacted soil. Day (1991) studied the effect of gravel content on the magnitude of one-dimensional swelling of specimens with an expansive silty clay matrix. The wetting tests were conducted at low stresses—σ_v = 0.1 to 0.2 psi (0.7 to 1.4 kPa)—and swelling occurred in all tests. The results from these tests are given in Fig. 6A.102 in the form of the wetting-induced strain for the specimens containing gravel normalized with respect to the wetting-induced strain for the comparable specimen with no gravel at the same applied stress ($\varepsilon_{vw}/\varepsilon_{vw0}$). Also shown in Fig. 6A.102 is the theoretical relationship for $\varepsilon_{vw}/\varepsilon_{vw0}$ obtained from the following equation, which was derived from the same assumptions discussed above except that the gravel was assumed to be dry:

$$\frac{\varepsilon_{vw}}{\varepsilon_{vw0}} = \frac{(1 - P_{ow}) \cdot G_{so} \cdot \gamma_w}{(1 - P_{ow}) \cdot G_{so} \cdot \gamma_w + P_{ow} \cdot \gamma_{dm}} \tag{6A.39}$$

A comparison of the experimental and theoretical results suggests that this method provides a reasonable estimate for predicting the one-dimensional swelling potential of compacted cohesive soils.

A similar study was conducted by the author and his associates (Larson et al., 1993) with regard to the influence of oversize particles on the one-dimensional collapse of compacted clayey sands.

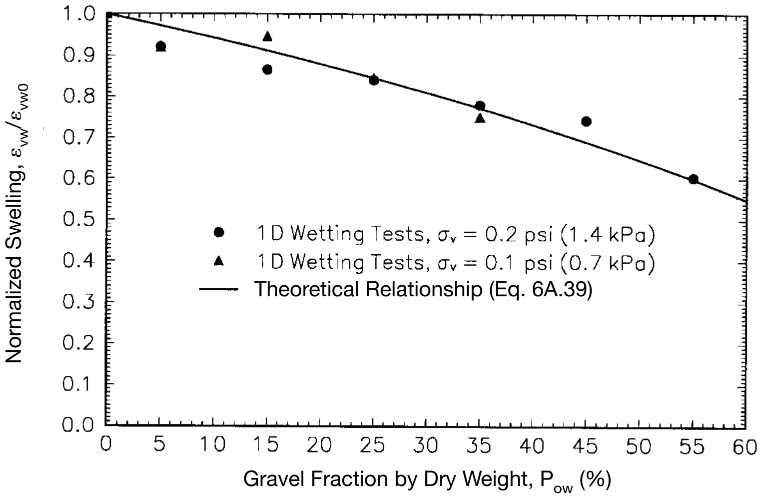


FIGURE 6A.102 Effect of gravel content on one-dimensional swelling of a compacted silty clay (modified from Day, 1991).

Experimental values of $\varepsilon_{vw}/\varepsilon_{vw0}$ from this study are plotted in Fig. 6A.103 as a function of percentage of oversize particles by volume (P_{ov}). The experimental results are compared with the following theoretical equation, which is equivalent to Eq. (6A.39) except that $\varepsilon_{vw}/\varepsilon_{vw0}$ is given in terms of P_{ov} rather than P_{ow}:

$$\frac{\varepsilon_{vw}}{\varepsilon_{vw0}} = 1 - P_{ov} \tag{6A.40}$$

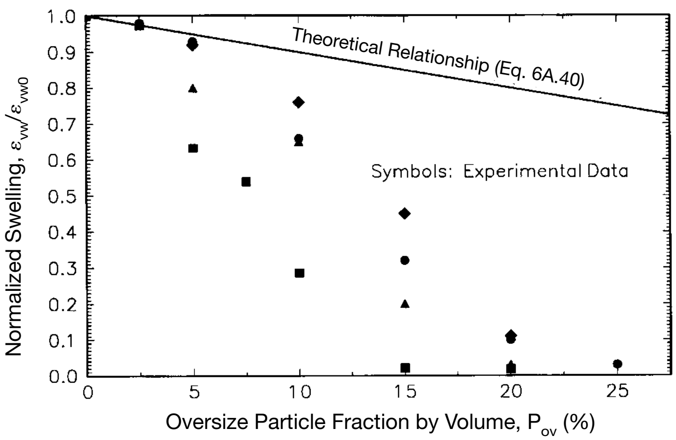


FIGURE 6A.103 Influence of oversize particles on the collapse of compacted clayey sands (from Larson et al., 1993).

For the soils and conditions studied, the theoretical relationship is valid only for small oversize fractions ($P_{ov} \cong 2.5$ to 5%; $P_{ow} \cong 4$ to 8%), thus indicating that at higher contents of oversize particles, the presence of the oversize particles changes the fabric of the matrix soil, the manner in which collapse occurs, or both. The magnitudes of collapse for the five specimens with $P_{ov} \geq 20\%$ ($P_{ow} \geq 28\%$) were less than 11% of the values for the comparable specimens without any oversize particles.

Why this method of correcting for oversize particles apparently works for swelling but not for collapse is not well understood. We know that the oversize particles affect the fabric of the as-compacted soil (e.g., Siddiqi et al., 1987; Fragaszy et al., 1990). In soils with P_{ow} less than about 40%, the oversize particles "float" in the matrix soil, as shown in Fig. 6A.104; therefore, there is little or no interaction between oversize particles, and the behavior of the soil is controlled by the matrix soil. Soils with the same overall matrix density and water content but varying amounts of oversize particles will have different fabrics, with the matrix soil adjacent to the surfaces of the oversize particles (near-field matrix) having higher void ratios than the matrix soil that is distant from the oversize particles (far-field matrix soil). For conditions where net swelling occurs, the amount of swelling in the near and far-field matrices is different, but the overall magnitude of swelling for the soil is about the same. Because the oversize particles move farther apart as the swelling occurs, they do not interact, and their presence does not appreciably affect the swelling process. The influence of the oversize particles on the collapse process is less well understood. For the results shown in Fig. 6A.103, as little as 5% oversize particles (by volume) decreased the collapse by as much as 37%. At this low-oversize particle content, appreciable interaction among the oversize particles is unlikely, so the reduction in collapse must be primarily related to changes in fabric. Apparently little additional collapse occurs within the looser near-field matrix compared to the soil without any oversize particles, and collapse within the denser far-field matrix is substantially reduced. Some of the reduction may occur from the oversize particles bridging the near-field matrix zones that underlie the particles so that even if collapse occurs in these areas, it may not decrease the total volume of the soil.

Time rate of wetting-induced volume change The time rate of one-dimensional wetting-induced volume change for two specimens of a clayey sand with the same as-compacted condition but loaded to a low and a high vertical stress and then soaked are shown in Fig. 6A.105. The specimen loaded to $\sigma_v = 1$ kPa swelled, and the specimen loaded to $\sigma_v = 400$ kPa collapsed. When plotted versus the log of time, the wetting-induced strain curves are similar in shape to those for loading-induced strain of saturated fine-grained soils. In Fig. 6A.105, three distinct regions of wetting-in-

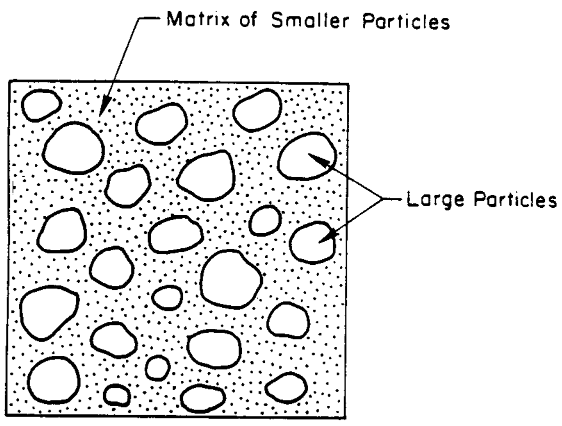


FIGURE 6A.104 Schematic representation of large particles floating in a matrix of smaller particles (from Siddiqi et al., 1987).

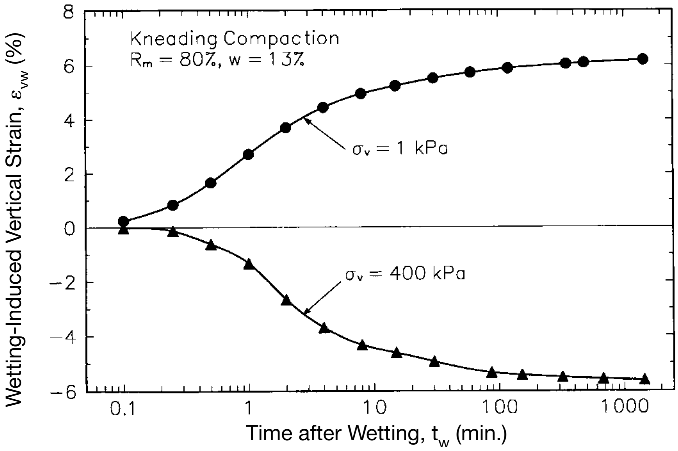


FIGURE 6A.105 Time rate of one-dimensional wetting-induced volume change for compacted clayey sand (data from Lawton, 1986).

duced strain are evident. Each curve is initially flat as the water flows through the porous stones and enters the specimen. As water infiltrates the specimen, the majority of the wetting-induced strain occurs and the curve is relatively steep. After the matric suction in the specimen is eliminated, the wetting-induced strain is nearly complete, and the curve again becomes flat. At some time after wetting, the effective stress in the specimen becomes constant, but secondary compression will continue to occur. This region of secondary compression is apparent in the specimen that collapsed but not in the specimen that swelled because the magnitude of the secondary compression for the low applied stress level is too small to be measured. Eventually, the secondary compression will become negative.

Wetting-induced volume changes occur quickly in oedometer specimens inundated in the laboratory, usually within a few hours or less, which suggests that the wetting-induced volume change at any location in a field-compacted soil takes place within a few hours of sustained wetting (Lawton et al., 1992). Thus, the time required for complete wetting-induced volume change to occur in a field-compacted soil depends on (*a*) the method by which wetting occurs, and (*b*) the rate of water infiltration, which is a function of the permeability of the soil.

Control of wetting-induced volume change The potential for wetting-induced volume change can be maintained within acceptable limits (*a*) before compaction by changing the engineering properties of the borrow material, (*b*) during design and construction by properly written compaction specifications and adequate supervision and inspection of the compaction process, or (*c*) after the compaction process by preventing water from infiltrating the soil or by modifying the properties of the soil. Preventing the infiltration of water into a compacted soil over a long period is nearly impossible and is not a viable alternative. Modifying the properties of the soil after compaction is generally difficult and expensive. Thus, the preferred alternatives are a and b, and they will be discussed in detail below.

From the data shown in Fig. 6A.97, there are two ways in which the potential for wetting-induced volume changes in a fill consisting of expansive compacted soil can be theoretically eliminated or substantially reduced during the compaction process without modifying the borrow soil: (*a*) by compacting the soil to a degree of saturation greater than the optimum saturation or (*b*) by compacting the soil at any depth within the fill to a dry density corresponding to the appropriate critical relative compaction based on the anticipated value of σ_v at that level in the fill. However, both methods have practical limitations that may preclude their use on any project. If the borrow materials are highly variable—and they usually are on large fills—determining appropriate values of S_{rwc} and R_{mwc} for the different materials would be difficult or very expensive. In addition, the quality control procedures required to ensure that the compacted soil has the desired properties are much more extensive and stringent than typically used for most projects. Some potential drawbacks to compacting to a high degree of saturation include:

1. The degree of saturation changes from the as-compacted condition owing to compression caused by placement of additional fill or structures built on or within the fill. However, compression causes an increase in S_r, so that a fill constructed with $S_r \geq S_{rwc}$ will meet the requirements as long as the soil does not dry out so much that S_r drops below S_{rwc}.

2. Postcompaction wetting or drying of the soil may occur, especially for shallow depths along the exposed surfaces of the fill. Continuous or intermittent wetting wherein the moisture content increases gradually over a long period constitutes part of the wetting process and will not result in any significant volume change if the as-compacted soil is properly compacted ($S_r \geq S_{rwc}$). Cyclic wetting and drying can be troublesome, especially near the top of the fill, where the stress levels are low and cyclic swelling/shrinkage may occur. Swelling and shrinkage are primarily elastic phenomena and will continue to occur so long as the wetting/drying cycles occur, although recent research suggests that the magnitude of the swelling and shrinkage may decrease somewhat for the first few cycles and then remain constant thereafter. Wetting-drying cycles at depths within the fill where the stress levels are high enough to result in collapse are unusual, but these cycles may occur near the bottom of moderately deep to deep embankments along the exposed surfaces. Wetting-drying cycles are generally less of a problem for collapsible conditions than for swelling conditions because collapse is primarily plastic, and any shrinkage caused by drying is small and decreases the potential for further collapse. Drying of the soil produces an increase in density owing to shrinkage and an increase in suction, which can be a major problem at low stress levels in an expansive fill because both phenomena increase the swelling potential. At high stress levels, the increase in density decreases the collapse potential, but this decrease is usually more than offset by the increase in collapse potential resulting from the reduction in suction. Thus, post-compaction drying at all depths within a compacted cohesive soil can be important, because it increases either the collapse potential or the swelling potential depending on the stress level. Postcompaction drying commonly occurs in embankments located in regions with semiarid and arid climates.

3. Compacted cohesive soils with high degrees of saturation have more time-dependent compression than the same soil at low degrees of saturation (for the same stress level and dry density), as discussed previously. For $S_r > S_{\text{opt}}$, the air phase is probably occluded and the time-dependent compression behavior may be similar to that for the same soil in the saturated condition.

Swelling of the bearing soils for a foundation can be eliminated by ensuring that the applied bearing stress from the foundation is greater than the *swelling pressure* of the bearing soil. *Swelling pressure* is defined as the pressure needed to prevent swelling of a soil as it is takes on moisture from its in situ condition to its stable moisture level. Swelling pressure is usually determined in the laboratory on a specimen contained within a cylindrical mold that is subjected to water and is nominally prevented from swelling. A calibrated steel bar or a strain gauge is commonly used to measure the force that develops in the specimen because it is prevented from expanding. However, in both types of devices, the developed force is determined from a calibration factor or curve that is based on that amount of deformation that occurs in the bar or the gauge. Thus, some upward movement of the specimen does occur, with the magnitude of movement depending on the stiffness of the force measuring device, and the calculated swelling pressure (measured force divided by the area of the specimen) is less than the actual swelling pressure by some unknown amount. An alternative method for determining swelling pressure is to perform a series of oedometric wetting tests on nominally identical specimens subjected to increasing applied stresses and to determine graphically the swelling pressure from a plot of ε_{vw} versus σ_v as the value of σ_v at which the wetting-induced volume change is zero. The results from such a series of tests would be similar to any one of the curves shown in Fig. 6A.100. For example, the swelling pressure is about 340 kPa (7.1 ksf) for the specimen in Fig. 6A.l00 prepared by kneading compaction at w = 10% to R_m = 90%. One must make sure, however, that the applied bearing stress is not so high that an unacceptable potential for wetting-induced collapse exists.

If the borrow soil is nonexpansive or slightly expansive, swelling potential is not a concern, and collapse potential can be controlled by compacting the soil to a high density. The critical relative compaction above which little or no collapse occurs for any reasonable compaction water content varies considerably from soil to soil but is typically in the range of R_m = 85 to 100% (R_s = 90 to 110%). The value for any soil can be estimated from a series of one-dimensional laboratory tests performed on specimens prepared at the driest water content to be used in the field compaction to various densities.

Chemical stabilization of expansive borrow soils prior to compaction has been used successfully for many years to control the swelling potential of the as-compacted soil. The most commonly used stabilizing agents are Portland cement, hydrated lime, and fly ash. Chemical stabilization can also be used to control the collapse potential of compacted soils but to date has been little used for this purpose. The primary advantage to using chemical stabilization to control wetting-induced volume change is that the potential for swelling or collapse is unaffected by post-compaction drying of the soil. Other advantages include increased stiffness and strength of the as-compacted soil and little or no reduction in stiffness and strength if the soil becomes soaked. Potential disadvantages include increased brittleness and decreased permeability.

As shown previously, the addition of gravel to a cohesive borrow material may substantially reduce the potential for collapse but not swelling. Thus, for nonexpansive or slightly expansive soils, blending gravel with the cohesive borrow material may be an economically viable method for controlling collapse potential if sufficient quantities of inexpensive gravel are available.

Shrinkage. Expansive soils are also susceptible to shrinkage (decrease in total volume) caused by drying, which is also known as *desiccation shrinkage* to differentiate it from other phenomena that produce a decrease in volume. During drying, the matric suction increases, which in turn increases the effective stresses acting on the soil, resulting in a decrease in volume that may be accompanied by cracking. In general, the trends of factors that affect shrinkage potential are opposite to the trends of the same factors for swelling, that is, if increasing a certain factor tends to increase the swelling potential of a compacted soil, increasing that same factor tends to decrease the shrinkage potential. The most important factors that affect the shrinkage potential of compacted cohesive soils are as follows:

1. *Soil type.* The more expansive a soil is, the more susceptible it is to shrinkage. Soils that have high shrinkage potentials generally contain some highly active clay minerals such as montmorillonite.
2. *Moisture condition.* The drier a soil is prior to desiccation, the greater its matric suction and the lesser the increase in effective stress for complete drying. Thus, shrinkage potential decreases with decreasing moisture condition (see Fig. 6A.106).
3. *Dry density*. For the same moisture condition and fabric, denser soils shrink less than looser soils.
4. *Fabric*. The influence of fabric on the axial shrinkage of a silty clay is shown in Fig. 6A.107 for three pairs of specimens where each pair had the same density and water content but different fabrics when shrinkage was initiated. For each pair of specimens, one specimen was compacted dry of optimum and the other was compacted wet of optimum, and then both specimens were soaked to the same composition. In each case the dry-compacted specimen (flocculated microfabric) shrank substantially less than the wet-compacted specimen (dispersed microfabric). The greater resistance to shrinkage for the flocculated microfabric can be attributed to two factors: (*a*) The flocculated microfabric contains more interparticle contacts that resist shrinkage, and (*b*) a portion of the load must be spent reorienting the nonaligned particles in the flocculated fabric to a more parallel arrangement before any reduction in average void ratio occurs.

During drying of compacted expansive soils with occluded air phases, the increase in suction resulting from drying acts on the entire surface area of all soil particles and thus produces an increase

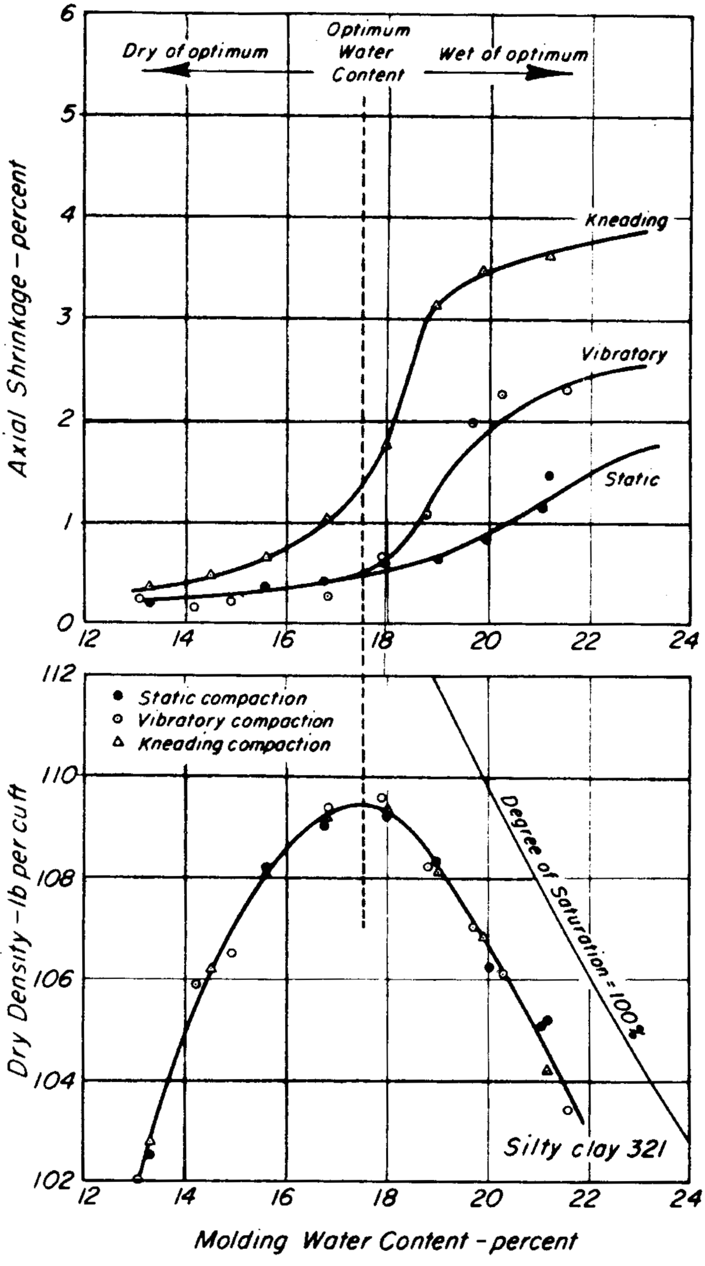


FIGURE 6A.106 Influence of method of compaction and molding water content on axial shrinkage of a silty clay (from Seed et al., 1960).

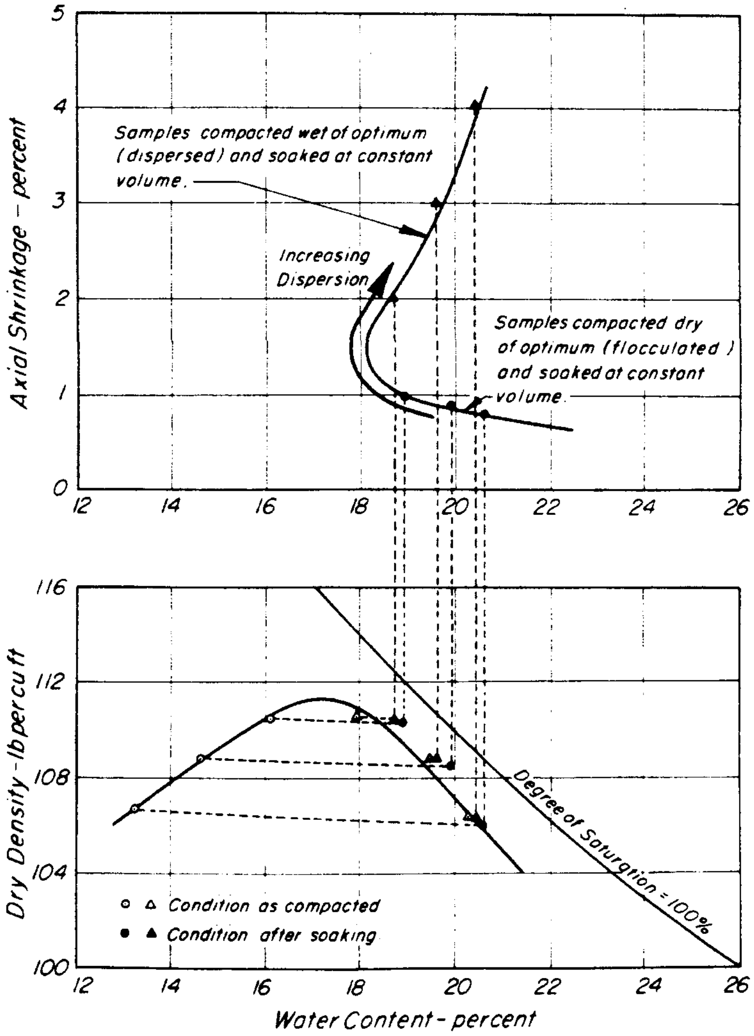


FIGURE 6A.107 Effect of fabric on the axial shrinkage of a silty clay (from Seed and Chan, 1959*a*).

in effective stress equal to the increase in suction. When some of the pore air spaces become continuous through adjacent void spaces, the suction no longer acts on the total surface area of the particles, and not all the increase in suction is transmitted to the soil as effective stress. As drying continues to occur, the portion of the increase in suction that is transmitted to the soil as effective stress decreases (see Fig. 6A.59). Shrinkage also produces an increase in stiffness of the soil owing to both the increase in density and the increase in suction. Shrinkage ceases when the increase in effective stress resulting from desiccation is no longer sufficient to produce a measurable decrease in volume. The water content of the soil when the shrinkage stops is called the *shrinkage limit*, which generally decreases with increasing plasticity of the soil. Partial drying of the soil to a water content above the shrinkage limit results in less shrinkage than drying to or below the shrinkage limit. The shrinkage limit of the soil as it dries from an as-compacted, partially saturated state, may be a somewhat different value than that determined by the standard shrinkage limit test (ASTM D427) where the soil is dried from a loose, saturated state. The shrinkage limit for compacted cohesive soils may also vary somewhat depending on the density and water content at which drying initiates.

It is important to note that shrinkage of an expansive soil may also occur from moisture use by vegetation (transpiration). However, according to Brown (1992), the potential for foundation distress relative to transpiration is probably much less than normally assumed, and as long as the moisture loss is confined to capillary water, the soil itself is not particularly affected. However, at some point other moisture may be pulled from the clay and may result in shrinkage and potential foundation settlement. The greatest potential for transpiration-induced settlement is from trees located close to shallow foundations. However, tree roots can be beneficial to foundation stability, since they tend to act as soil reinforcement and increase the soil's stiffness and shearing resistance. Thus, the practical result of a tree located close to a shallow foundation may be to impede foundation settlement, even though the roots remove moisture from the soil (refer to Sec. lA).

Available data indicate that significant structural distress occurs much more frequently from swelling-induced heave than from shrinkage-induced settlement. Of the 502 foundation inspections in Dallas County, Texas, reviewed by Brown (1992) during a particular study period, 216 showed some movement but did not warrant repairs. Of the 286 cases that did warrant foundation repairs, 87 resulted from settlement and 199 resulted from heave. Because it is likely that nearly all of the heave cases were caused by swelling, and the settlement cases could have resulted from a number of phenomena other than shrinkage (e.g., loading-induced compression, marginal bearing capacity, or erosion of supporting soil), it is clear that structural distress is much more likely to occur from swelling than from shrinkage.

Cyclic Swelling and Shrinkage. Cyclic swelling and shrinkage of expansive bearing soils is a serious problem that introduces two issues that are not a concern for a single cycle of either swelling or shrinkage: (*a*) Cyclic swelling and shrinkage may cause fatigue cracking or failure in the foundation or structure for situations where the same amount of movement in one cycle of swelling or shrinkage would not, and (*b*) the magnitude of cyclic swelling and shrinkage cannot be controlled by compaction alone. Cyclic swelling and shrinkage is of most concern in regions where expansive surficial soils are prevalent and therefore might commonly be used as borrow material, where the summers are hot and dry, and where there is a substantial rainy season in the winter (e.g., southern California and Texas).

As discussed in the two previous sections, the potential for either swelling or shrinkage can be kept to acceptable levels by proper compaction of the soil—compaction wet of optimum to control swelling, and compaction dry of optimum to control shrinkage. However, both swelling and shrinkage potential cannot be controlled by compaction moisture content alone because reducing the potential for either phenomenon increases the potential for the other. In addition, the beneficial effect of compaction moisture control is eliminated by subsequent cycles of wetting and drying. For example, Day (1994) showed that the one-dimensional swelling of a silty clay compacted wet of optimum ($R_m = 92\%$, $S_{ri} = 89\% > S_{\text{opt}} = 80\%$) was only 1.0% for the first cycle of wetting (after an aging period of 21 days) but increased to 8.0% for the second and subsequent cycles of wetting after drying.

Because problems from cyclic swelling and shrinkage cannot be eliminated by compaction moisture control alone, alternative methods must be used. It is theoretically possible to reduce substantially postcompaction wetting or drying of the bearing soil, but this is difficult to accomplish in practice over a long period of time. Blending gravel with the borrow material may effectively eliminate the shrinkage potential if a sufficient amount is used, but swelling potential will only be reduced in proportion to the volume of gravel added; thus, this method is unlikely to be economical in many instances. Therefore, chemical stabilization is the preferred alternative on most projects.

Permeability. Compacted cohesive soils have low permeabilities, with values of coefficient of permeability (k) for small specimens tested in the laboratory typically within the range of 1×10^{-4} cm/s to 1×10^{-11} cm/s (3×10^{-1} to 3×10^{-8} ft/day). The low permeability of compacted cohesive soils generally causes two types of concerns in applications for building foundations:

1. Rainfall and snowmelt tend to pond on the surface or run off the surface rather than infiltrate the compacted soil. Thus, more extensive drainage systems are required around buildings founded in compacted cohesive soils than are needed for more permeable granular soils.
2. Local or regional ground-water aquifers and flow patterns may be affected by compacting an existing cohesive soil or by constructing a compacted cohesive fill. This factor may require extensive (and expensive) measures to mitigate potential problems or to compensate adjacent property owners who may be adversely affected. Potential problems include reduction in available water supplies for nearby landowners and damage from diversion of additional water onto adjacent properties because of erosion or wetting-induced volume changes.

The low permeability nature of compacted cohesive soils can be put to productive engineering use in the form of compacted clay liners, which are used to impound water, hazardous waste, and other liquids. Some information on compacted clay liners is given below, but because they are used infrequently in building projects, an in-depth discussion of this topic is beyond the scope of this book. A good reference for additional details on compacted clay liners is Daniel (1993).

The in situ permeability of a compacted cohesive soil is generally controlled by the macrofabric, which may include macroscopic characteristics such as cracks, fissures, root holes, animal burrow holes, and flow channels between lifts that increase the permeability of the in situ soil. Because laboratory specimens are typically quite small, these macroscopic features are generally not included in laboratory specimens. Therefore, laboratory specimens contain only the microfabric and minifabric features of the in situ soil, so that values of k obtained from laboratory testing represent lower-bound values. To obtain a realistic estimate of the in situ permeability, field tests—such as the sealed double ring infiltration test (see Sec. 6A.3.5)—should be conducted over an area of the compacted soil that is large enough to represent the macrofabric of the soil. The following factors can have a major influence on the field permeability of a compacted cohesive soil:

1. *Soil gradation and mineralogy.* The permeability of any soil depends on the size distribution and shape of the portions of the voids through which water can flow. In cohesionless soils, the water essentially can flow through the entire voids. In cohesive soils, the adsorbed (double-layer) water is strongly attracted to the clay particles, and the flow of water is limited to the portions of the voids outside the boundaries of the double layers. Thus, the permeability of a cohesive soil depends on the clay content as well as the size distribution and activity of the clay particles. In general, the permeability of a cohesive soil decreases with increasing clay content and activity and decreasing size of the clay particles. Because activity of the clay minerals is inversely related to particle size, plasticity index is a general indicator of relative permeability, with k decreasing with increasing plasticity of the soil. For clay contents greater than about 30% by weight, the clay fraction dominates the permeability to the extent that the gradation of the rest of the soil has little influence on the permeability unless there is a high percentage of particles gravel size or larger. Laboratory tests on uniformly blended mixtures of gravel and cohesive soils have shown that gravel contents less than about 60% have little effect on permeability (Fig. 6A.108) because the gravel floats in the finer matrix (see Fig. 6A.104). At higher gravel contents, the finer particles exist within the void spaces of a skeleton formed by the gravel particles, and the finer soil may not completely fill the voids of the skeleton. Thus, the permeability of the soil decreases rapidly with increasing gravel content above 60%. In the field, the borrow materials cannot be blended as uniformly as in the laboratory so that some segregation of the gravel particles occurs, with the result that gravel skeletons may form at gravel contents as small as 30%. For compacted clay liners where very low permeability is required, it is recommended that the gravel content of the borrow soil be limited to a maximum of 30%.

Large particles (coarse gravel, cobbles, and boulders) inhibit the compaction of the near-field matrix soil (finer soil close to the particles), whereas the far-field matrix soil is unaffected by the presence of the large particles. This produces zones of near-field matrix soil that may be significantly less dense and much more permeable than the far-field matrix soil. These relatively high permeability zones provide a preferential path for water flow through the soil and hence substantially increase its overall permeability. Thus, for low-permeability applications, all particles larger than

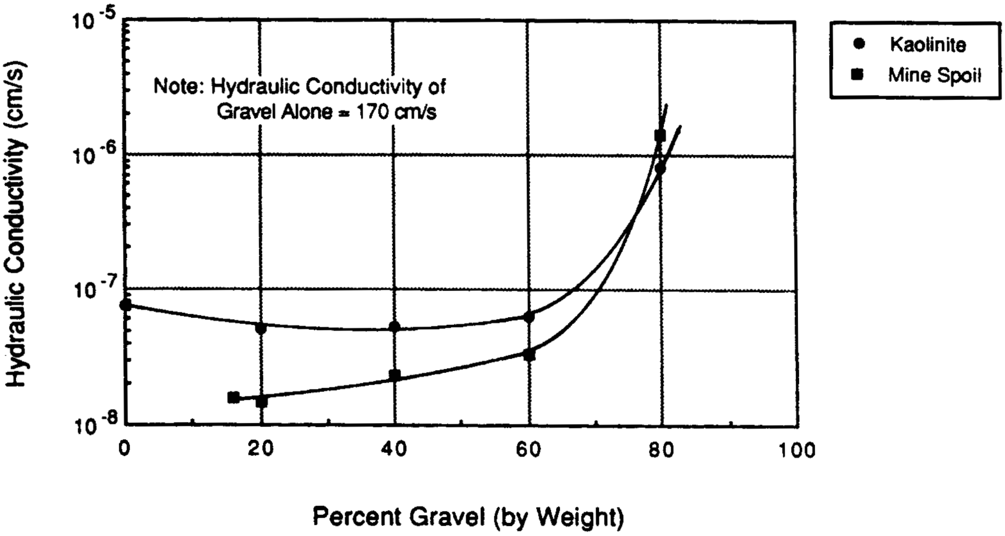


FIGURE 6A.108 Influence of gravel content on hydraulic conductivity of kaolinite/gravel and mine spoil gravel mixtures (from Shelley and Daniel, 1993).

about 1 to 2 in (25 to 50 mm) should be removed from the borrow material prior to moisture conditioning and compaction.

2. *Fabric.* The ease with which water can flow through any void in a given soil depends on the smallest dimension of the void in the direction perpendicular to the flow. The voids in flocculated microfabrics may be irregular in shape but tend to be equidimensional, whereas the voids in dispersed microfabrics are long and narrow. Therefore, for the same composition and density of the microfabric, flocculated fabrics are more permeable than dispersed fabrics. At the minifabric scale, a fabric wherein the clods have been remolded into a relatively homogeneous material is much less permeable than one in which the clods retain some of their individual identity and thus have relatively large interclod pores.

3. *As-compacted moisture condition and density.* Wetter cohesive soils have softer clods that are more easily remolded. For otherwise comparable compaction conditions, wetter compacted cohesive soils tend to have more dispersed microfabric and more homogeneous minifabric and therefore lower permeability. For the same method of compaction and compaction water content, increased density tends to reduce the permeability in two ways: (*a*) smaller interclod and intraclod void spaces, and (*b*) more dispersed microfabric. Isograms of k for laboratory specimens of silty clay prepared by kneading compaction are shown in water content-density space in Fig. 6A.109; k for this soil varied by over four orders of magnitude depending on the compaction water content and as-compacted density. (Note: Not all permeability data for the silty clay is reflected in Fig. 6A.109; see Mitchell et al., 1965 for additional data.) Wet of optimum ($S_r > S_{opt}$), the permeability of the soil was primarily a function of degree of saturation and was little affected by density. Note also the rapid decrease in k with increasing S_r. Density played a more substantial role dry of optimum, but S_r was still the primary factor affecting k. To achieve low permeability in compacted clay liners, the soil should be compacted wet of the line of optimums. Other engineering characteristics of the soil—such as shear strength and shrinkage potential—may also be important, so a moisture-density zone should be specified that meets all the important criteria (Fig. 6A.110).

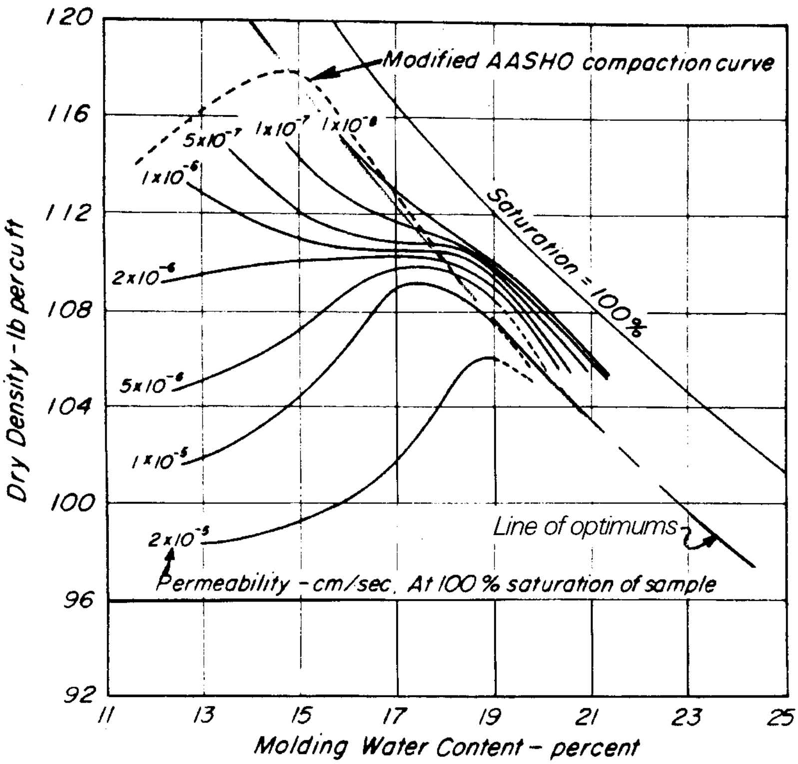


FIGURE 6A.109 Isograms of coefficient of permeability for specimens of silty clay prepared by kneading compaction (from Mitchell et al., 1965).

4. *Method of compaction.* Very dry of optimum, the method of compaction generally has little influence on the fabric of the compacted soil. Wet of optimum, increasing the shearing strain induced in the soil during compaction tends to produce a more dispersed microfabric and greater remolding of the clods. Therefore, kneading compaction results in lower permeability than does static compaction for otherwise comparable conditions, and impact compaction produces an intermediate permeability. A comparison of permeability for kneading and statically compacted specimens of a silty clay is shown in Fig. 6A.111. All kneading rollers (sheepsfoot, tamping foot, and rubber-tired) can remold a wet cohesive borrow material into a relatively homogeneous mass for low permeability applications. However, unless good bonding is achieved between lifts, hydraulic defects in adjacent lifts may be hydraulically connected to each other. To produce good interfacial bonding in compacted clay liners, sheepsfoot rollers with feet longer than the lift thickness are usually specified. Heavier rollers are needed in drier soils to break down the clods; in wet soils, lighter rollers should be used that do not become bogged down in the soft soil.

5. *Compactive effort.* Increased compactive effort generally results in higher density, greater dispersion of the microfabric, and increased remolding of clods, all of which result in lower permeability. At some point, additional passes of the compaction equipment will produce no significant changes in the compacted soil. In some soft cohesive soils, additional passes beyond a certain number may produce loosening and weakening of the soil and a concomitant increase in permeability.

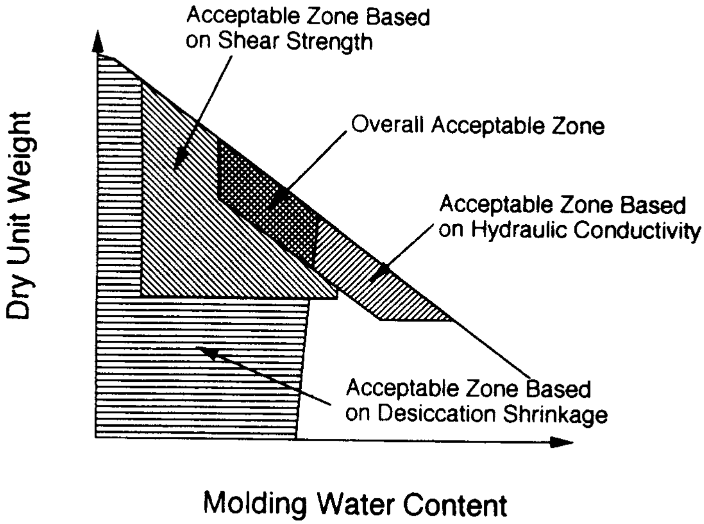


FIGURE 6A.110 Overall acceptable moisture-density zone based on hydraulic conductivity, shear strength, and shrinkage upon desiccation (from Daniel, 1993).

6. *Compaction water content and moisture conditioning period.* The water content of the soil and the distribution of moisture within the clods determine to a great extent the amount of remolding that will occur during compaction for a given method of compaction. Very dry clods can be extremely hard and brittle and will not remold but may be broken during compaction, and the permeability of dry-compacted cohesive soils is relatively high owing to flocculated microfabrics and large interclod pores. Wet clods are soft and are easily remolded, and wet-compacted soils have relatively low permeabilities because of dispersed microfabrics and small interclod pores. The distribution of moisture within the clods also influences the as-compacted fabric. Long periods of time—up to several weeks in some cases—may be required to achieve moisture equilibrium within highly plastic clods, with the required length of time increasing with the size of the clods. If an adequate moisture conditioning time is not used, the outer portions of the clods will be wet and soft, and the inner portions will remain dry and hard. For this reason—and because of potential problems with postcompaction shrinkage and swelling (see number 9 below)—highly plastic borrow materials are not recommended for use in compacted clay liners. The water content of the borrow material for compacted clay liners should be wet and soft enough to allow easy remolding of the clods but not so wet that the compaction and construction equipment have inadequate traction to function properly.

7. *Clod size.* Laboratory studies have shown that the size of the clods may affect k by several orders of magnitude (Benson and Daniel, 1990; Daniel, 1984; Houston and Randeni, 1992) and that k tends to increase with increasing size of clods, as illustrated in Fig. 6A.112. The influence of clod size on k decreases with increasing compaction water content, as shown in Fig. 6A.113. Because of this effect, a maximum clod size of about 0.75 in (19 mm) should be specified for compacted clay liners.

8. *Lift thickness.* The lift thicknesses for cohesive soils are usually in the range of 4 to 12 in (0.1 to 0.3 m). If the lift thickness is too large for the borrow soil and compaction equipment being used, the lower portions of the lift will not be properly compacted, resulting in a lift with lower k near the top and higher k near the bottom.

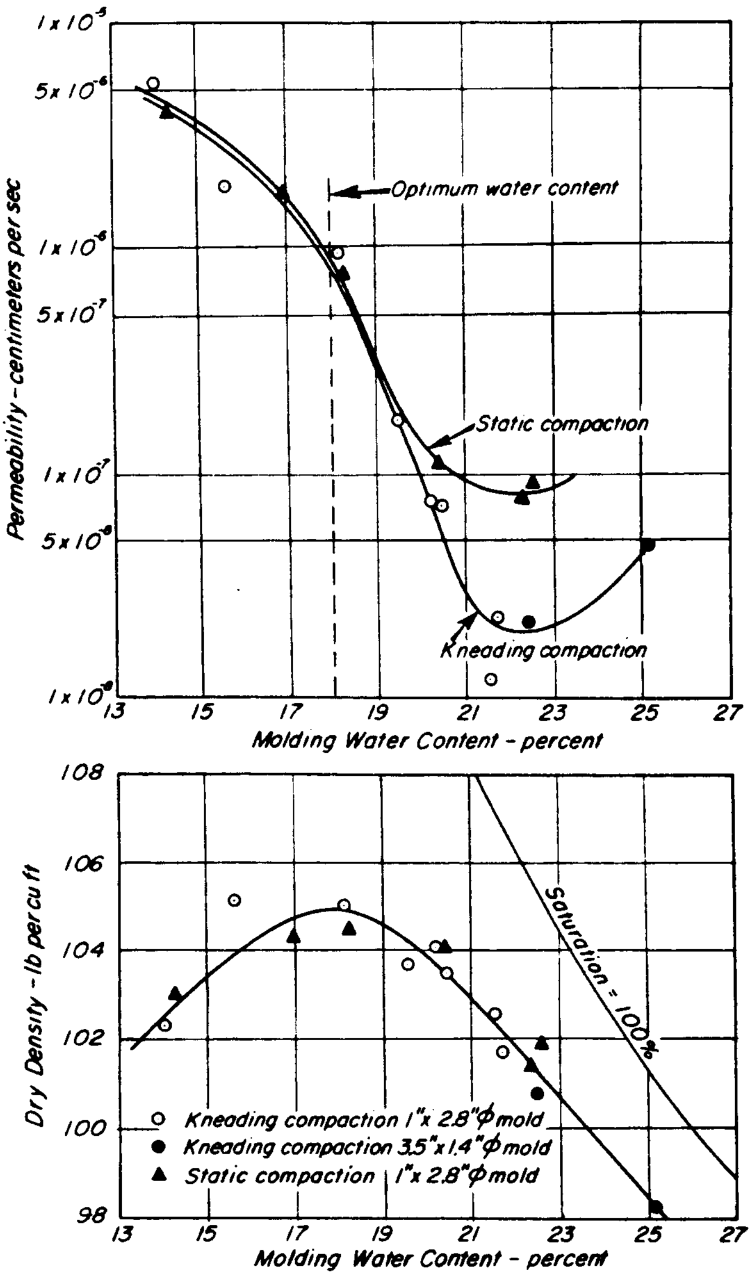


FIGURE 6A.111 Effect of method of compaction on the permeability of silty clay (from Mitchell et al., 1965).

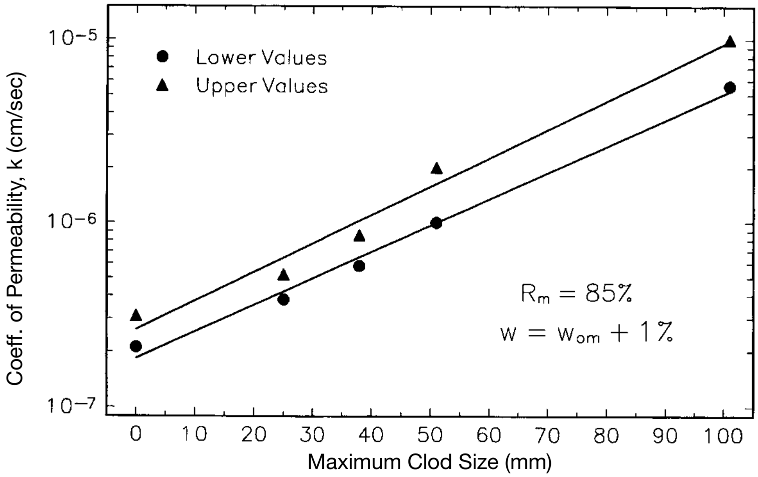


FIGURE 6A.112 Influence of clod size on the permeability of a compacted lean clay (data from Houston and Randeni 1992).

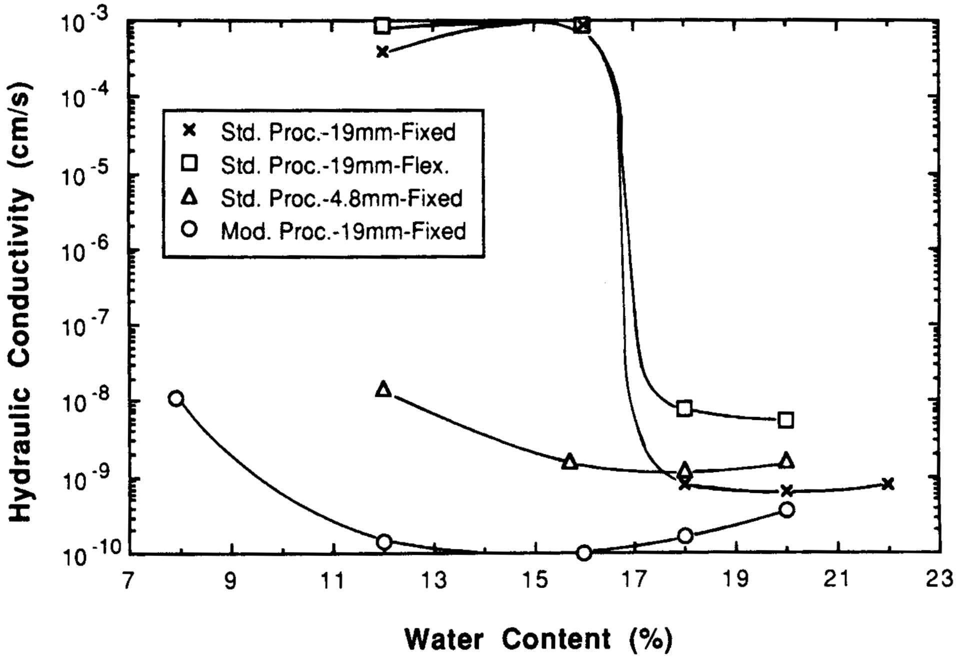


FIGURE 6A.113 Influence of clod size, compactive effort, and method of testing on the hydraulic conductivity of highly plastic, compacted clay (from Benson and Daniel, 1990).

9. *Postcompaction moisture changes*. Postcompaction wetting and drying of the compacted soil may result in changes in permeability, especially in highly plastic soils. Wetting may result in swelling and increased k owing to decreased density. Drying may produce cracks that can penetrate compacted cohesive soils to a depth of several inches in just a few hours, with a substantial increase in k (Boynton and Daniel, 1985). In low-permeability applications, it is especially important that swelling and shrinkage be prevented to the extent possible. Shrinkage cracking can be prevented by lightly spraying the surface of each compacted lift with water as needed to prevent drying, but care should be taken not to add so much water that swelling is induced. The compacted soil can also be protected from shrinkage by covering it with a light-color plastic liner. If heavy rain is expected, the soil should be covered with a plastic liner to prevent swelling and erosion. To prevent postcompaction drying of the compacted clay liner in composite liner systems, the geomembrane liner should be placed as quickly as possible after proofrolling the final lift of the clay liner.

10. *Postcompaction freeze-thaw cycles*. Postcompaction freezing and thawing of compacted cohesive soils may produce cracking and an increase in permeability of about 5 to 1000 times that of the as-compacted soil. The effect of freeze-thaw on four compacted clays is shown in Fig. 6A.114. The change in permeability caused by freezing and thawing appears to increase with increasing compaction water content. From the results of permeability tests conducted on a glacial clay (Fig. 6A.115), it can be seen that five freeze-thaw cycles increased the permeability by about 2 to 6 times for specimens compacted dry of optimum and about 100 to 250 times for specimens compacted wet of optimum (Kim and Daniel, 1992). Field tests conducted on a test pad constructed of compacted clay in which the uninsulated portion of the pad underwent up to 10 cycles of freeze-thaw showed that the permeability of the soil within the depth of frost penetration (0.5 m = 1.6 ft) increased by approximately 50 to 300 times compared to its as-compacted condition (Benson et al. 1995). However, because the frost penetrated only about 30% of the total thickness (1.5 m = 4.9 ft) of the test pad, the overall hydraulic conductivity of the liner was not affected by the freeze-thaw cycles. Compacted clay liners in cold climates should be insulated to protect them from the potential effects of freeze-thaw.

11. *Macrofabric*. It is obvious from the preceding discussions that cracks produced by such phenomena as desiccation shrinkage and freeze-thaw cycles can substantially increase the permeability of a compacted cohesive soil. Macroscopic hydraulic channels can also be opened by other

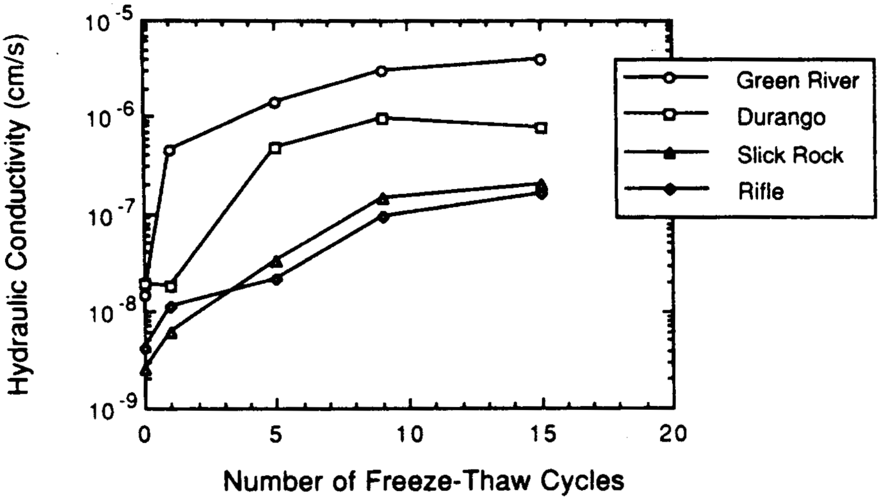


FIGURE 6A.114 Effect of freeze-thaw on the hydraulic conductivity of four compacted clays (from Kim and Daniel, 1992, data from Chamberlain et al., 1990).

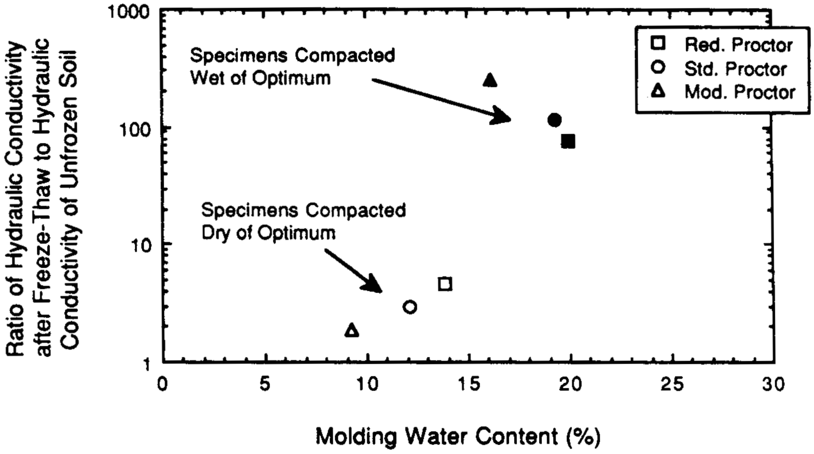


FIGURE 6A.115 Ratio of average hydraulic conductivity after five cycles of freeze-thaw to hydraulic conductivity of unfrozen soil for a glacial clay (from Kim and Daniel, 1992).

phenomena, including burrowing animals, erosion by wind or water, penetration of plant roots, dissolution of water-soluble coarse-grained particles (e.g., gypsum), and chemical dissolution of soil particles.

12. *Degree of saturation during flow.* The influence of degree of saturation during flow on the permeability of a compacted silty clay is shown in Fig. 6A.116. The increase in permeability with increasing degree of saturation is expected because of the increased pore volume through which water can flow (Mitchell et al., 1965). Theory and experimental results suggest that the permeability varies approximately as the cube of degree of saturation ($k \propto S_r^3$).

13. *Stress level.* The permeability of compacted cohesive soils decreases with increasing compressive stress level owing to an increase in density and perhaps some increase in dispersion of the microfabric, as depicted in Fig. 6A.117 (Boynton & Daniel 1985). Because field permeability tests are usually conducted on test pads at essentially zero overburden stress and therefore overestimate the in situ permeability, the results from field permeability tests should be corrected for stress level based on the results of laboratory permeability tests performed over a range of compressive stresses, as shown in Fig. 6A.118 (Daniel, 1993). Higher stress levels also tend to “heal” desiccation cracks, as illustrated in Fig. 6A.117. The data indicate that the cracks began to close at confining pressures of about 4 psi (28 kPa) and that the cracks were essentially closed at confining pressures greater than about 8 psi (55 kPa).

14. *Aging.* The effect of 21 days of aging on specimens of a compacted silty clay stored at constant water content and density is shown in Fig. 6A.119. This increase in permeability with aging was ascribed to an increase in flocculation of clay particles at the microscale (Mitchell et al., 1965). Boynton and Daniel (1985) reported that the permeability of the fire clay they studied increased for storage times up to 2 months and then decreased up to a storage time of 6 months. Thus, the effect of aging on the permeability of cohesive soils is not completely understood. Additional details on the effects of aging on the engineering characteristics of soils can be found in Sec. 6A.3.4.3.

Liquefaction. Both laboratory tests and field performance data indicate that the great majority of clayey soils will not liquefy during earthquakes (Seed et al., 1983). However, clayey soils with the following characteristics may be vulnerable to severe loss of strength from earthquake shaking:

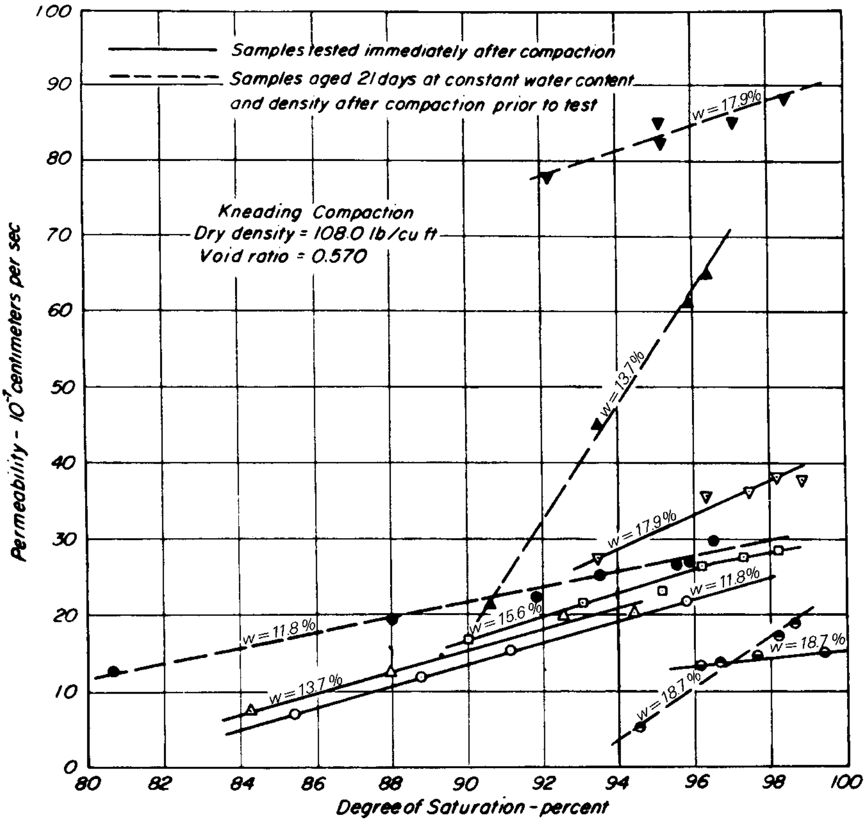


FIGURE 6A.116 Effect of degree of saturation during flow on the permeability of compacted silty clay (from Mitchell et al., 1965).

Percent finer than 0.005 mm < 15%

Liquid limit < 35

Water content > 0.9 × liquid limit

A rough estimate of the liquefaction potential for these soils may be obtained using the procedures given previously for sands and silty sands, but the best means of determining their cyclic load characteristics is by test.

6A.3.4.3 Aging, Thixotropy, and Cementation

Changes occur in the engineering properties of many compacted soils as the soils age. In general, the strength, stiffness, and permeability of compacted soils increase with aging, whereas the potentials for liquefaction, swelling, and shrinkage typically decrease. Examples of these changes are given in Figs. 6A.119 to 6A.123. Although the reasons for these changes are not completely understood, the following phenomena have been suggested as likely contributors: thixotropy, secondary compression, particle interference, clay dispersion, and cementation. Each of these phenomena is described in limited detail below. The reader is referred to Mitchell (1960) for additional details on thixotropy and to Schmertmann (1991) for further information about mechanical aging.

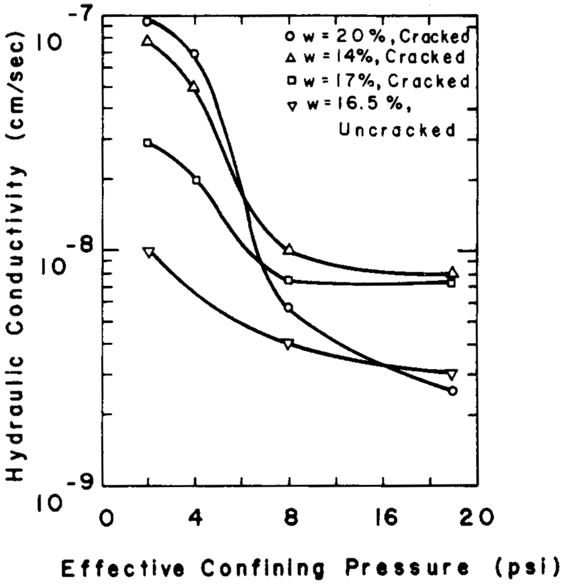


FIGURE 6A.117 Hydraulic conductivity versus effective confining pressure for uncracked and desiccation-cracked specimens of fire clay compacted at various molding water contents (from Boynton and Daniel, 1985).

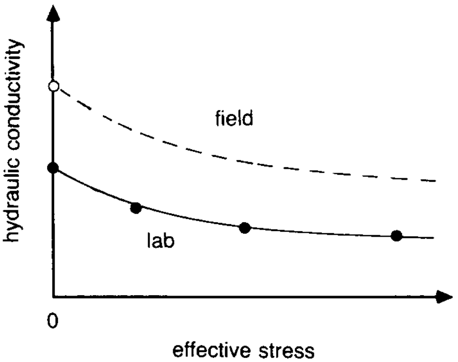


FIGURE 6A.118 Procedure for adjusting the hydraulic conductivity measured in the field on a test pad for the influence of compressive stress (from Daniel, 1993).

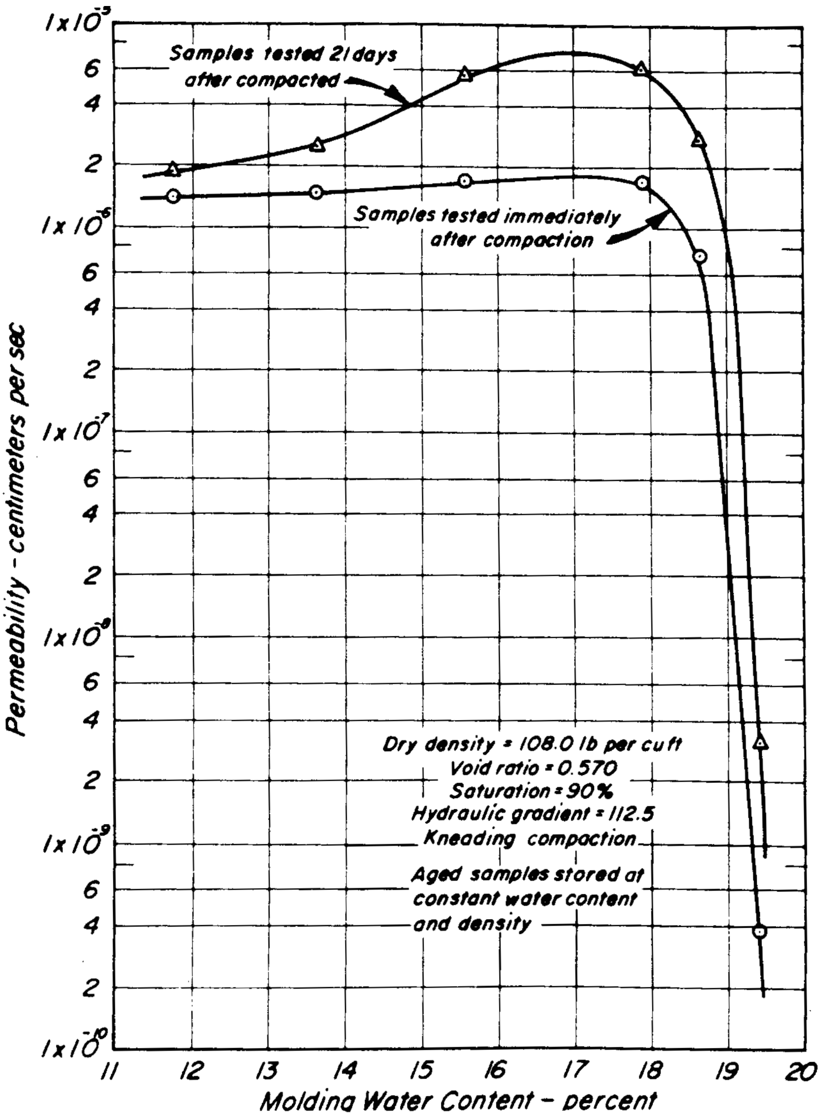


FIGURE 6A.119 Influence of aging at constant water content and density on the permeability of a compacted silty clay (from Mitchell et al., 1965).

Thixotropy. There is no universally accepted definition of *thixotropy* in soil mechanics. Mitchell (1960) defines thixotropy as an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby a cohesive soil stiffens while at rest and softens upon remolding. Simply stated, thixotropy is an increase in flocculation (randomness) of a cohesive soil fabric in undrained (constant-volume) conditions. The properties of a purely thixotropic material are given in Fig. 6A.124.

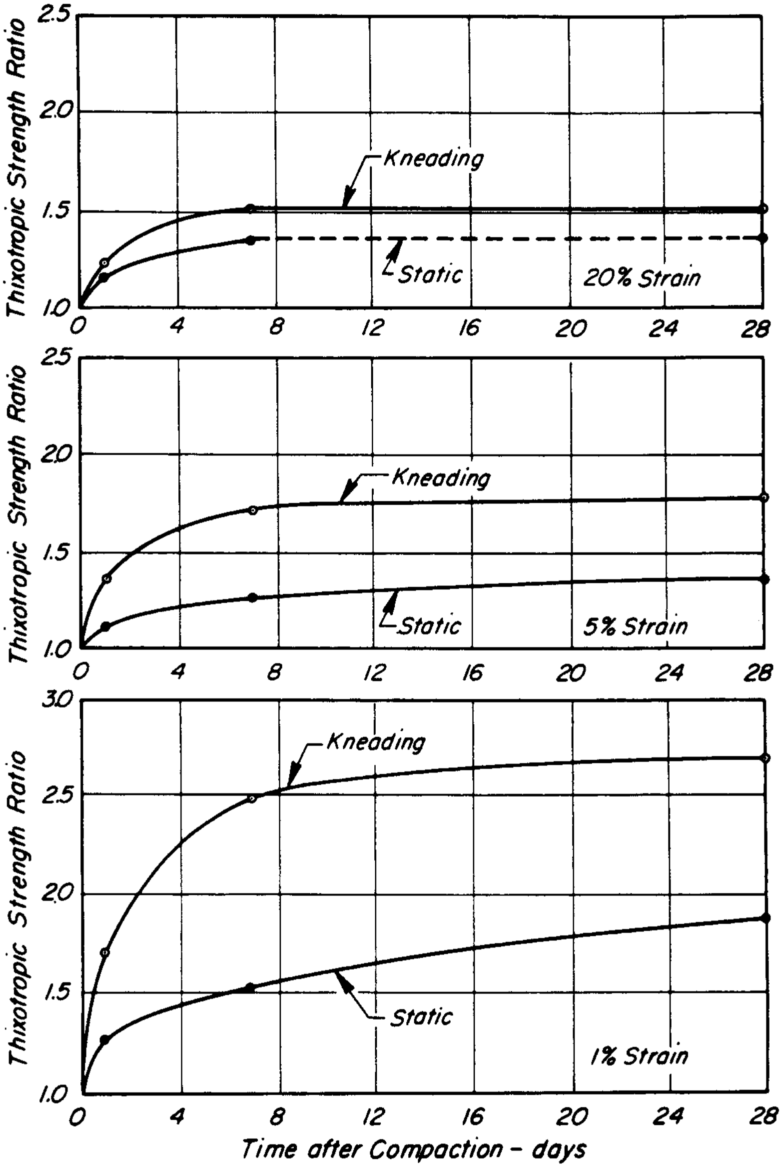


FIGURE 6A.120 Effect of method of compaction on the thixotropic strength ratio for a compacted silty clay (from Mitchell, 1960).

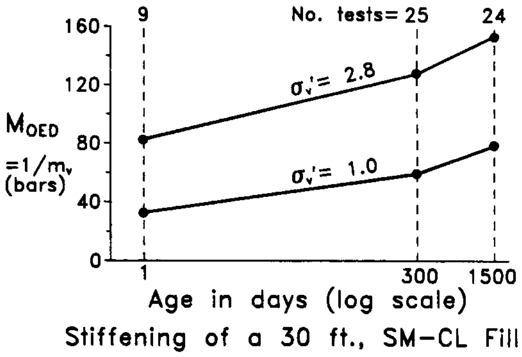


FIGURE 6A.121 Aging-induced increase in oedometer modulus of a compacted, cohesive fill (from Schmertmann, 1991).

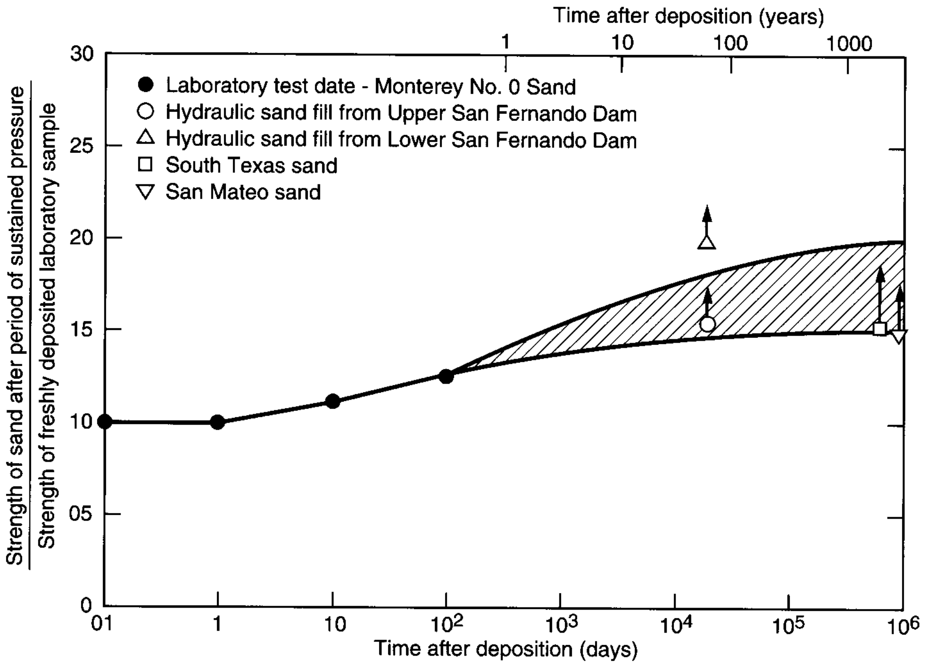


FIGURE 6A.122 Influence of aging under sustained pressure on the stress ratio causing liquefaction of sand (from Seed, 1979).

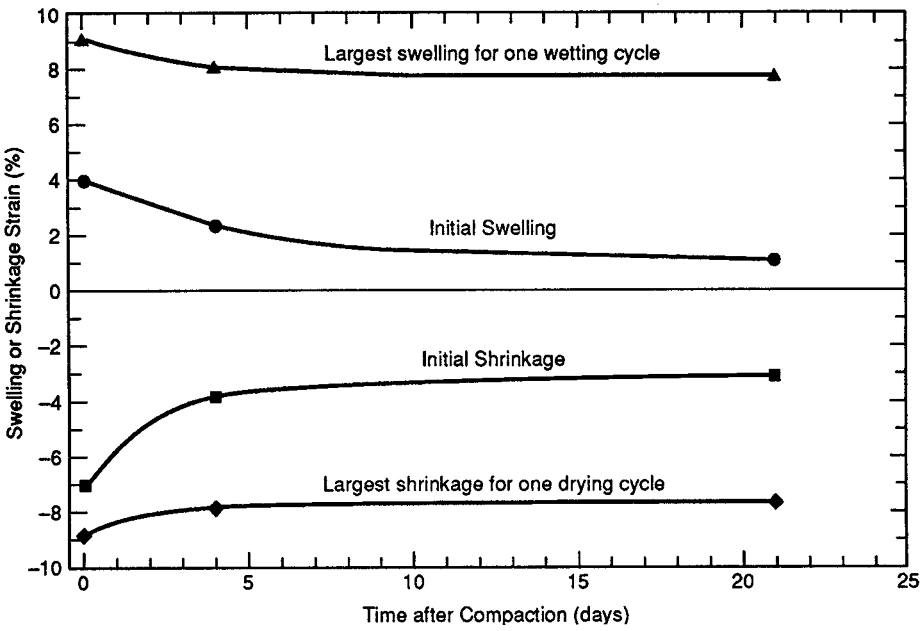


FIGURE 6A.123 Effect of aging on the swelling and shrinkage of a compacted silty clay (data from Day, 1994).

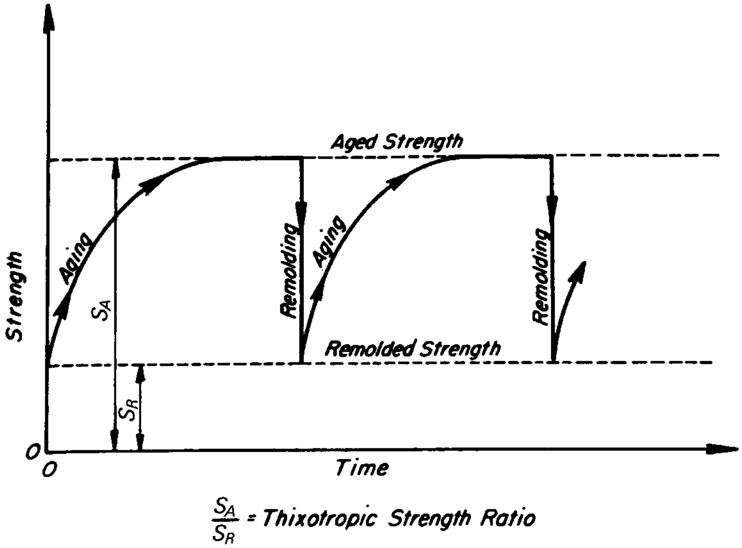


FIGURE 6A.124 Properties of a purely thixotropic material (from Mitchell, 1960).

The energy that drives thixotropy comes from the forces of attraction and repulsion between clay particles. A cohesive soil will exhibit thixotropy if the following two conditions exist (Mitchell 1960):

1. The net interparticle force balance is such that the clay particles will flocculate if given the chance.
2. The flocculation is not so strong, however, that the particles cannot be dispersed by applied shearing strains.

Mitchell (1960) has postulated that the mechanism of thixotropy may occur in the following manner:

1. When a thixotropic soil is compacted, a portion of the compactive shearing energy acts with the repulsive forces between platy particles to produce a more parallel arrangement. During compaction, the energy of interaction between particles is at a level commensurate with the externally applied compactive forces, and the adsorbed water layers and ions are distributed according to this high energy level. The net result is a structure similar to that shown schematically in Fig. 6A.125(*a*).
2. When compaction ceases, the externally applied energy drops to zero, and the net repulsive force decreases; that is, the attractive forces exceed the repulsive forces for the particular arrangement of particles and distribution of water, and the structure attempts to adjust itself to a lower energy condition. The dissipation of energy may be accompanied by changes in particle arrangements, adsorbed water structure, and distribution of ions. These structural changes are time-dependent, because physical movement of particles, water, and ions must take place. Since the process occurs at constant volume, the particle movements are probably small and of a rotational nature. A schematic diagram of the structure at some intermediate time after compaction is given in Fig. 6A.125(*b*).
3. After some time, the soil will achieve an equilibrium structure, as depicted in Fig. 6A.125(*c*). The time required to reach equilibrium is related to such factors as water content, particle size distribution, particle shape, the ease of displacement of adsorbed water molecules, pore water chemistry, and the magnitude of effective stress during aging.

Nalezny and Li (1967) suggested that thixotropic flocculation may result from Brownian motion that brings some clay particles close enough to flocculate.

Available evidence suggests that all compacted cohesive soils may undergo thixotropy under certain conditions. The following factors may affect the magnitude of the gain in strength and stiffness that occurs during thixotropy (Mitchell, 1960; Seed et al., 1960):

1. *As-compacted fabric*. Little or no thixotropic hardening occurs in soils with an initially flocculated fabric, such as those produced by compaction to dry-of-optimum saturation. Thixotropic hardening increases with increasing dispersion (all other factors being the same).
2. *Water content*. The influence of compaction water content on thixotropic hardening of a kneading-compacted silty clay after 6 days of aging is shown in Fig. 6A.126 (Mitchell 1960). The thixotropic strength ratio is defined as the strength of the aged soil to the strength of the as-compacted soil. Dry of optimum ($w < 17.7\%$), the fabric is essentially flocculated, and the thixotropic strength ratio is small. Wet of optimum, the thixotropic strength ratio increases with increasing water content up to about 28 to 29%, beyond which the strength ratio decreases with increasing water content. The disparity between the induced and equilibrium structures is apparently as great as it can be for this soil and method of compaction at a water content of about 28 to 29%. The decrease in thixotropic effect at higher water contents may be attributable to the natural dispersing tendency of the soil at these water contents.
3. *Magnitude of strain*. The thixotropic increase in strength is greater at low strains and decreases with increasing strain (Fig. 6A.126) and is attributable to progressive destruction during shearing of the thixotropically formed structure.

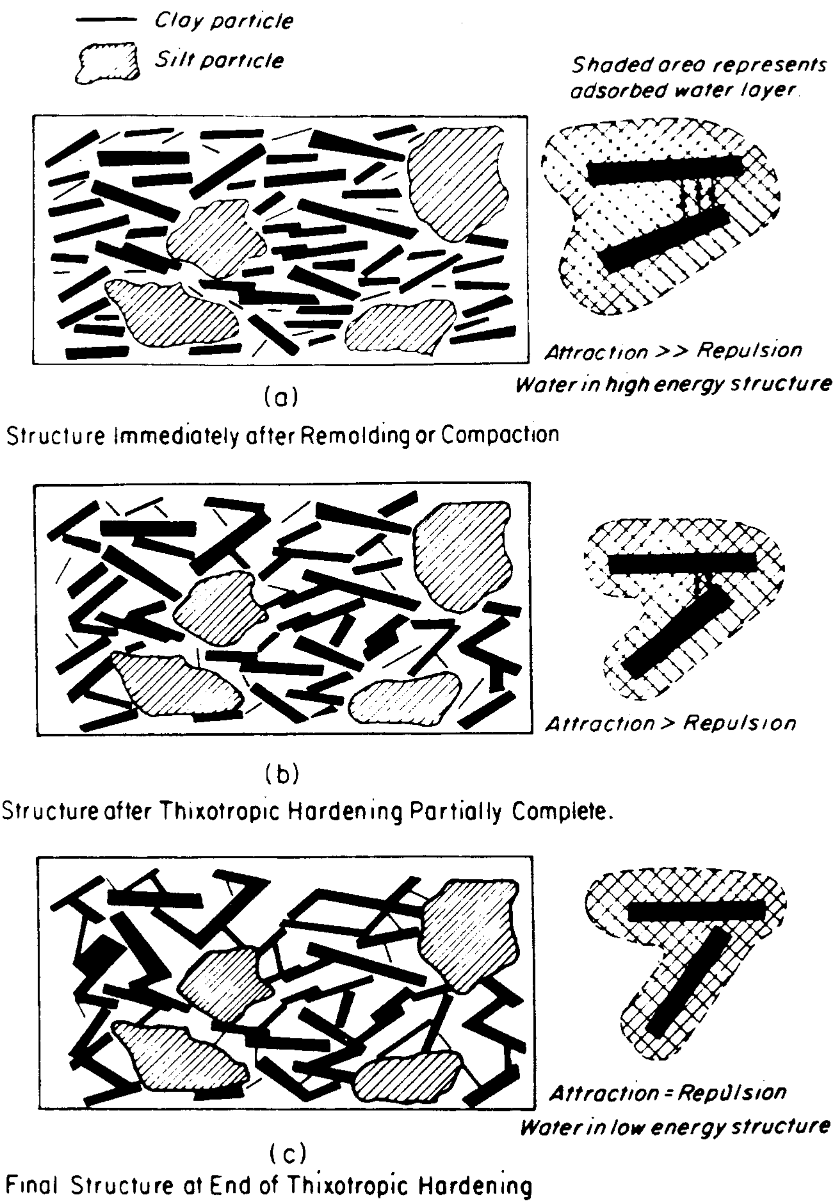


FIGURE 6A.125 Schematic diagram of thixotropic change in structure for a fine-grained soil (from Mitchell, 1960).

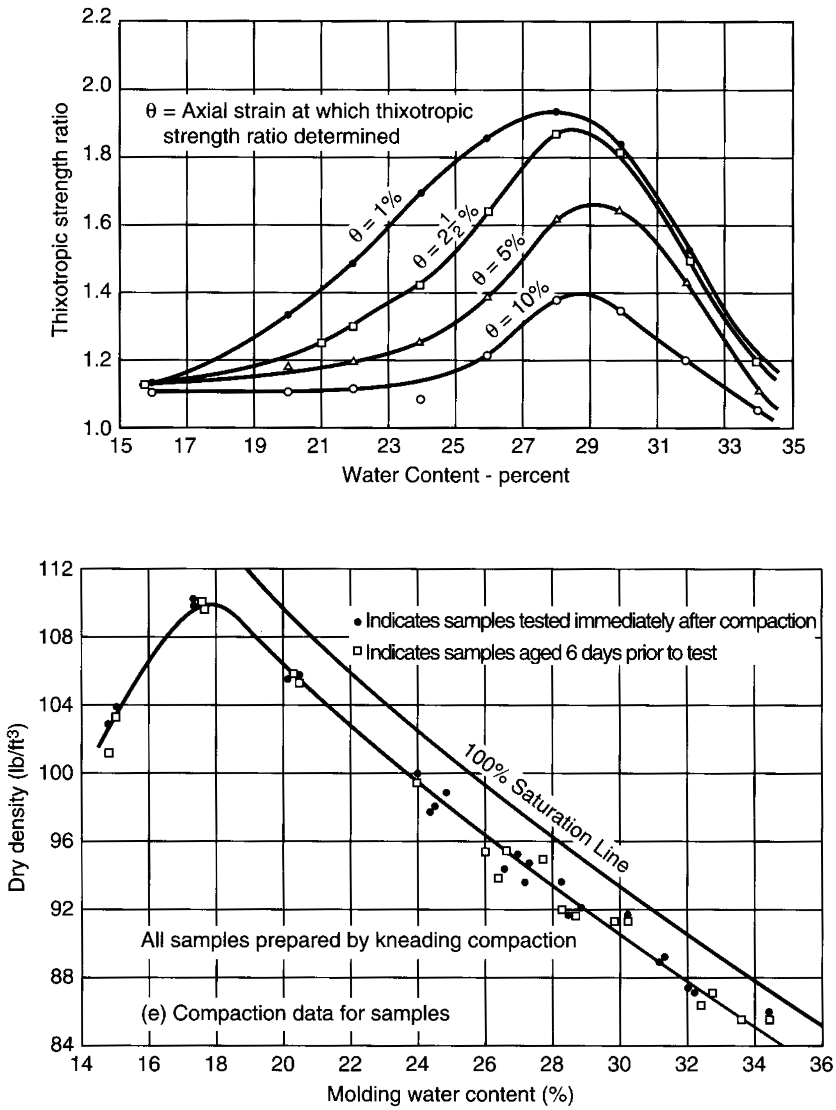


FIGURE 6A.126 Thixotropic strength ratio for a silty clay as a function of molding water content and axial strain (from Mitchell, 1960).

4. *Method of compaction*. The effect of method of compaction on thixotropic strength ratio for the same silty clay is shown in Fig. 6A.120. As expected, the kneading-compacted soil is more thixotropic than the statically compacted soil because of the greater dispersion-and hence greater tendency toward thixotropic flocculation—produced by kneading compaction.

Secondary Compression. Secondary compression, which is a creep-type rearrangement of particles into a slightly denser fabric, is closely associated with aging effects (Schmertmann 1991). The driving energy for secondary compression is supplied by the in situ effective stresses acting on the soil. Although some engineers believe that aging effects can be fully explained in terms of secondary compression, the magnitude of aging-induced changes in engineering behavior cannot be entirely attributed to the small changes in density that occur from secondary compression. Thus, secondary compression is one of several phenomena that contribute to aging effects in soils.

Particle Interference. The term *particle interference* is used to describe the tendency of aged soils to show increased dilation during shearing (see Fig. 6A.127). This increased dilatancy probably results from small slippages of particles during secondary compression, which produce additional particle-to-particle interlocking (Schmertmann, 1991).

Clay Dispersion and Internal Arching. The drained movements associated with secondary compression tend to disperse plate-shaped particles in cohesive soils, which leads to an increased basic frictional capability of the soil. Schmertmann (1991) has presented two possible mechanisms of internal arching by which this increased frictional resistance may occur:

1. As parts of the fabric of the soil stiffen owing to dispersion under drained conditions during aging, the stresses may arch to these stiffer parts.
2. The dispersive movements may occur primarily in the weaker, softer parts of the fabric and cause an arching stress transfer to the stiffer parts.

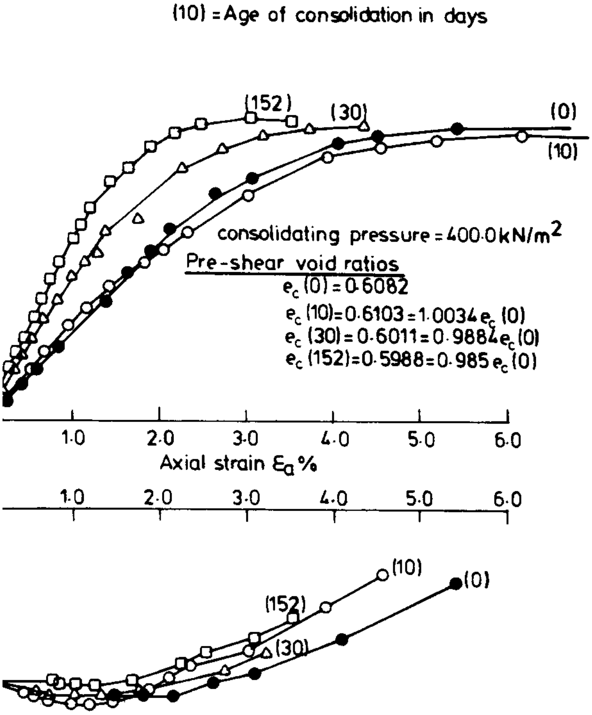


FIGURE 6A.127 Effect of aging on the triaxial stress-strain and volume change characteristics of Ham River sand (from Daramola, 1980).

In either case the arching is assumed to occur internally at the minifabric level, with the average effective stress remaining constant.

Cementation and Bonding. Although Schmertmann (1991) has shown that substantial increases in stiffness and strength can occur with aging in the absence of any measurable cementation, postcompaction cementation may play a significant role in aging effects for some soils, especially those containing carbonates, iron oxide, alumina, and organic matter. The decrease in swelling potential with aging shown in Fig. 6A.123 cannot be explained by any combination of factors that excludes cementation or some other form of bonding. Day (1994) suggested that the decrease in swelling potential for the silty clay he studied may be partially related to bonds that develop during aging. These bonds were manifested in the form of an aging-induced increase in effective cohesion intercept at zero effective stress, with c' ($\sigma'_v = 0$) increasing from zero for no aging to 45 psf (2.2 kPa) after 21 days of aging (see Day, 1992). The nature of this bonding was not determined. Nalezny and Li (1967) also reported experimental evidence that showed swelling potential decreased with aging in the highly expansive clay they studied, which they attributed to an increase in cross-linking of clay particles. This additional cross-linking—which results from Coulombic and van der Waals attractive forces that develop at edge-to-face contacts—is assumed to occur with aging as Brownian motion brings some clay particles close enough to flocculate. Note, however, that increasing flocculation in freshly compacted soils tends to increase the swelling potential (compare the results for the kneading and statically compacted specimens shown in Fig 6A.100). Therefore, if the aging-induced decrease in swelling potential is produced entirely by cross-linking, the reduction in swelling potential caused by the cross-linking must be greater than the increase in swelling potential resulting from increased flocculation. Whatever its nature, some form of bonding occurs in cohesive soils with aging that contributes not only to a reduction in swelling potential but also to the other aging-induced changes in engineering behavior.

Summary. Aging effects have been reported in virtually all types of remolded and compacted soils. Aging effects in sands appear to result from increased density and particle interlocking as a consequence of secondary compression. Cementation may also be a contributing factor in some cases. In clays, aging effects may also be associated with thixotropy, dispersion, and some form of bonding. Schmertmann (1991) estimates that for most soils, the aging-induced improvement in many key soil properties is on the order of 50 to 100% during engineering times. Unfortunately, the mechanisms by which these improvements occur are not completely understood, and the beneficial effects of these improvements are seldom considered in engineering practice. However, because the potential degree of improvement is significant, a substantial savings in foundation costs may be realized on some projects involving compaction of bearing soils by performing laboratory and field tests to determine the extent of this improvement for important properties of the borrow soils or in situ soils to be compacted.

6A.3.5 Quality Control and Assurance

To ensure that a compacted soil has the desired engineering characteristics, a two-step quality control and assurance process is commonly employed that consists of compaction specifications and postcompaction testing of the compacted soil. The compaction specifications are usually written by a geotechnical engineer and become part of the legal construction documents. The primary purpose of the compaction specifications is to provide guidelines that assist the compaction contractor in achieving the desired engineering product. The postcompaction testing program is used to verify that compacted soil with appropriate engineering properties has been constructed.

6A.3.5.1 Compaction Specifications

Most compaction specifications fall into one of two general categories—*end product specifications* or *method specifications*. In end product specifications, the desired characteristics of the compacted soil are specified, and the compaction contractor may use any equipment and procedures to produce compacted soil that meets the requirements. In method specifications, the methods and equipment that the contractor must use to compact the soil are prescribed. In some instances, combination end product and method specifications are provided.

End Product Specifications. The desired engineering characteristics for a compacted soil depend significantly on the use to which the soil will be put and therefore may vary considerably from project to project and at different locations within the same project. For the bearing soils beneath a building, the most important properties (in decreasing order of importance for most projects) are (*a*) volume change characteristics, (*b*) strength, and (*c*) permeability. Section 6A.3.4 provided a detailed discussion of these characteristics for compacted soils.

In many end product specifications, either a criterion for dry density or criteria for density and compaction water content are furnished. These specifications are based on the assumption that the desired engineering characteristics—usually low compressibility and high strength—are related to water content and/or density. Density and water content specifications are commonly used primarily for economic reasons, that is, it is much faster and cheaper to verify that density and water content specifications have been met than to verify, for example, that compressibility and strength specifications have been met. However, the engineer writing the specifications should remember that in some instances the desired characteristics may not be achieved even if the density and water content criteria are met.

Density specifications Dry density specifications are generally given in terms of *relative compaction* (or *percent compaction*), R, defined as

$$R(\%) = \frac{\gamma_d}{\gamma_{d\max}} \cdot 100 \tag{6A.41}$$

where γ_d = dry density of the compacted soil
$\gamma_{d\max}$ = "maximum" dry density of the same soil

Relative compaction specifications are used rather than density specifications in most fills because of the inherent variability of the borrow soils, for which small changes in size and gradation may result in significant changes in absolute values of maximum dry density and optimum water content (Hilf, 1991). The test method from which $\gamma_{d\max}$ is to be determined should be specified because the value of $\gamma_{d\max}$ for any soil depends on the method of compaction and compactive effort, as discussed in Sec. 6A.3.2. The most common tests used to determine $\gamma_{d\max}$ are the standard and modified Proctor tests (ASTM D698 and D1557, or equivalent). The same dry density specifications can be given using $\gamma_{d\max}$ from either test, and in the United States the preferred test varies from region to region. Poulos (1988) recommends using the modified Proctor test because the test errors are likely to be smaller than for the standard Proctor test.

The importance of specifying the method from which $\gamma_{d\max}$ is to be determined cannot be overemphasized. If the required method is not given in the specifications, the compaction contractor can legally use any reasonable method she or he wishes. The author knows of several projects in which the method was not specified, and the contractor assumed standard Proctor $\gamma_{d\max}$ (to the contractor's advantage) when the specifying engineer intended modified Proctor $\gamma_{d\max}$ In these cases, either the contractor was paid more than his or her bid price to compact the soil according to modified Proctor $\gamma_{d\max}$, or the soil was compacted using standard Proctor $\gamma_{d\max}$ and the compacted soil did not meet the desired criteria (which resulted in either expensive litigation or removal and replacement of the unacceptable material). The net result in each case was that more money than necessary was paid to compact the soil, an expense that could have been avoided if the engineer writing the compaction specifications had simply specified the method for determining $\gamma_{d\max}$ rather than assume that everyone would understand what was meant.

Density requirements for clean granular soils are sometimes given in terms of *relative density* (D_r) rather than relative compaction. Relative density can be calculated in terms of either void ratios or dry densities using one of the following equations:

$$D_r(\%) = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \cdot 100 \tag{6A.42a}$$

$$D_r(\%) = \frac{\gamma_{d\max}}{\gamma_d} \cdot \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \cdot 100 \tag{6A.42b}$$

where e = void ratio of the compacted soil
$e_{\min}$ = "minimum" void ratio for the same soil
$e_{\max}$ = "maximum" void ratio for the same soil
$\gamma_{d\min}$ = "minimum" dry density for the same soil
$\gamma_{d\max}$ = "maximum" dry density for the same soil

$e_{\min}$ and $e_{\max}$ (or $\gamma_{d\min}$ and $\gamma_{d\max}$) are commonly determined from laboratory tests as described in Sec. 6A.3.3.4. and the method on which they are to be based should be specified (usually ASTM D4253 and D4254). Nominal values of relative density vary from 0% for a soil in its "loosest" condition to 100% for a soil in its "densest" state. However, note that, because $\gamma_{d\min}$ and $\gamma_{d\max}$ are generally based on laboratory tests that do not exactly model field compaction, the actual values of relative density for naturally deposited and compacted soils may be less than 0 or greater than 100%. Compacted soils never have D_r less than 0% but may have D_r greater than 100% depending on the method of compaction and the procedure used to determine $e_{\min}$ or $\gamma_{d\max}$ (refer also to Sec. 2A).

A statistical investigation by Lee and Singh (1971) of data found in the literature for 47 granular soils has shown that values for relative density and relative compaction for any soil are related. By assuming that the values of $\gamma_{d\max}$ in Eqs. (6A.41) and (6A.42) are the same, they obtained the following equation relating D_r to R:

$$R = \frac{R_0}{1 - D_r(1 - R_0)} \tag{6A.43}$$

where

$$R_0 = \frac{\gamma_{d\min}}{\gamma_{d\max}} \tag{6A.44}$$

The term R_0 has the physical meaning of being the relative compaction at zero relative density. The mean value of R_0 for the soils studied was 81.8. Inserting this value of R_0 in Eq. (6A.43) yields

$$R = \frac{0.818}{1 - 0.182D_r} \tag{6A.45}$$

Sixty-seven percent of the data fell within ± 1 standard deviation. Unfortunately, the values for $\gamma_{d\max}$ and $\gamma_{d\min}$ from which the mean value of R_0 was calculated were obtained using a wide variety of test methods, and frequently the procedures used were not adequately described in the literature. Thus, Eq. (6A.45) is of limited practical use because the bases for calculating R and D_r are not standardized to particular methods.

According to the obsolete standard ASTM D2049 (replaced by D4253 and D4254), relative density rather than relative compaction should be used if the soil contains less than 12% fines. Lee and Singh (1971) recommend using relative density for granular soils because small deviations in field density appear as large numbers (a change in relative compaction of one percentage point is approximately equivalent to a change in relative density of five percentage points) and thus tend to convey the severity of the differences to the compaction contractor. However, there appears to be a growing trend among engineers toward using relative compaction for all types of soils. For example, Poulos

(1988) recommends against the use of relative density for compaction control of granular soils for the following reasons:

1. Both $\gamma_{d\min}$ and $\gamma_{d\max}$ are functions of soil composition (gradation, particle shape, mineralogy, etc.) and are linearly related (with some scatter) for most granular soils. Therefore, only one of the two factors is needed for controlling the compaction of granular soils.
2. $\gamma_{d\min}$ is not of great interest in engineering practice.
3. Relative compaction appears to be a slightly better index of engineering properties than relative density.

In most density specifications, only a minimum acceptable relative compaction or relative density is prescribed, which results in an allowable moisture-density range as shown in Fig. 6A.128. It may also be prudent in some instances to include a maximum acceptable relative compaction specification, as indicated in Fig. 6A.129. Typical values of relative compaction specification for different applications are summarized in Table 6A.9 and are provided only as general guidelines. Actual specifications should be based on the desired engineering characteristics and economics.

Moisture condition specifications The moisture condition of the soil during and after compaction determines, to a great extent, the maximum density that can be achieved with a given compaction roller, as well as the engineering characteristics of the as-compacted soil. The moisture condition of soils can be described in terms of either water content or degree of saturation. In many projects, therefore, it is imperative that the compaction specifications include some requirements as to the moisture condition of the soil during compaction. The moisture condition can be specified in terms of water content, degree of saturation, or both. In addition, because of the potential for volume changes owing to changes in the availability of free moisture, especially in compacted cohesive soils, it may be necessary to specify methods for preventing excessive drying or wetting of the compacted soil either prior to compaction of the next lift or after the compaction process is completed.

Compaction moisture condition is nearly always specified as a water content range, usually referenced to either standard or modified Proctor optimum water content. Typical water content speci-

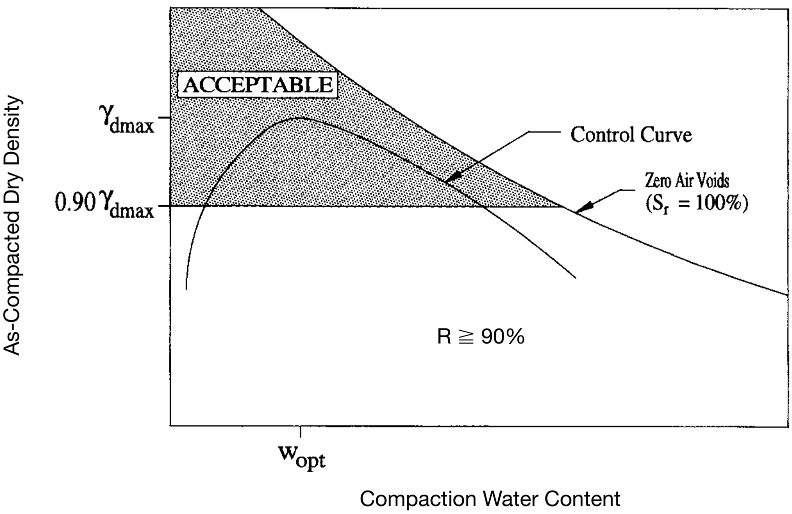


FIGURE 6A.128 Acceptable moisture-density region for specified minimum relative compaction.

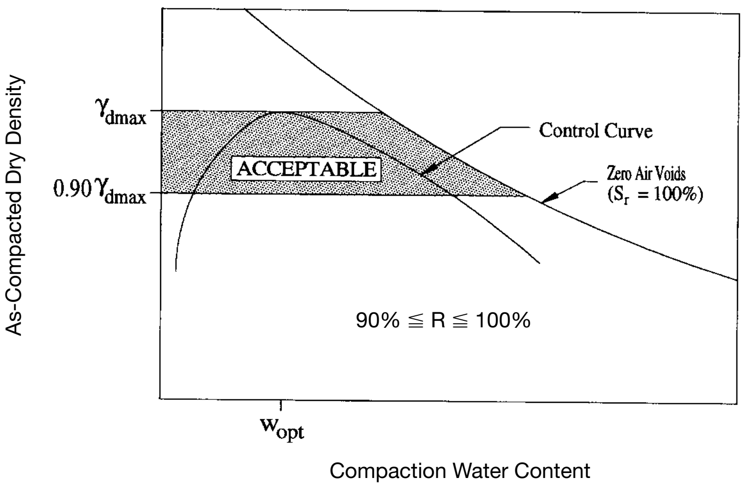


FIGURE 6A.129 Acceptable moisture-density region for specified minimum and maximum relative compaction.

fications for various applications are provided in Table 6A.9. When water content specifications are combined with relative compaction specifications, the resultant acceptable moisture-density zone is depicted in Fig. 6A.130. However, because of the confusion regarding optimum water content and optimum moisture condition (line of optimums) discussed in Sec. 6A.3.2, moisture condition specifications given in terms of water content only may not result in compacted soil with the desired engineering properties. For example, the permeability of a compacted clay may be several orders of magnitude lower when compacted wet of optimum than when compacted dry of optimum (see Fig. 6A.109). Optimum moisture condition refers to the line of optimums rather than one value of optimum water content referenced to some standard test. An example is given in Fig. 6A.131 showing the permeabilities that would be achieved in a compacted silty clay by specifying moisture condition as a water content range (Benson and Daniel, 1990). Because the specified water content range is above "optimum" water content, some engineers would believe that the given specifications would result in permeabilities less than the specified maximum value of 1×10^{-7} cm/s (3×10^{-4} ft/day). As can be seen in Fig. 6A.131, the actual permeabilities within some portions of the specified moisture-density zone are more than 100 times the desired maximum value.

In most compacted soils it is desirable to control more than one property. For example, in a compacted clay liner, low permeability is the dominant consideration, but moderate strength and low shrinkage potential are also desirable characteristics. An acceptable moisture-density range that would result in a compacted soil with all the desired characteristics can be established by delineating acceptable zones separately for each characteristic, as illustrated in Fig. 6A.110. Moisture-density specifications that meet the desired objectives in this type of situation can be given in terms of relative compaction and degree of saturation, as shown in Fig. 6A.132. Since many compaction personnel are familiar with the concept of water content but not degree of saturation, a water content range can also be specified to alleviate somewhat any concern they may have because of their unfamiliarity with degree of saturation, but the specifications must be clearly written to emphasize that the requirement for degree of saturation must also be met. It may also be prudent when working with cohesive soils to specify a maximum water content so that the soil remains dry enough that the hauling, spreading, and compaction equipment have sufficient traction to operate efficiently. The engineering properties of many compacted cohesive soils are better correlated with degree of satu-

TABLE 6A.9 Typical Density, Water Content, and Lift Thickness Specifications*

Use for compacted soil	Relative compaction† standard	Relative compaction† modified	Water content range†‡	Lift thickness, in (mm)
Bearing soils for structures	98–100	92–95	–2 to +2	6–10 (152–254)
Lining for canal or reservoir	95	90	–2 to +2	6 (152)
Low earth dam	95	90	–1 to +3	6–12 (152–305)
High earth dam	98	93	–1 to +2	6–12 (152–305)
Highway or airfield	95	90	–2 to +2	6–12 (152–305)
Backfill surrounding structures	95–98	90–93	–2 to +2	6–10 (152–254)
Backfill in pipe or utility trenches	95–98	90–93	–2 to +2	6–8 (152–203)
Drainage blanket or fitter	98	93	Thoroughly wetted	10 (254)
Subgrade of excavation for structure	98	93	–1 to +2	—
Rock fill	—	—	Thoroughly wetted	24–36 (610–914)

*After NAVDOCKS (1961).
†All values for relative compaction and water content in percent.
‡Referenced to optimum water content (either standard or modified).

ration than water content. Hence, the desired engineering behavior frequently will be better achieved by specifying the as-compacted moisture condition in terms of degree of saturation rather than compaction water content.

Many compaction contractors are resistant to specifications that include degree of saturation because it cannot be measured directly but must be calculated indirectly from phase relations using an equation of the following form:

$$S_r = \frac{w}{(\gamma_w/\gamma_d) - (1/G_s)} \tag{6A.46}$$

Therefore, to calculate degree of saturation, γ_d and w must be measured, and G_s either estimated or determined from laboratory tests. Since γ_d and w are normally measured during the construction control process anyway, the only additional factor needed is G_s, which can be estimated or measured fairly easily. Because each of these three terms has an inherent variability associated with measuring it, the calculated value of S_r has a greater variability associated with it than any of the three terms from which it was calculated. This inherent variability in calculated values for degree of saturation must be considered when performing quality control and assurance checks on the compacted soil (Schmertmann, 1989). For example, this variability will result in a certain percentage of calculated values for S_r plotting above the zero air voids line, indicating that $S_r > 100\%$, which is physically impossible. Routinely rejecting moisture-density tests simply because they plot above the zero air

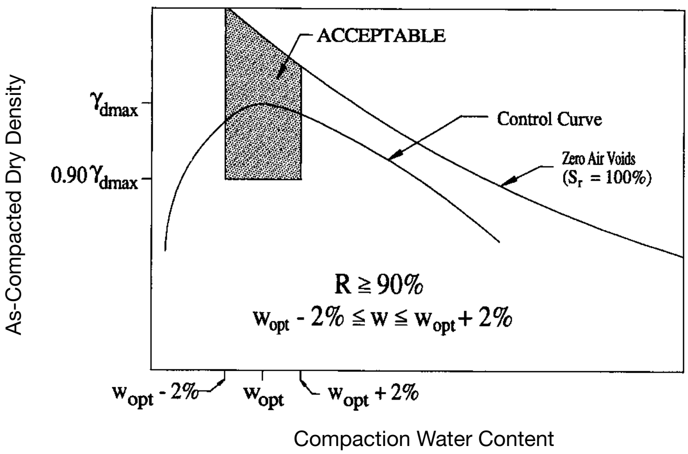


FIGURE 6A.130 Acceptable moisture-density region for specified minimum relative compaction and water content range.

voids line may result in a fill that has a higher S_r than intended and the fill may therefore be weaker, more compressible, and more susceptible to developing high pore water pressures during construction. A detailed procedure for using test data with S_r > 100% is given by Schmertmann (1989) (refer also to Sec. 2A).

Other End Product Specifications Although end product specifications are generally given in terms of dry density and water content, it is also possible—and sometimes more effective—to specify other criteria for quality control and assurance of compacted soil. The types of tests that may be used include the following:

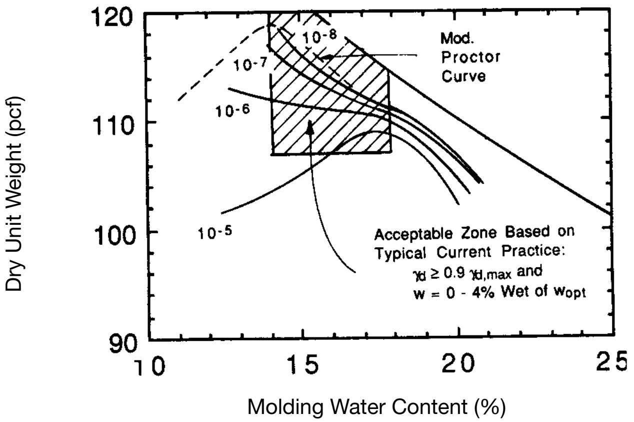


FIGURE 6A.131 Acceptable moisture-density zone based on typical current practice superimposed on isograms of coefficient of permeability for a silty clay prepared by kneading compaction (from Daniel and Benson, 1990).

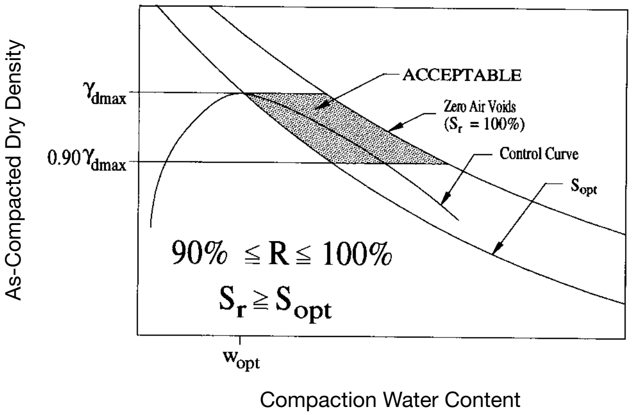


FIGURE 6A.132 Acceptable moisture-density region for specified minimum and maximum relative compaction and minimum degree of saturation.

- Proctor penetrometer (ASTM D1558)
- Plate load test (ASTM D1194, D1195, D1196)
- California bearing ratio (CBR) test (ASTM D4429)
- Dynamic penetration test
- Unconfined compression test (ASTM D2166)
- One-dimensional compression test (ASTM D2435 and D4186)
- Triaxial compression test (ASTM D2850 and D4767)
- Standard penetration test (ASTM D1586)
- Cone penetration test (ASTM D3441)
- Sealed double ring infiltration test (ASTM D5093)
- One-dimensional swelling/collapse test (ASTM D4546)
- Laboratory permeability test (ASTM D2434 and D5084)
- Suction measured using tensiometer

Although many of these tests are more expensive to conduct than density/water content tests, they frequently will provide better correlations with the desired engineering characteristics. For example, if settlement of a building is the primary consideration, results from plate load tests will give a better indication of the compressibility of the compacted soil than density/water content tests. Hausmann (1990) has reported that in Europe it is not uncommon to have acceptability of compacted soil specified in terms of the reload modulus determined by the plate load test. For compacted clay liners for waste containment facilities, it is often required that a sealed double ring infiltration test be performed on a field test pad constructed using the same equipment and procedures to be used on the actual liner. Laboratory permeability tests on samples taken from the actual liner are also typically required. Therefore, it is the responsibility of the engineer writing the specifications to ensure that the quality control/assurance testing procedures will result in a compacted soil that has the desired engineering characteristics. This cannot always be accomplished with density/water content criteria only, and in those instances the engineer should specify supplemental testing criteria that will assist in reliably assessing the appropriate engineering characteristics of the compacted soil.

Testing variability and statistical specifications In traditional compaction specifications wherein only a minimum acceptable value or a range of acceptable values is specified for each parameter (density, water content, and so on), the acceptability of any compacted lift or portion of a lift is judged strictly by whether all measured values of that parameter fall within the acceptable range. If not, that lift or portion of the lift is deemed unsuitable and must be either recompacted in place to a greater density (if the water content is acceptable) or removed, moisture conditioned, and recompacted to meet the specifications. However, these traditional specifications do not consider the inherent variability of the soil and the variation between measured values and true values. Therefore, in recent years it has become more common to base compaction specifications and quality control on statistical considerations of soil and testing variability.

Compacted soil masses, whether compacted fills or natural soil deposits compacted at their original location, often vary considerably in terms of their composition and engineering characteristics. In addition, all tests conducted on the field-compacted soil or on samples taken therefrom all have some inherent inaccuracies, that is, the measured values will always vary from the "true" value by some amount. The more accurate the test, the less deviation between measured and true values. The accuracy of a particular test depends on random errors and systematic errors related to the particular equipment being used and the operator (Hausmann, 1990). Because of the many variables associated in testing compacted soils, the results of water content and density tests tend to be normally distributed (Turnbull et al., 1966; Hausmann, 1990). As an example, the distribution of water content variation from optimum and dry density variation from maximum dry density for the impervious fill of Ferrells Bridge dam are shown in Fig. 6A.133.

In a statistical approach to specification and control, it is recognized that measured values of density and water content will vary from the desired values. If the measured values are normally distributed about the desired value, every undervalue is balanced by an overvalue, and if the stan-

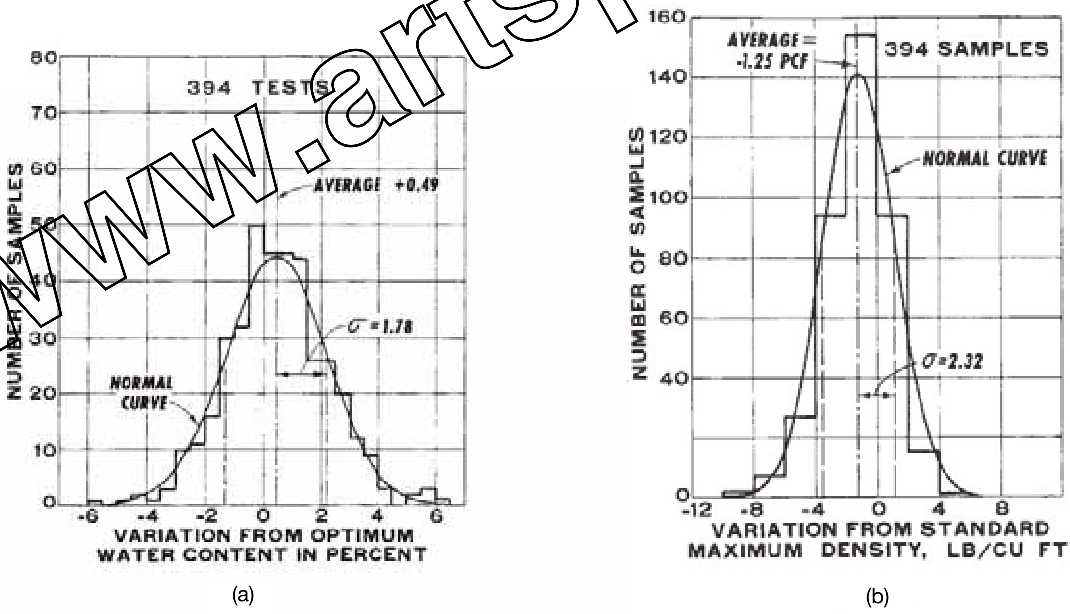


FIGURE 6A.133 Variation in (a) compaction water content and (b) as-compacted dry density for the impervious fill of Ferrells Bridge dam (from Turnbull et al., 1966).

dard deviation is relatively small, the overall behavior of the compacted soil is not likely to be much different than if the entire soil had been compacted to the desired value. Therefore, some deviation from the desired value may be acceptable, so long as there is no large, contiguous zone of material for which the minimum acceptable value is not met.

The following abbreviated specifications for Cutter Dam illustrate the use of statistical control of compaction based on frequency distribution concepts (from Hilf, 1991). Note that these specifications are referenced to optimum water content and maximum dry density from the modified Proctor test.

1. *Moisture control*. The moisture content of the earthfill prior to and during compaction shall be distributed uniformly throughout each layer of the material. In addition, the moisture content of the compacted earthfill determined by testing shall be within the following limits.
   a. If $w < w_{opt} - 3.5\%$ or $w > w_{opt} + 1.0\%$, the material shall be removed or reworked until the water content is between these limits.
   b. No more than 20% of the samples shall be drier than $w_{opt} - 3\%$, and no more than 20% of the samples shall be wetter than $w_{opt} + 0.5\%$.
   c. The average water content of all accepted embankment material shall be between $w_{opt} - 1.0\%$ and $w_{opt} - 0.5\%$.
   d. As far as practicable, the material shall be brought to the proper water content in the borrow pit before excavation. Supplementary water, if required, shall be added to the material by sprinkling on the earthfill on the embankment, and each layer of the earthfill shall be conditioned so that the moisture is uniform throughout the layer.
2. *Density control*. The dry density of the compacted material shall conform to the following limits:
   a. Material will be rejected if $R < 96.0\%$. Rejected material shall be rerolled until $R \geq 96.0\%$.
   b. No more than 20% of the samples shall have $R < 97.0\%$.
   c. The average density of all material shall be greater than or equal to $R = 100\%$.

Method Specifications. Method specifications typically prescribe the type of equipment to be used, the thickness of the lift to be compacted, and the number of passes of the equipment per lift (Hilf, 1991). Moisture requirements and a maximum size of material may also be specified. The most important advantages and disadvantages of method specifications are summarized as follows (Holtz and Kovacs, 1981):

1. An expensive field testing program generally must be conducted to determine the most efficient and economical methods and equipment for obtaining a compacted soil with the desired characteristics. Method specifications, therefore, are normally used only on major compaction projects such as earth dams.
2. A major part of the uncertainty associated with compaction will be eliminated for the contractor. Therefore, the contractor should be able to estimate the construction costs more accurately, and a substantial savings in earthwork costs should be realized.
3. The owner or owner's engineer has the major portion of the responsibility for the quality of the earthwork rather than the contractor. If the contractor follows the specifications but the compacted material does not meet the desired characteristics, the contractor will be paid extra for any work needed to make the material meet the requirements.

6A.3.5.2 Postcompaction Testing and Verification

Moisture-Density Tests. The most common type of postcompaction testing and verification program consists of density and water content tests conducted at random locations within the compacted material. In many fills, at least several tests are conducted on each lift. The frequency with which moisture-density tests are conducted in compacted fills varies considerably, with typical values as follows (after Hausmann 1990 and Hilf 1991):

Type of fill	Volume of fill per test, yd^3 or m^3
Embankment	500–4000
Impermeable liner	200–1000
Subgrade	500–1500
Base course	500–1000
Backfill	100–500

Additional tests should be conducted when the borrow material changes significantly. If the surface of the most recent lift is uneven (e.g., sheepsfoot or tamping foot roller) or some densification may occur during compaction of overlying lifts (e.g., sand), testing should be done one or two lifts below the top lift. To avoid potential bias of testing personnel, the locations for moisture-density testing should be randomly selected (that is, each location should have the same chance for being chosen). This can be accomplished using a table of random numbers such as that shown in Table 6A.10 or by generating random numbers on a computer or handheld calculator. The following example is given to illustrate the procedure for randomly selecting locations for testing (from Sherman et al., 1967):

> A compacted lift is 30,000 ft (9,144 m) long and 26 ft (7.92 m) wide, and 50 moisture-density tests will be conducted. Starting at any point in the random number table and proceeding up or down (but not omitting any numbers), 50 *pairs* of numbers are read. For example, starting at the top of column 4, the following pairs of numbers are read: (.732, .721); (.153, .508); (.009, .420); . . . (.698, .539). The first or A number in each pair is multiplied by the length, and the second or *B* number in each pair is multiplied by the width to establish the locations for testing. Using the first pair of numbers, a moisture-density test would be conducted at (21,960 ft, 19 ft) or (6693 m, 5.7 m). The locations of the 50 tests are then plotted and numbered in the order in which they will be performed. Should two locations be so close together that they both could not be tested properly, the second one is discarded and the next pair of numbers in the table is substituted.

Sand cone and rubber balloon tests Moisture-density tests can be either destructive (fill material is excavated and removed) or nondestructive (density and water content are determined indirectly). The most common destructive tests are the sand cone test (ASTM D1556) and the rubber balloon method (ASTM D2167) (Fig. 6A.134). The following steps are used in these tests (after Holtz and Kovacs, 1981):

1. A hole is excavated in the compacted fill at the desired elevation. The size of the hole depends on the maximum size of included particle in the excavated material (ASTM D1556 and D2167):

Sand cone				Rubber balloon			
Maximum particle size		Minimum test hole volume		Maximum particle size		Minimum test hole volume	
in	mm	ft^3	cm^3	in	mm	ft^3	cm^3
½	12.5	0.05	1420	No.4 sieve	4.75	0.04	1130
1	25	0.075	2120	¾	19.0	0.06	1700
2	50	0.1	2830	1½	37.5	0.10	2840

The mass or weight of the excavated material is determined in the field using a scale or balance.

2. The water content is determined from either the total excavated material or a representative sample taken from the excavated material. The standard water content procedure consists of oven drying the sample at 110°C (230°F) (ASTM D2216). If the total excavated material is not used, requirements for minimum mass of the representative sample are given in ASTM D2216. The material is dried in the oven until a constant mass is reached. The time required to obtain con-

TABLE 6A.10 Random Numbers for Selecting Locations of Field Testing (from Sherman et al., 1967)

1		2		3		4		5	
A	B	A	B	A	B	A	B	A	B
.576	.730	.430	.754	.271	.870	.732	.721	.998	.239
.892	.948	.858	.025	.935	.114	.153	.508	.749	.291
.669	.726	.501	.402	.231	.505	.009	.420	.517	.858
.609	.482	.809	.140	.396	.025	.937	.310	.253	.761
.971	.824	.902	.470	.997	.392	.892	.957	.640	.463
.053	.899	.554	.627	.427	.760	.470	.040	.904	.993
.810	.159	.225	.163	.549	.405	.285	.542	.231	.919
.081	.277	.035	.039	.860	.507	.081	.538	.986	.501
.982	.468	.334	.921	.690	.806	.879	.414	.106	.031
.095	.801	.576	.417	.251	.884	.522	.235	.398	.222
.509	.025	.794	.850	.917	.887	.751	.608	.698	.683
.371	.059	.164	.838	.289	.169	.569	.977	.796	.996
.165	.996	.356	.375	.654	.979	.815	.592	.348	.743
.477	.535	.137	.155	.767	.187	.579	.787	.358	.595
.788	.101	.434	.638	.021	.894	.324	.871	.698	.539
.566	.815	.622	.548	.947	.169	.817	.472	.864	.466
.901	.342	.873	.964	.942	.985	.123	.086	.335	.212
.470	.682	.412	.064	.150	.962	.925	.355	.909	.019
.068	.242	.667	.356	.195	.313	.396	.460	.740	.247
.874	.420	.127	.284	.448	.215	.833	.652	.601	.326
.897	.877	.209	.862	.428	.117	.100	.259	.425	.284
.875	.969	.109	.843	.759	.239	.890	.317	.428	.802
.190	.696	.757	.283	.666	.491	.523	.665	.919	.146
.341	.688	.587	.908	.865	.333	.928	.404	.892	.696
.846	.355	.831	.218	.945	.364	.673	.305	.195	.887
.882	.227	.552	.077	.454	.731	.716	.265	.058	.075
.464	.658	.629	.269	.069	.998	.917	.217	.220	.659
.123	.791	.503	.447	.659	.463	.994	.307	.631	.422
.116	.120	.721	.137	.263	.176	.798	.879	.432	.391
.836	.206	.914	.574	.870	.390	.104	.755	.082	.939
.636	.195	.614	.486	.629	.663	.619	.007	.296	.456
.630	.673	.665	.666	.399	.592	.441	.649	.270	.612
.804	.112	.331	.606	.551	.928	.830	.841	.602	.183
.360	.193	.181	.399	.564	.772	.890	.062	.919	.875
.183	.651	.157	.150	.800	.875	.205	.446	.648	.685

stant mass depends on such factors as type of material, size of specimen, and oven type and capacity. As a general rule of thumb, sands will dry to constant mass in about 4 h, whereas highly plastic soils may require 16 h or more. For most compaction projects, even 4 h is unacceptable for proper control of compacted fills, and one of the following faster but approximate methods are commonly used (after Hilf, 1991):

a. The Proctor penetration needle, used to determine the field-compacted water content by comparing the penetration resistance of the field-compacted soil with a calibration curve of penetration resistance versus water content from tests on Proctor specimens. (Additional details are given below.)

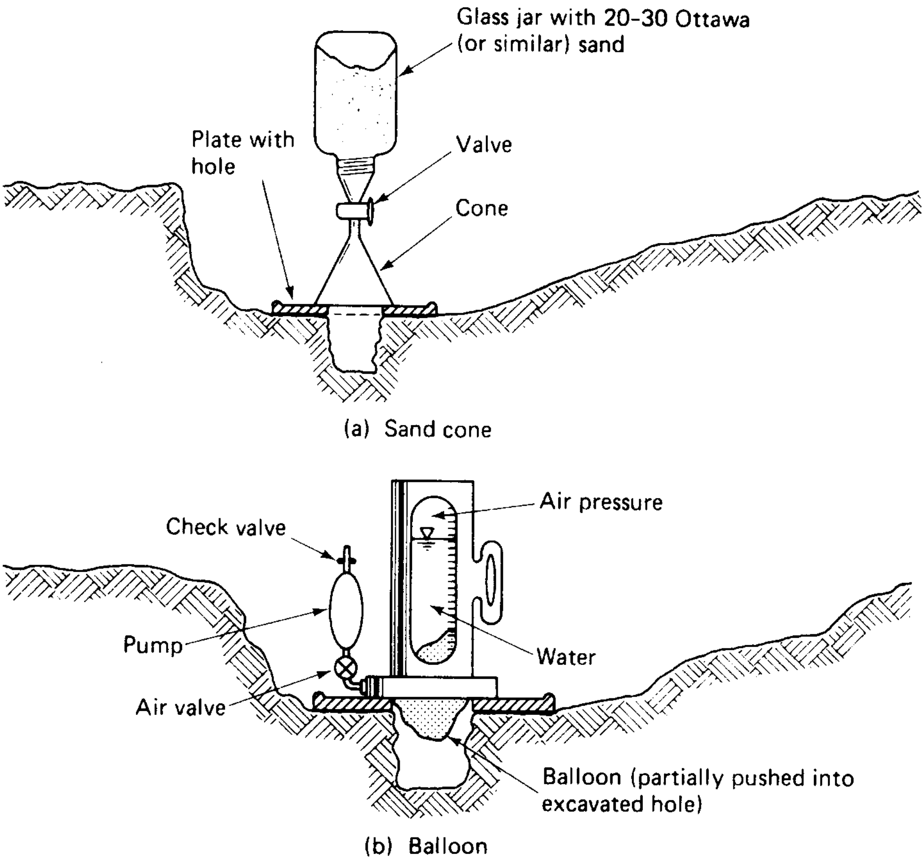


FIGURE 6A.134 Schematic illustrations of sand cone and rubber balloon tests (from Holtz and Kovacs, 1981).

b. Alcohol burning method (HRB, 1952).
c. Bouyoucos alcohol method using a hydrometer (Bouyoucos 1931).
d. A moisture meter using calcium carbide to generate acetylene in a closed container connected to a pressure gauge (Reinhold 1955; ASTM D4944).
e. Drying the soil in a container using any of a variety of types of direct heating equipment, such as a hotplate, stove, or blowtorch (ASTM D4959).
f. Drying the soil in a microwave oven (ASTM D4643).

If one or more of these approximate methods are used, they must somehow be correlated with the oven-drying method. Usually this involves testing duplicate samples at selected intervals using the oven-drying method, and comparing the result with the result from the approximate method being used.

3. The original volume of the excavated material is determined by measuring the volume of the hole.
   a. *Sand cone.* Clean, dry, uniform sand of known "loose" density is allowed to flow through a cone-shaped pouring device into the hole. The volume of the hole is calculated from the

weight of the sand in the hole (by weighing the sand container before and after placing the sand) and the calibrated loose dry density of the sand. Care must be taken to ensure that the loose sand is not densified by vibrations from any source, such as construction vehicles, nearby pile driving or blasting operations, or personnel walking near the hole. In soft soils, volume changes of the hole may occur by deformations induced by personnel (including those performing the test) who walk or stand close to the hole. In dry, cohesionless soils, raveling of the sides of the hole may occur by personnel or vibrations.

b. *Rubber balloon*. The volume of the hole is determined by expanding the balloon directly into the hole and reading the change in volume of the water in the apparatus directly from the calibration marks. As with the sand cone test, care must be exercised to avoid deformation of the hole in soft soils or raveling of the sides of the hole in loose, dry, cohesionless soils.

4. Compute the total density and dry density of the compacted soil (γ_{field} and $\gamma_{d\text{-field}}$) from the total mass or weight of material excavated from the hole, the volume of the hole, and the water content.
5. Compare $\gamma_{d\text{-field}}$ with $\gamma_{d\max}$ and calculate relative compaction (R).

The following destructive methods are also sometimes used.

Drive-cylinder method (ASTM D2937) This method involves driving a short, thin-walled cylinder into the soil using a special drop-hammer sampler. The cylinder is dug from the ground, excess soil is removed from the sides of the cylinder, and the ends of the sample are trimmed flush with the ends of the cylinder using a sharpened straightedge. The volume of the sample is equal to the inner volume of the cylinder. The total mass is determined by separately weighing the cylinder only (usually before sampling) and the cylinder with the trimmed sample. Water content tests are also performed, from which the dry density is calculated. This method is not appropriate for organic soils, very hard soils, soils of low plasticity that will not readily stay in the cylinder, or soils which contain appreciable amounts of coarse material.

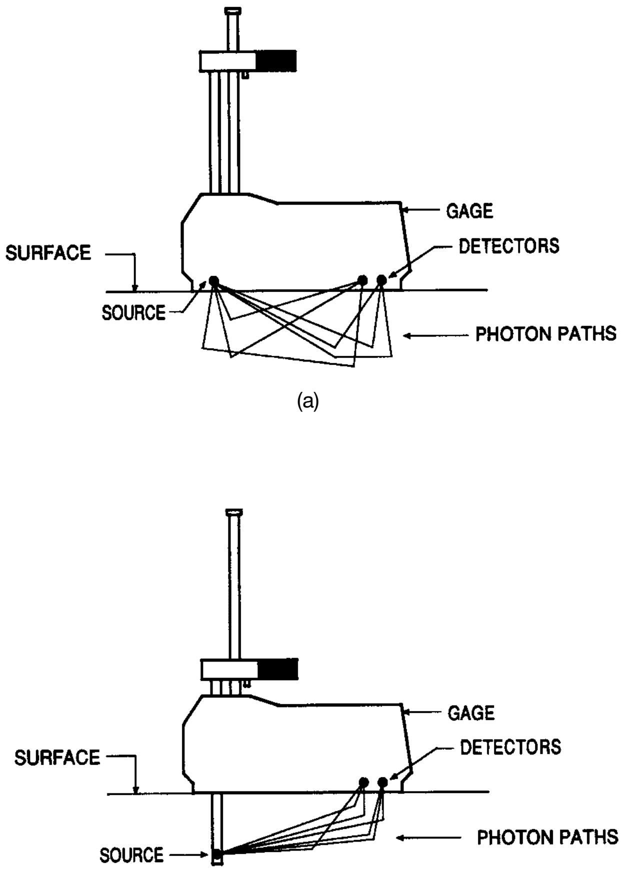
Sleeve method (ASTM D4564) The sleeve method is used primarily for soils that are predominantly fine gravel size, with a maximum of about 5% fines, and a maximum particle size of 0.75 in (19 mm). The density is obtained by rotating a metal sleeve into the soil, removing the soil from the sleeve, and calculating the dry mass of soil removed per linear inch (per linear 25 mm). The mass per inch is correlated with the dry density of the in-place material using a calibration equation that has been predetermined for the soil being tested. Details can be found in ASTM D4564.

Test pit methods (ASTM D4914 and D5030) In these methods, a test pit is excavated in materials containing particles larger than about 3 in (75 mm). Test pit methods are generally limited to unsaturated materials and are not recommended for materials that are soft or friable or where water will seep into the excavated hole. Two methods are used which differ primarily by the manner in which the volume of the test pit is determined. In the *sand replacement method*, calibrated sand is poured into the excavated hole, and the mass of sand within the pit is determined. The volume of the hole is determined from the calibrated density of the sand and the total mass of sand within the hole in the same manner as for the sand cone test. In the *water replacement method*, an impervious liner is placed on the surface of the test pit, water is poured into the pit up to the appropriate level, and the mass or volume of the water within the pit is measured. Comparisons for the two tests are given below:

	Sand replacement	Water replacement
Maximum particle size	3–5 in (75–125 mm)	>5 in (>125 mm)
Excavated volume	1–6 ft^3 (0.03–0.17 m^3)	3–100 ft^3 (0.08–2.83 m^3)

Nuclear methods The use of nondestructive nuclear methods for moisture density testing of soils (ASTM D2922) has become increasingly popular in recent years because both density and water content readings can be obtained within a few minutes, which allows rapid feedback on the quality of the compacted soil. Density is determined by the emission of electromagnetic radiation (gamma rays or photons) from a radioactive source (usually either radium or cesium), as shown in Fig.

6A.135. Some of the gamma rays are absorbed by the soil particles while others are reflected. The number of photons reaching the detector (usually a Geiger-Mueller tube) is an indication of the total density of the soil; a lesser number of photons reaching the detector indicates a denser soil. Three modes of operation can be used. In the *backscatter* mode, the source and detector are on the bottom of the gauge and only about the top 2 to 3 in (50 to 75 mm) of the soil is tested. The backscatter technique is very sensitive to surface roughness and quality of site preparation (U.S. Army, 1985) and is not recommended for general use in soils. The gamma source is lowered into the ground through a hole punched into the soil in the *direct transmission* mode, which is therefore a pseudonondestructive technique. The depth of measurement can be varied in 1- or 2-in (25- or 50-mm) increments on most gauges, allowing a better measurement of the average density in a lift. The air gap mode, in which an open space is maintained between the bottom of the gauge and the ground surface, may also be used. Density calibration is achieved using standard blocks consisting of mate-



(a) (b)

FIGURE 6A.135 Measurement of density using a nuclear density meter: (*a*) backscatter; (*b*) direct transmission (after Troxler Electronic Laboratories, Inc., Research Triangle Park, NC).

rials of known density. Best calibration occurs using blocks of uniform rock because these materials best simulate those found in soils (Hausmann, 1990). Limestone and granite blocks are recommended in ASTM D2922. The gauge should be calibrated every day prior to testing. Sand cone or rubber balloon tests are commonly conducted at selected intervals of nuclear testing to provide comparison with the density and moisture results obtained from the nuclear gauges.

The measurement of moisture is nondestructive and is accomplished in the backscatter mode by emitting fast neutrons from a radioactive source (usually americium mixed with beryllium) located in the base of the gauge. The detector (usually a boron trifluoride or helium 3 tube) can only detect slow (thermal) neutrons and is also located in the base at some distance from the source. Hydrogen is the only atom normally found in soils that can slow down the neutrons to a level where they can be detected, and thus the moisture detector is really a hydrogen analyzer (U.S. Army, 1985). The moisture per unit volume of the soil is related to the total hydrogen content of the soil, and the water content (per unit mass of dry solids) can be calculated if the density is known. If hydrogen is present in the soil in any form other than free water, errors in measuring the water content will occur. Such errors may occur from the presence of various forms of hydrogen, including the following (U.S. Army, 1985; Hausmann, 1990):

1. Water of hydration in the mineral matrix
2. Organic matter
3. Oil or bitumen
4. Geosynthetics which contain hydrogen

Another factor which affects the accuracy of water content determined from a nuclear gauge is absorption interaction (Ballard and Gardner, 1965). Certain elements—particularly boron and cadmium, and to a lesser extent chlorine and iron—will absorb thermal neutrons to a very high degree. Thus, the presence of these elements in significant quantities may substantially affect the accuracy of the measured water content.

The major advantages and disadvantages of using nuclear gauges for moisture-density determination are summarized as follows (after Holtz and Kovacs, 1981 and Hausmann, 1990). Advantages of nuclear gauges are:

1. Tests can be performed quickly and results obtained within minutes. If necessary, corrective action can be taken before much additional fill has been placed.
2. Better statistical control of the fill is obtained because more tests can be conducted in the same amount of time.
3. The use of nuclear gauges on large projects or over extended periods is often more economical than other alternatives.

Disadvantages of nuclear gauges include:

1. The use of nuclear gauges is regulated by governmental agencies concerned with radiation safety. A radiation license must be obtained by the organization using the nuclear equipment.
2. Personnel using and supervising the use of nuclear gauges must be licensed and trained.
3. Operators are exposed to very small amounts of radiation and must wear film badges that monitor their exposure to radiation.
4. Extraordinary care must be taken during storage and transport of the gauges. If the seal containing the radioactive source is broken or is possibly broken, such as by a construction accident, a large area surrounding the gauge must be evacuated and the appropriate governmental agency notified. This may result in the shutdown of the entire construction project—and possible evacuation of nearby buildings and shutdown of nearby roads—for several hours or days while the status of the radioactive source is determined and necessary remedial measures taken.
5. Excessive concern or fear of the uninformed public and some transportation authorities when the radiation warning labels are sighted.

6. The initial cost of a nuclear gauge is relatively high compared to other moisture-density testing equipment.
7. There are differences in measured values of density and water content obtained using nuclear gauges versus the traditional tests (sand cone or rubber balloon for density, oven drying for moisture). Since some engineers consider the values obtained from the traditional tests to be "correct" even though those tests are also subject to inaccuracies, some engineers and governmental agencies have prohibited the use of nuclear gauges on some projects.
8. As discussed previously, there are some problems associated with determining density and moisture content indirectly using nuclear methods for certain soils and conditions. In some instances, therefore, the nuclear gauges simply will not give reliable or correctable answers. However, in most cases, if comparisons are made at selected intervals with the method specified as the standard, reliable values for density and water content can be obtained directly from the gauges or by applying corrections based on comparison with the values obtained from the specified standard test.

Hilf's rapid method Because of the problems associated with determining the water content of field-compacted cohesive soils—the long time required for oven drying and the differences in values obtained by the faster methods compared to those obtained by oven drying—Hilf (1959) developed a method that rapidly gives an exact value for standard Proctor relative compaction (R_s) and a close approximation of the field water content relative to the standard Proctor optimum water content (w_{os}). This method has been used satisfactorily on about 150 earth dams since 1957 and has been adopted for control of all compacted cohesive soils on U.S. Bureau of Reclamation projects (Hilf, 1991). The theoretical background and procedure for using this rapid method of field control is given as follows (after Hilf, 1991):

1. Excavate a field density hole in the compacted soil, weigh the excavated material, and determine the volume of the hole. Calculate the total density of the field-compacted soil (γ_{tf}) from the total weight and total volume. The amount of material removed should be enough to perform two reliable oven-dried water content tests and at least three standard Proctor tests without reusing any of the excavated material.
2. Perform a water content test by placing the excavated material in an oven at the site immediately after weighing it and drying the soil in the oven for a minimum of 16 h at 110°C (230°F) (preferred), or by placing the soil in an airtight container and transferring it to a laboratory for oven drying.
3. Conduct a minimum of three Proctor tests on the excavated material—one at the field water content and at least two others at different water contents.
4. Plot the results from the Proctor tests as total density (γ_t) versus added water in percentage of the total weight of the soil (Z), as shown by curve A in Fig. 6A.136. Z may be either positive or negative; that is, the water content of the soil during the Proctor tests can be either greater than or less than the water content of the field-compacted soil. Note that $\gamma_t = \gamma_d(1 + w_f)$, where w_f is the water content of the field-compacted soil. A relationship between Z and w (the water content of the soil for any Proctor test) can be developed as follows:

$$\Delta W_w = W_s \cdot \Delta w = W_s \cdot (w - w_f) \tag{6A.47}$$

$$W_{tf} = W_s \cdot (1 + w_f) \tag{6A.48}$$

$$Z = \frac{\Delta W_w}{W_{tf}} = \frac{W_s \cdot (w - w_f)}{W_s \cdot (w + w_f)} = \frac{w - w_f}{1 + w_f} \tag{6A.49}$$

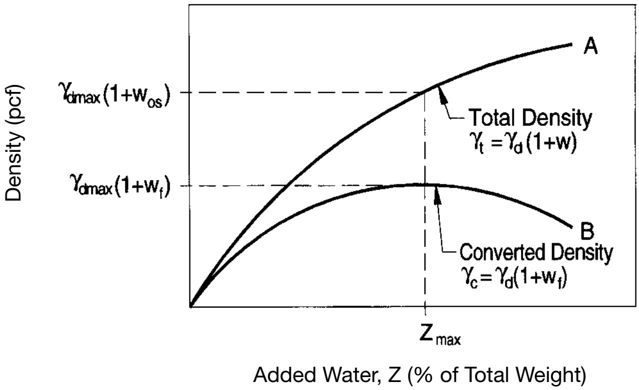


FIGURE 6A.136 Compaction curves for Hilf's rapid method of control (after Hilf, 1990).

where ΔW_w = weight of water added or removed from soil
W_s = weight of the solids (dry weight of the soil)
Δw = change in water content of the soil
w_f = water content of the field-compacted soil
W_{tf} = total weight of the soils at w_f

5. Calculate the converted density (γ_c) for each of the Proctor tests as follows:

$$\gamma_c = \frac{\gamma_t}{1+Z} = \frac{\gamma_d \cdot (1+w)}{1 + \dfrac{w - w_f}{1 + w_f}} = \frac{\gamma_d \cdot (1+w)}{\dfrac{1+w}{1+w_f}} = \gamma_d \cdot (1 + w_f) \tag{6A.50}$$

Plot a curve of γ_c versus Z (curve B in Fig. 6A.136). Curve B can be drawn with only three data points by assuming that the curve is parabolic and using the method shown in Fig. 6A.137.

6. Obtain Z_{max} as the abscissa at the peak point of curve B. Since w_f is a constant for every point on curve B, the maximum ordinate of curve B must have the value of $\gamma_{dmax}(1 + w_f)$. Dividing this value into γ_{tf} gives the desired value of relative compaction as follows:

$$R_s = \frac{\gamma_{tf}}{\gamma_{dmax} \cdot (1+w_f)} = \frac{\gamma_{df} \cdot (1+w_f)}{\gamma_{dmax} \cdot (1+w_f)} = \frac{\gamma_{df}}{\gamma_{dmax}} \tag{6A.51}$$

7. Estimate w_{os} from Fig. 6A.138, which shows a best-fit curve of γ_t versus w_{os} based on data from 1300 specimens. Estimate $(w_{os} - w_f)$ and w_{os} from the following equation:

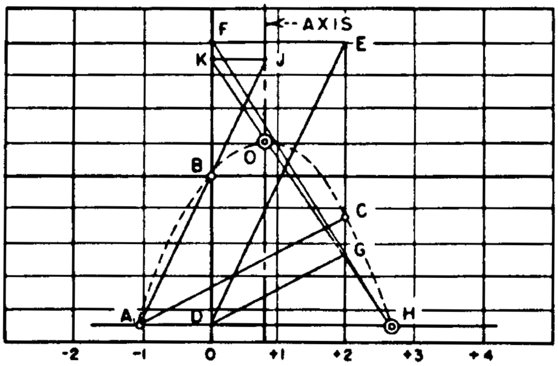
$$w_{os} - w_f = Z_{max} \cdot (1 + w_f) = \frac{Z_{max} \cdot (1 + w_{os})}{1 + Z_{max}} \tag{6A.52}$$

$$w_{os} = w_f + (w_{os} - w_f) \tag{6A.53}$$

The estimate of $(w_{os} - w_f)$ obtained in this manner is usually very close to the actual value and can be used to determine the acceptability of the field-compacted soil in terms of moisture con-

Parabola Method

Graphical solution for vertex, O, of a parabola whose axis is vertical, given three points A, B, and C. If more than three points are available, use the three closest to optimum.



1. Draw horizontal base line through the left point, A, and draw vertical lines through points B and C.
2. Draw line DE parallel to AB, point E lies on the vertical line through point C, project E horizontally to establish point F on the vertical line through B.
3. Draw line DG parallel to AC, point G lies on the vertical line through point C.
4. Line FG intersects the base line at H. Axis of parabola bisects AH, draw the axis.
5. Intersection of line AB with the axis is at J, project J horizontally to K, which lies on the vertical line through point B.
6. Line KH intersects the axis at O, the vertex.

NOTE: If points A, B, and C are equally spaced horizontally (this is true when 2 points are obtained by adding water or when soil is dried exactly 2 percent) steps 2 and 3 above are eliminated. Point F coincides with point B and point G is halfway between the base line and point C. Hence, point H is obtained by drawing BG and point O is obtained by steps 5 and 6 as usual. See graph below.

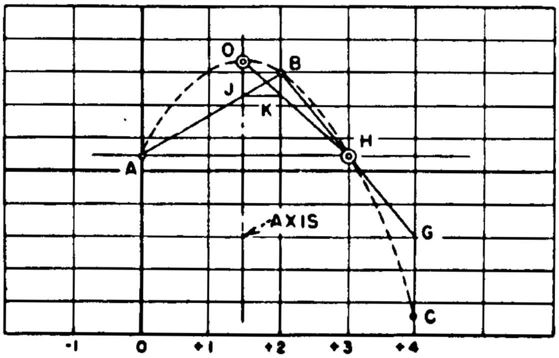


FIGURE 6A.137 Hilf's parabolic method for obtaining the peak point for the curve of converted density versus added water based on three data points (from Hilf, 1991).

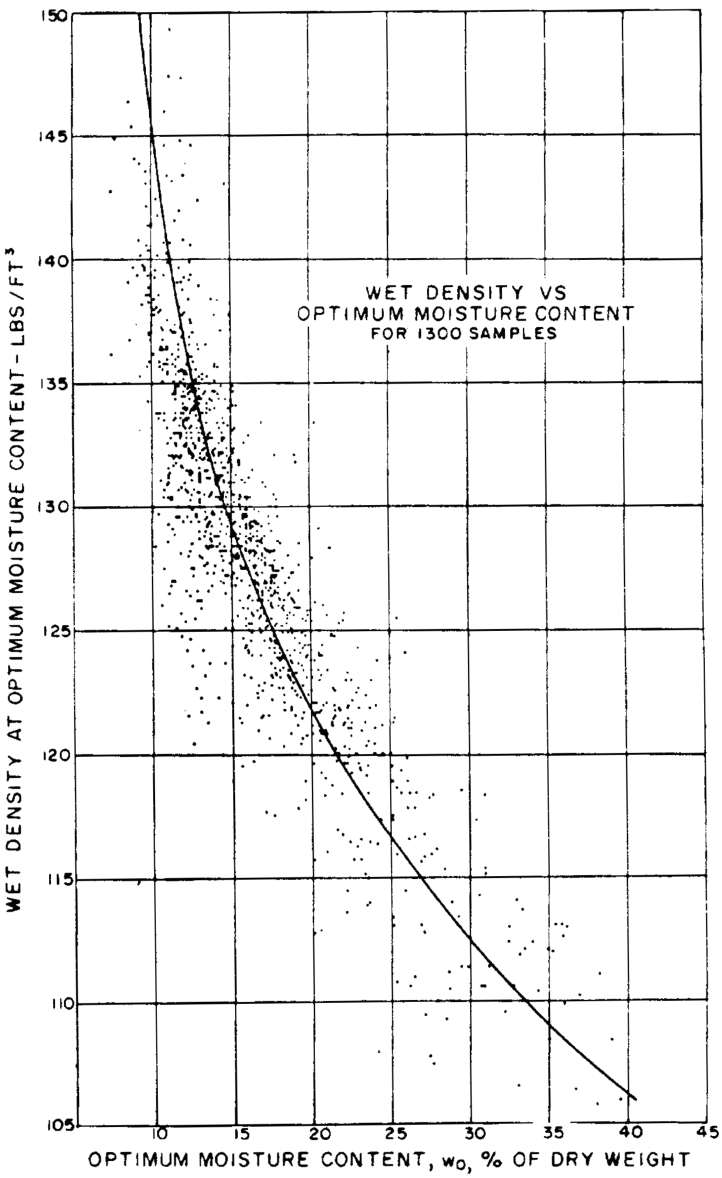


FIGURE 6A.138 Best-fit curve for total (wet) density versus standard Proctor optimum water content based on data from 1300 specimens (from Hilf, 1991).

dition only when the moisture condition is specified in terms of a range of water contents referenced to w_{os}. The value of w_{os} obtained from Eqs. (6A.52) and (6A.53) is an approximation only and may vary significantly from the actual value of w_{os}.

8. After the oven-dried water content has been determined, calculate the actual values of R_s and ($w_{os} - w_f$) and compare with the estimated values determined previously. Both values of R_s should be the same, and the actual value of ($w_{os} - w_f$) should be close to the estimated value.

The following is an abbreviated example given to illustrate the procedure (from Hilf, 1991): From a field density test we have γ_{tf} = 127.5 pcf (20.03 kN/m³), and from standard Proctor tests on excavated material we have these values:

		γ_t		γ_c	
Point	Z, %	pcf	kN/m³	pcf	kN/m³
1	0	123.4	19.39	123.4	19.39
2	2	128.6	20.20	126.1	19.81
3	4	124.6	19.57	119.8	18.82

By the parabolic method:

$$Z_{\max} = 1.6\%$$

$$\gamma_{d\max}(1 + w_f) = 126.3 \text{ pcf } (19.\ 84 \text{ kN/m}^3)$$

From Eq. (6A.51):

$$R_s = \frac{127.5}{126.3} \cdot 100 = 101.0\%$$

From Fig. 6A.138:

$$w_{os} \cong 16.0\%$$

From Eq. (6A.52):

$$w_{os} - w_f = \frac{1.6 \cdot (1 + 0.160)}{1 + 0.016} = +1.8\% \text{ (dry of } w_{os})$$

After oven-drying water content specimens, the actual values are calculated as follows:

$$w_f = 15.0\%$$

$$\gamma_{df} = \frac{\gamma_{tf}}{1 + w_f} = \frac{127.5}{1 + 0.150} = 110.9 \text{ pcf } (17.42 \text{ kN/m}^3)$$

$$\gamma_{d\max} = \frac{\gamma_{d\max} \cdot (1 + w_f)}{1 + w_f} = \frac{126.3}{1 + 0.150} = 109.8 \text{ pcf } (17.25 \text{ kN/m}^3)$$

$$R_s = \frac{\gamma_{df}}{\gamma_{d\max}} = \frac{110.9}{109.8} \cdot 100 = 101.0\% \text{ (checks exactly with previous value)}$$

$$w_{os} - w_f = Z_{\max} \cdot (1 + w_f) = 1.6 \cdot (1 + 0.150) = 1.8\% \text{ (checks exactly with previous value)}$$

$$w_{os} = w_f + (w_{os} - w_f) = 15.0 + 1.8 = 16.8\% \text{ (compared to estimated 16.0\%)}$$

Other Verification Tests. As noted in Section 6A.3.5.1, many different types of tests may be used to verify that the compacted soil will be acceptable from an engineering standpoint. Some of these tests are discussed in more detail here.

Proctor penetrometer (ASTM D1558) Proctor (1933) developed the soil plasticity needle—now more commonly known as the *Proctor penetrometer*—to assist in evaluating the suitability of compacted soils for use in earth dam construction. A modern version of the penetrometer is depicted in Fig. 6A.139 and can be used in the field and the laboratory. It consists of a special spring dynamometer with a pressure-indicating stem on the handle that is graduated to 90 lb in 2-lb divisions with a line encircling the stem at each 10-lb interval (or 45 kg in 1-kg divisions with a line at each 5-kg interval). The needle is pushed into the field-compacted soil or laboratory-compacted specimen at a rate of 0.5 in/s (13 mm/s) for a distance of at least 3 in (76 mm). As the spring deflects under load, a sliding ring on the stem indicates the developed force and remains at the maximum value as the force decreases during additional penetration beyond the point where the maximum force is developed. Interchangeable needles with heads varying in area from 0.025 in^2 (0.16 cm^2) to 1 in^2 (6.45 cm^2) are available so that the maximum force in any soil can be measured within a consistent range (40 to 80 lb or 10 to 20 kg according to ASTM D1558). The maximum penetration resistance is calculated as the maximum force divided by the area of the head.

The procedure used to determine the acceptability of field-compacted soils is as follows:

1. Establish the relationship between maximum penetration resistance and compaction water by conducting tests on Proctor specimens compacted in the laboratory or field, or on field-compacted test pad sections, for the soil compacted at various water contents with the same method and energy. A minimum of three tests at different water contents is specified by ASTM D1558, but at least five tests are recommended by the author.
2. Plot the results of the penetrometer tests on the same graph with the moisture-density results, as illustrated in Fig. 6A.140. Conduct laboratory or field tests to correlate penetration resistance with the desired engineering properties. Establish an acceptable range of penetration resistances. For bearing situations where strength and settlement are the most important engineering characteristics, a minimum value of penetration resistance is usually prescribed. The specifications can also be in the form of a range of values.
3. Conduct penetrometer tests at random locations within each compacted lift using a procedure for selecting testing locations similar to that given previously for density-water content testing. Compare the measured penetration resistance with the allowable criteria to determine the acceptability or unacceptability of the compacted soil.

Plate load test (ASTM D1194, D1195, D1196) The plate load test can be used to determine the stiffness and/or bearing capacity of a compacted soil. The steps performed in a plate load test are as follows (see Fig. 6A.141):

1. A steel plate [commonly 12 in (305 mm) in diameter] or a small concrete footing is placed at the desired depth within the compacted soil.
2. Vertical load is applied to the plate or footing by jacking against a reaction beam, a dead weight loading frame, or an axle of a heavy construction vehicle. The magnitude of the load is usually determined from a pressure gauge located on a hydraulic jack that has been calibrated

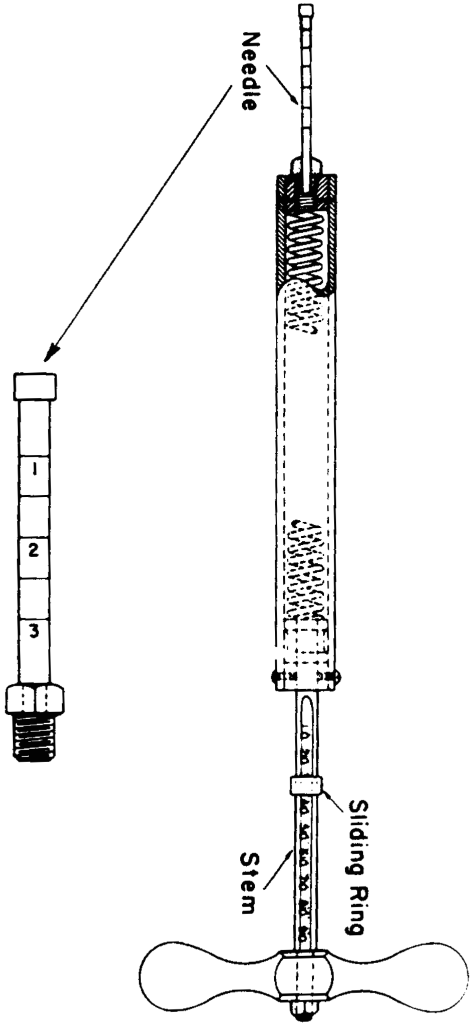


FIGURE 6A.139 Schematic illustration of Proctor penetrometer (from ASTM D1558).

to indicate applied force or is read directly from an electronic load cell. The load is increased until a predetermined load or the maximum load for the setup has been reached or until failure has occurred.

3. Vertical settlement of the plate is measured at various times after the application of each loading increment. Settlement readings are taken at one or more locations on the plate or footing using a dial gauge or electronic distance transducer attached to an independent reference beam.

The results from a plate load test are plotted as settlement, S, versus average applied pressure, q_0 (Fig. 6A.142), from which the subgrade modulus ($k_s = \Delta q_0/\Delta S$) and ultimate bearing capacity (q_{ult},

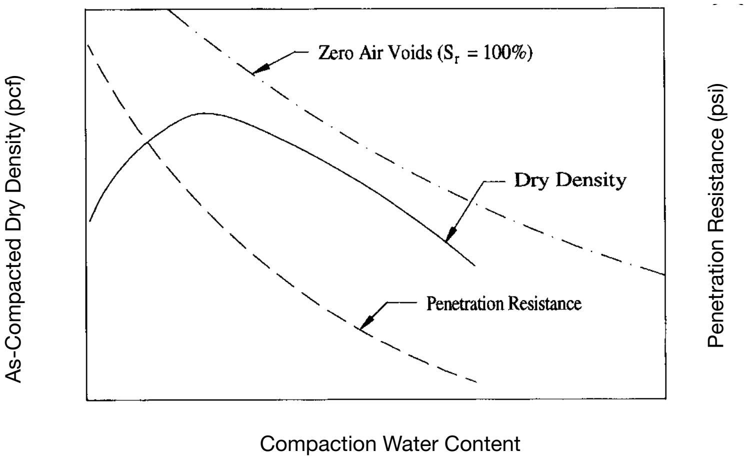


FIGURE 6A.140 Proctor penetration resistance and as-compacted dry density as a function of compaction water content.

if loaded to failure) can be calculated. Because soils have nonlinear stress-deformation characteristics, a variety of methods can be used to calculate k_s, so the method to be used must be stated in the compaction specifications. Common methods for calculating k_s are shown in Fig. 6A.142 and include the initial tangent modulus (line *A*), secant modulus between zero and one-half the maximum or ultimate applied pressure (line *B*), tangent modulus at one-half the maximum or ultimate applied pressure (line *C*), or reload secant modulus from zero to the maximum or ultimate applied pressure (line *D*). The reload curve is often deemed to give better prediction of the stiffness of the field-compacted soil than the curve for first loading because problems associated with bedding errors are reduced or eliminated on reloading.

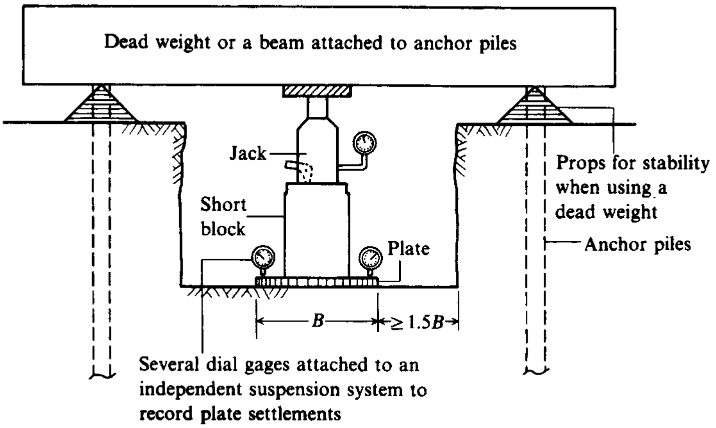


FIGURE 6A.141 Schematic diagram of a typical setup for a plate load test (from Bowles, 1988).

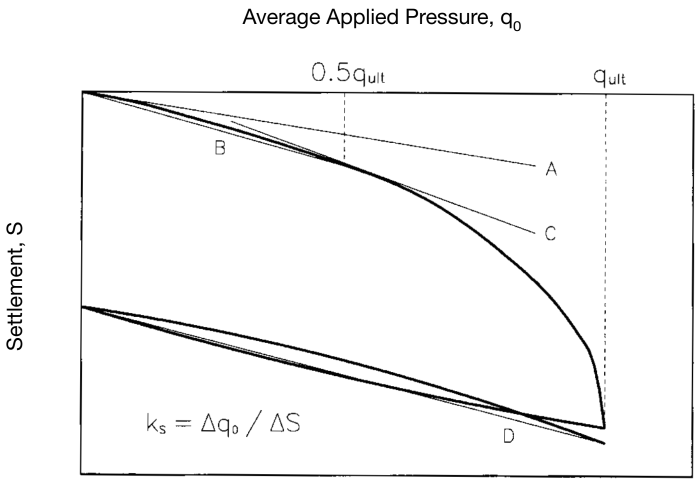


FIGURE 6A.142 Typical results from a plate load test.

Because the subgrade modulus is a function of the width and shape of the loaded area, a scaling relationship is needed to extrapolate the results of plate load tests to full field scale (e.g., the width and shape of the footings for a building). It is best to establish this scaling relationship at the site for the actual soils and conditions. This can be accomplished by conducting a series of plate load tests, wherein the size of the plates or small footings are varied but are of the same shape as the actual foundation. For example, if the actual footings will be square, a series of tests using square plates with widths of 12, 18, and 24 in (305, 407, and 610 mm) could be conducted at stress levels comparable to those expected in the actual foundation. From the results of these tests, k_s can be plotted as a function of plate width and the curve extrapolated to obtain an estimate of k_s for the actual foundation width. It is desirable from an engineering standpoint to conduct tests at full field scale, but this is seldom done because of the cost and time needed to perform full-scale tests.

In the absence of data obtained for the particular site and soil, the scaling laws established by Terzaghi (1955) are often used. Terzaghi considered two types of soils: (1) Those whose stiffness is proportional to the effective confining pressure (e.g., sands and most partially saturated soils of all types), and (2) those whose stiffness is independent of the confining pressure (e.g., saturated, undrained clays). The following relationships were developed for estimating subgrade modulus for a full-scale footing based on the results of plate load tests on 1 ft (0.3 m) wide square plates.

For sands and square or rectangular foundations:

$$k_S = k_{S1} \cdot \left(\frac{B+1}{2B}\right)^2 \text{ for } B \text{ in units of ft} \tag{6A.54a}$$

$$k_S = k_{S1} \cdot \left(\frac{B+0.3}{2B}\right)^2 \text{ for } B \text{ in units of m} \tag{6A.54b}$$

where k_S = subgrade modulus for a foundation of width B
k_{S1} = subgrade modulus for a 1-ft (0.3-m) *square* plate
B = width of actual foundation

For saturated, undrained clays and rectangular foundations:

$$k_S = \frac{k_{S1}}{B} \cdot \frac{n + 0.5}{1.5n} \quad \text{for } B \text{ in ft} \tag{6A.55a}$$

$$k_S = \frac{0.3k_{S1}}{B} \cdot \frac{n + 0.5}{1.5n} \quad \text{for } B \text{ in m} \tag{6A.55b}$$

where $n = L/B$
L = length of actual foundation

For a square foundation ($n = 1$), Eq. (6A.55) reduces to

$$k_S = \frac{k_{S1}}{B} \quad \text{for } B \text{ in units of ft} \tag{6A.56a}$$

$$k_S = \frac{0.3k_{S1}}{B} \quad \text{for } B \text{ in units of m} \tag{6A.56b}$$

and for a strip or continuous foundation ($n \approx \infty$), Eq. (6A.55) reduces to

$$k_S \cong \frac{2}{3} \cdot \frac{k_S}{B} \quad \text{for } B \text{ in units of ft} \tag{6A.57a}$$

$$k_S \cong \frac{1}{5} \cdot \frac{k_S}{B} \quad \text{for } B \text{ in units of m} \tag{6A.57b}$$

These relationships are valid for applied stresses less than one-half the ultimate bearing capacity of the actual footing.

These scaling laws were developed for relatively homogeneous soils, and one should exercise caution when the depth of influence for the actual foundation (approximately $2B$) extends into a stratum or strata that are not influenced during the plate load test. This concept is illustrated in Fig. 6A.143 for the case of a dense sand overlying a soft clay layer wherein the depth of influence for the plate load test is entirely within the sand layer, while the depth of influence for the actual footing extends into the soft clay layer. Therefore, extrapolating the results from the plate load test to field scale using Terzaghi's scaling relationships would substantially overestimate the subgrade modulus, which would not be conservative.

Some engineers prefer to use stress-strain modulus (E_s) for the soil rather than k_s. An average value of E_s for the actual foundation can be estimated from the equations for settlement based on elastic theory for a uniformly loaded foundation on a semi-infinite, homogeneous, linearly elastic material. These equations for settlement are of the following general form:

$$S = \frac{q_0 \cdot B \cdot (1 - v^2)}{E_s} \cdot I_s \cdot I_d \tag{6A.58}$$

where q_0 = applied pressure
B = width or diameter of loaded area
v = Poisson's ratio

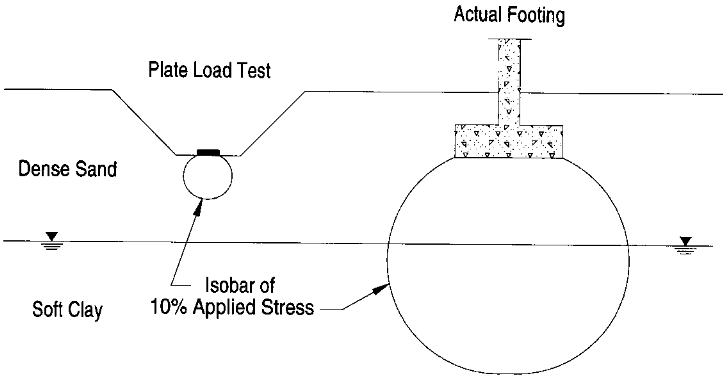


FIGURE 6A.143 Comparison of depth of influence for a plate load test and an actual footing.

E_s = stress-strain modulus for the soil
I_s = influence factor that accounts for the shape and rigidity of the foundation, and the location where settlement is calculated
I_d = correction factor for depth of embedment (D)

Using the secant value of subgrade modulus ($k_s = q_0/S$), Eq. (6A.58) can be rearranged to solve for $E_s = f(k_s)$ as follows:

$$E_s = k_s B(1 - v^2) I_s I_d \tag{6A.59}$$

When calculating E_s for the actual foundation, the value of k_s input into Eq. (6A.59) should be the field scale k_s. Equations or values of I_s for either rigid or flexible foundations of circular or rectangular shape can be found in a number of reference books (e.g., Poulos and Davis, 1974; Das, 1983). Selected values of I_s for circular and square foundations—the most common shapes used in plate load tests—are given in Table 6A.11. The plates in most plate load tests are quite rigid. If the plate is wide, usually a stack of increasingly narrower plates is placed on the bearing plate to increase the rigidity of the system. Thus, values of I_s for rigid foundations generally should be used. For example, $I_s = 0.79$ for a rigid, circular foundation on a homogeneous half-space, and this value is commonly used for circular plate load tests. If the compressible layer or layers being tested are underlain by a hard layer, a value of I_s for a finite height of compressible layer (H_c) should be used. E. Fox (1948) presented equations to calculate I_d for rectangular foundations embedded in a homogeneous half-space, and charts for estimating I_d based on these equations are given in Fig. 6A.144. Equations for calculating I_d for circular foundations can be found in Nishida (1966). Figure 6A.144 can also be used for circular foundations with little loss of accuracy by converting to an equivalent square based on area. Note that the values of I_d in Fig. 6A.144 are not strictly applicable to finite-height compressible layers but may be used as approximations. In tests conducted below the ground surface, the bottom of the excavated pit is typically several times wider than the width of the plate, so the use of a correction factor for embedment in these cases is probably not justified. v is usually estimated based on the soil type and moisture condition, but also can be determined from triaxial testing either directly by measuring radial deformations using a Poisson's ratio clamp or Hall effect transducers or indirectly by measuring the changes in height and total volume of the specimen and assuming that the specimen retains the shape of a right circular cylinder during the testing.

TABLE 6A.11 Selected Values of Settlement Influence Factor I_s for Circular and Square Foundations

			I_s		
				Flexible	
Shape	v	H_c/B	Rigid	Center	Edge
Circular	0 to 0.5	∞	0.79	1.00	0.64
	0.45	3		0.80	0.47
	0.45	2		0.77	0.43
	0.45	1		0.61	0.29
	0.3	3		0.85	0.50
	0.3	2		0.81	0.46
	0.3	1		0.67	0.34
	0.15	3		0.86	0.51
	0.15	2		0.83	0.47
	0.15	1		0.70	0.36
Square	0 to 0.5	∞	0.89	1.12	0.56
	0.5	5		1.00	0.43
	0.5	2	0.77	0.82	0.29
	0.5	1	0.57	0.57	0.14
	0.4	5		1.01	0.45
	0.4	2		0.84	0.31
	0.4	1		0.61	0.17
	0.3	5		1.01	0.45
	0.3	2		0.86	0.32
	0.3	1		0.64	0.19
	0.2	5		1.02	0.46
	0.2	2		0.87	0.33
	0.2	1		0.67	0.20

The advantages of the plate load test over moisture-density testing can be summarized as follows (after Hausmann, 1990):

1. The stiffness and strength of the compacted soil are evaluated directly.
2. The results are available immediately.
3. No laboratory testing (e.g., compaction tests) is needed.
4. The test is suitable for a wide range of soil types and maximum particle sizes.
5. Once established as a routine test, the plate load test can be conducted rapidly at low cost.

California bearing ratio (CBR) test (ASTM D4429) In the CBR test, a 1.95 in (49.6 mm) diameter piston with a flat head is slowly pushed into the soil, and the resisting force developed at various depths of penetration is measured. CBR value is calculated at penetrations of 0.1 and 0.2 in (2.54 and 5.08 mm) by dividing the calculated bearing stress by the appropriate standard stress (which varies as a function of penetration), and multiplying by 100. The higher of the two values is reported as the CBR value. If the piston is penetrated deeply enough, failure occurs in the soil beneath the piston, and the test is essentially a miniature bearing capacity test.

CBR value is an indication of the stiffness and strength of the soil and is widely used in the design and control of subsoils beneath pavements. By correlating CBR value with the desired strength and compressibility requirements for a particular compacted soil through laboratory testing or em-

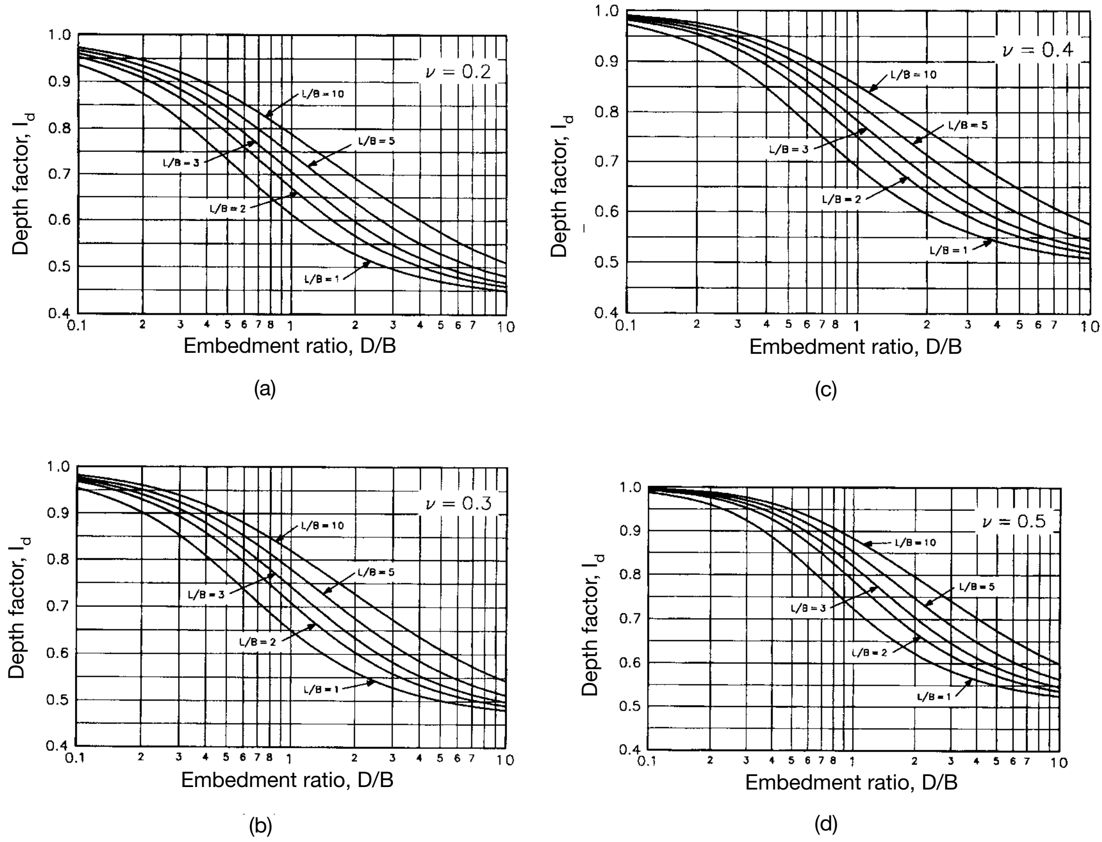


FIGURE 6A.144 Charts for estimating settlement correction factor for depth of embedment (I_d) for rectangular foundations based on E. Fox's (1948) equations: (*a*) $\nu = 0.2$; (*b*) $\nu = 0.3$; (*c*) $\nu = 0.4$; and (*d*) $\nu = 0.5$ (format adapted from Bowles, 1988).

pirical data on similar soils, the CBR test can, in some circumstances, provide a rapid and effective method for assessing the suitability of compacted soils.

The CBR test is similar to the plate load test in the type of results (penetration or settlement versus stress) that are obtained. In fact, one can easily calculate subgrade modulus from the CBR data. The primary difference in the two tests is the width of the loaded area–approximately 2 in (51 mm) in the CBR test and commonly 12 in (305 mm) in the plate load test–with a depth of influence of about 4 in (102 mm) in the CBR test and about 24 in (610 mm) in the plate load test. Therefore, the plate load test is suitable for a wider range of soils. However, for thin lifts (no more than about 8 in = 203 mm) and soils for which particles larger than medium sand size are only a small fraction of the soil, CBR tests often can be conducted faster and cheaper in many instances than can plate load tests, usually with equivalent success.

Standard penetration test (ASTM D1586) The standard penetration test (SPT) is seldom used for quality control of compacted fills for the following reasons: (*a*) The test is conducted over a depth of 12 in (305 mm), so that it would be valid only for this lift thickness or greater; (*b*) the setup time and cost for performing one test per location is excessive; and (*c*) the results are reliable only for granular soils with little or no particles gravel-size or larger. The main application for the SPT in compacted soils is for checking the degree of improvement in compacted in situ soils (primarily deep methods such as dynamic compaction and vibrocompaction).

Cone penetration test (ASTM D3441) The cone penetration test (CPT) has limited application in near-surface compacted soils primarily because of setup time and cost. The CPT is not applicable in stiff or hard clays or soils that contain appreciable amounts of gravel or larger particles and is primarily applicable for establishing the improvement in both deeply compacted sands and silty sands.

Sealed double ring infiltration test (ASTM D5093) The sealed double-ring infiltration (SDRI) test has become increasingly popular in recent years for determining the field coefficient of permeability (hydraulic conductivity) of compacted clay liners. It was developed to eliminate the two major problems associated with using other field hydraulic conductivity tests on low permeability soils—difficulties in measuring small changes in elevation of the water surface (caused by low flow rates and evaporation) and separating lateral flow from vertical flow (Trautwein 1993).

The SDRI consists of an open outer ring and a sealed inner ring (Fig. 6A.145). The rings may be either circular or square, but square rings are usually used because they make it easier to excavate straight trenches in the soil. The most common sizes for the rings are 12 ft (3.7 m) and 5 ft (1.5 m) square for the outer and inner rings, respectively.

The general procedure for conducting an SDRI test is summarized as follows (ASTM D5093; Trautwein, 1993):

1. The rings are embedded in trenches excavated in the soil and the trenches are sealed with grout to prevent water loss.
2. The outer ring is filled with water to a level such that the inner ring is completely submerged.
3. The rate of flow is measured by connecting a flexible bag filled with a known weight of water to a port on the inner ring. As water from the inner ring infiltrates into the ground, an equal amount of water flows from the flexible bag into the inner ring. The flexible bag is removed after a known interval of time and weighed. The volume of water that has infiltrated the ground is determined from the weight loss of the flexible bag.
4. The infiltration rate I is calculated from the following equation:

$$I = \frac{Q}{At} \tag{6A.60}$$

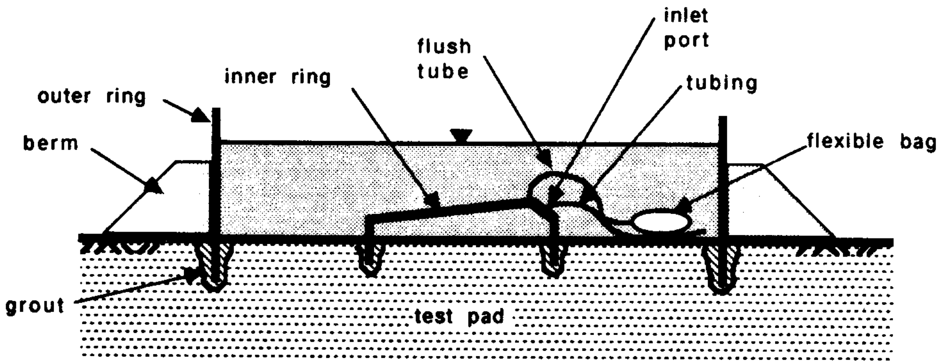


FIGURE 6A.145 Schematic illustration of a typical setup for a sealed double ring infiltration test (from ASTM D5093).

where Q = volume of infiltrated water
A = area of the inner ring
t = time interval over which Q is determined

5. The process is repeated, and a plot is constructed of infiltration rate versus time. The test is continued until the infiltration rate becomes steady or until it becomes less than or equal to a specified value.
6. The coefficient of permeability k is calculated as follows:

$$k = \frac{Q}{iAt} = \frac{I}{i} \tag{6A.61}$$

where i = hydraulic gradient $\Delta H/\Delta L$
ΔH = total head loss
ΔL = length of flow path for which ΔH is measured

This equation for k assumes steady state, vertical, one-dimensional flow under saturated conditions.

Although Eq. (6A.61) seems simple, the calculation of i is complex because the flow is transient and the soil is usually unsaturated at the start of testing. The problem of transient flow is overcome by waiting for quasi-steady-state conditions to be reached, which unfortunately results in long testing times. The unsaturated flow makes it difficult to determine the gradient, but this can be alleviated by making some simplifying assumptions. Three methods are commonly used for calculating i (refer to Fig. 6A.146 for illustration of parameters).

Apparent Hydraulic Conductivity Method This is the simplest method, and it yields the most conservative (highest) value of k. It is assumed that the wetting front has passed completely through the compacted liner, which means that the depth of the wetting front is equal to the thickness of the liner ($D_W = H_L$) and that the suction head at the bottom of the liner is zero ($H_s = 0$). This results in $\Delta H = H_W + H_L$, and $\Delta L = H_L$. The advantage of this method is that neither the location of

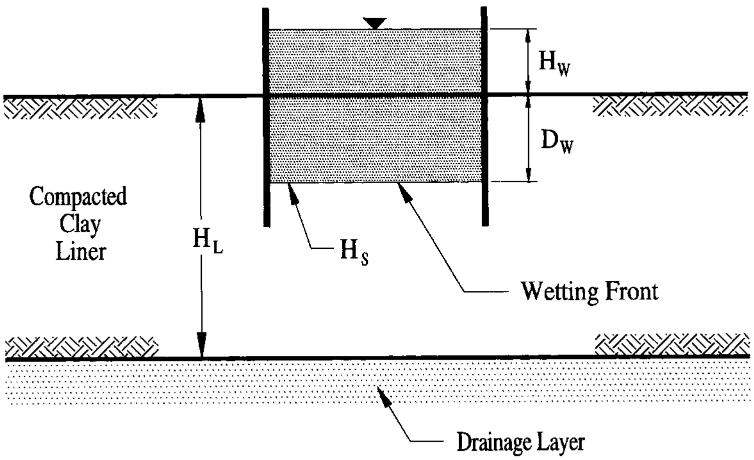


FIGURE 6A.146 Parameters for calculating hydraulic gradient for the sealed double-ring infiltration test (after Trautwein, 1993).

the wetting front nor the suction at the wetting front need be known. The disadvantage is that the correct value of k is calculated only if the wetting front has passed through the test zone; otherwise, k is overestimated. Therefore, this method can be used to determine compliance with the specified maximum acceptable value of k ($k \leq k_{max}$) but cannot be used to determine noncompliance ($k > k_{max}$).

Suction Head Method In this method it is assumed that H_s is equal to the ambient suction in the soil below the wetting front, and the location of the wetting front and the ambient soil suction must be known. For this method, $\Delta H = H_W + D_W + H_s$, and $\Delta L = D_W$. Unfortunately, the basic assumption appears to be incorrect. Although the suction below the wetting front may have some impact on infiltration rate, this impact is apparently offset by low hydraulic conductivity in the unsaturated transition zone between the wetting front and the unaffected unsaturated soil, which restricts downward flow. This method is not recommended because it can yield highly unconservative (low) values of k.

Wetting Front Method This method is based on the assumption that the suction head at the wetting front is zero ($H_s = 0$) and requires that the location of the wetting front be known. With this assumption and a known D_w, $\Delta H = H_W + D_W$ and $\Delta L = D_W$. Because ambient suction may have a small effect on infiltration (and hence k), this method is considered to be conservative. Recent research suggests that the wetting front method is the best of the three methods, and it is the recommended procedure.

The location of the wetting front and changes in suction within the compacted soil as the water advances can be monitored by installing tensiometers in the compacted soil. Typically nine tensiometers are used per test, three at each depth of 6, 12, and 18 in (152, 305, 457 mm). If desired, tensiometers can also be installed in the native soil to measure and monitor ambient suctions.

The SDRI test is the best test currently available for determining the hydraulic conductivity of low-permeability in situ soils. Many governmental agencies now require the SDRI test to establish the acceptability of compacted clay liners for containing waste and hazardous materials. The disadvantages of SDRI testing include lack of overburden, long test times (a few weeks to several months), and high cost ($8000 to $15,000 per test).

6A.4 DEEP COMPACTION

The methods for compacting near-surface soils described in Section 6A.3 can be used to a maximum depth of about 10 ft (3 m) under highly favorable circumstances and generally much less than this depth. Techniques for compacting soil to greater depths include dynamic compaction, vibrocompaction, and blasting. These three methods are discussed in the following sections.

6A.4.1 Dynamic Compaction

Dynamic compaction consists of repeatedly raising and dropping heavy tampers onto the ground surface to compact the underlying soil deposits to typical depths of improvement (D) of about 10 to 35 ft (3.0 to 10.7 m) (Lukas 1986, 1995). With special lifting equipment and a heavy tamper, the ground can be affected to depths as great as 100 ft (30.5 m). This process is illustrated in Figs. 6A.147 and 6A.148 and has also been referred to as pounding, dynamic consolidation, dynamic precompression, impact densification, and heavy tamping. The weight of the tampers (W) is generally in the range of 6 to 30 tons (5.4 to 27.2 MN), with usual drop heights (H) of 40 to 100 ft (12.2 to 30.5 m).

Although this technique was used long ago by the Romans and as early as the 1870s in the United States (Kerisel 1985), it was resurrected and systematically developed in the early 1970s by the

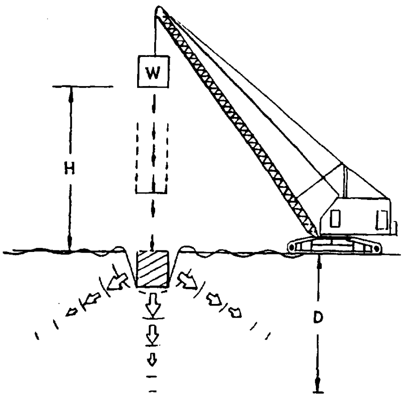


FIGURE 6A.147 Schematic illustration of the dynamic compaction process (modified from Lukas 1995).

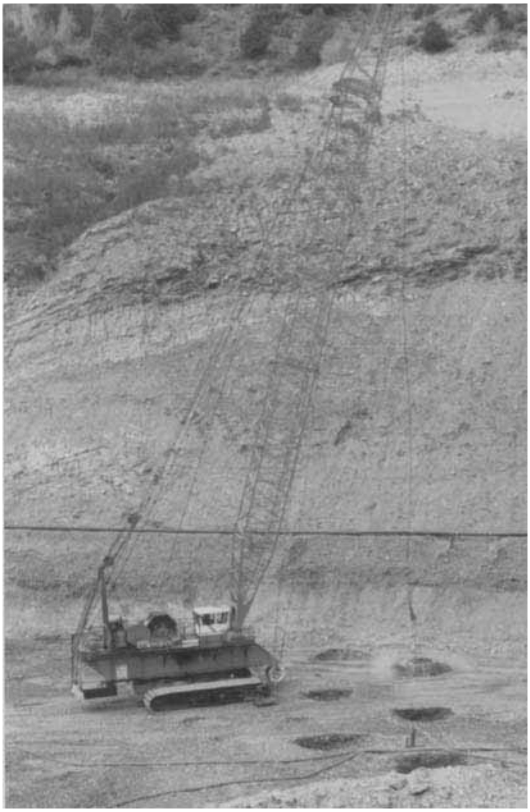


FIGURE 6A.148 Dynamic compaction showing tamper at impact.

Frenchman Louis Menard (Welsh 1986). This method has seen considerable growth in the last decade, with over 500 projects completed in the U.S. (ASCE 1997).

Dynamic compaction has been successful in improving many types of weak ground deposits, including the following (Lukas 1995):

1. Loose, naturally occurring soils such as alluvial, flood plain, and hydraulic fill deposits
2. Recent and old landfill deposits
3. Deposits of construction debris and building rubble
4. Spoil from strip mines
5. Partially saturated clay fill deposits that are located above the groundwater table
6. Formations where large voids are present close to final grade; for example, karst topography and sinkholes
7. Saturated loose sands and silts to reduce liquefaction potential

6A.4.1.1 Equipment

Tampers with weights of 25 tons (220 kN) or less are generally raised and dropped with a conventional heavy crawler crane using a single cable with a free spool to allow the drop to be nearly free fall. For heavier tampers, either conventional equipment is modified to reinforce certain components or specially designed equipment is used to raise and drop the tamper. A summary is given in Table 6A.12 of required crane and cable sizes as a function of tamper weight.

The tampers are typically constructed of steel or steel shells filled with sand or concrete (Mayne et al. 1984). Reinforced concrete tampers have also been used but tend to fall apart after repeated use. The bases of the tampers are typically square, circular, or octagonal. The craters formed during the primary phase eventually assume a circular shape, so circular or octagonal bases are best suited for this phase. Square bases are best suited for the ironing phase where the tamper is dropped on a contiguous or overlapping pattern to densify the surficial soils only. A steel tamper with a circular base is shown in Fig. 6A.149.

6A.4.1.2 General Procedures

The spacing of the impact points, the energy per drop (combination of weight of tamper and drop height), and the chronological sequence of energy application depend primarily on the following factors:

1. Depth and thickness of the compressible layer or layers
2. Depth to the groundwater table
3. Permeability and moisture condition of the compressible materials

In general, there are at least two phases of the treatment process, which are normally called passes. The first phase—called the *primary phase* or *high-energy phase*—is intended to improve the deeper portions of the compressible zone. Impact points are widely spaced in this phase to prevent the creation of a dense zone of material at an intermediate depth. If an intermediate dense zone is created,

TABLE 6A.12 Equipment Requirements for Different Weights of Tampers (from Lukas, 1986)

Tamper weight		Crawler crane size		Cable size	
tons	kN	tons	kN	in	mm
6 to 8	53 to 71	40 to 50	360 to 440	3/4 to 7/8	19 to 22
8 to 14	71 to 120	50 to 100	440 to 890	7/8 to 1	22 to 25
15 to 18	130 to 160	100 to 125	890 to 1,100	1 to 1 1/8	25 to 29
18 to 25	160 to 220	150 to 175	1,300 to 1,600	1 1/4 to 1 1/2	32 to 38

FIGURE 6A.149 Steel tamper with a circular base.

it tends to spread the induced pressures over a wider area, lessening the magnitude of the energy transmitted beneath it. This makes it difficult or impossible to densify the underlying materials. A general rule of thumb is that the spacing of impact points in the primary phase should be at least equal to the depth to the bottom of the compressible layer or the maximum depth to be improved. Also, the spacing should be at least 1.5 to 2.5 times the diameter or width of the tamper and usually not less than about 10 ft (3 m). There may be more than one pass at the same impact points during the primary phase. The craters are backfilled after each pass (see Figs. 6A.150 through 6A.152), normally with nearby site materials. In this case, the ground surface is lowered by an amount commensurate with the degree of densification achieved during that pass. In some situations, for example when the groundwater table is high, it may be desirable to keep the ground surface at the same elevation, which necessitates the use of imported materials to backfill the craters and the intermediate areas that have settled.

The final phase is called the *ironing phase* or *low energy* phase and is intended to densify the surficial soils to a depth of about 6 ft (2 m). A square tamper, low drop height, and low contact pressure are typical for this phase. The whole treated area is densified on a contiguous or overlapping pattern. If the depth of the craters is less than about 1.5 ft (0.5 m), the surficial soils generally can be densified by conventional compaction rollers rather than by dropping tampers.

If the compressible layer or layers is deep, it is sometimes necessary to use one or more *intermediate phases* to densify the intermediate depths. If one intermediate phase is used, the intermediate drop points are centered between the primary drop points.

6A.4.1.3 Soil Type and Moisture Condition

The following characteristics of the soil significantly influence the effectiveness of dynamic compaction:

1. Classification, geologic origin, and layering of the soil mass to be compacted
2. The moisture condition (degree of saturation) of the soil
3. For saturated soil, the permeability of the soil and length of drainage paths, which control the rate of dissipation of induced excess pore water pressures

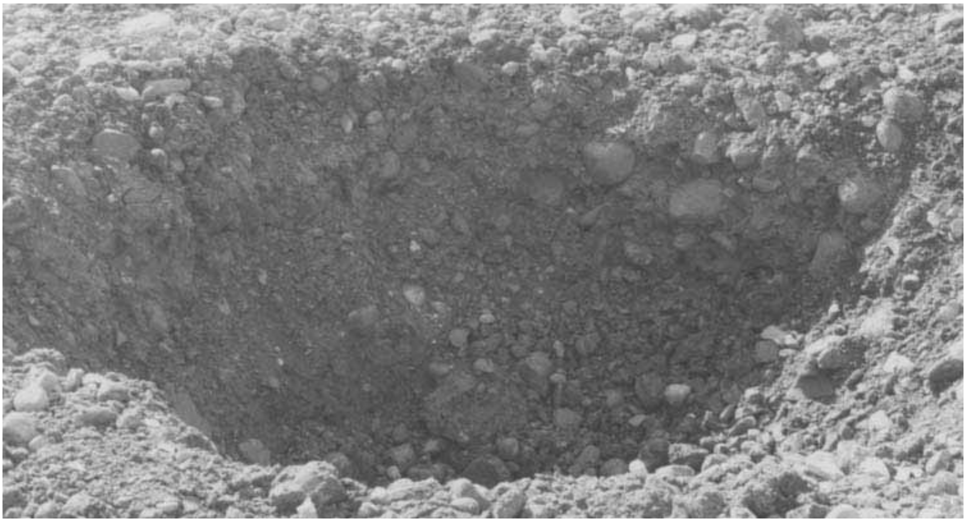


FIGURE 6A.150 Crater created during primary phase.

Lukas (1986) has classified the suitability of soil for dynamic compaction into the following three zones based on grain-size distribution (Fig. 6A.153): Pervious soils (Zone 1), semipervious soils (Zone 2), and impervious soils (Zone 3).

Soils with high degrees of saturation, high permeabilities, and good drainage are best suited for dynamic compaction. The method is generally effective in all types of soils that are partially saturated with a continuous air phase within the voids. When energy from the tamper impacting the surface is transmitted within the ground, densification of the soil occurs as air is squeezed from the voids.

FIGURE 6A.151 Dumping backfill into crater.

FIGURE 6A.152 Leveling backfilled crater.

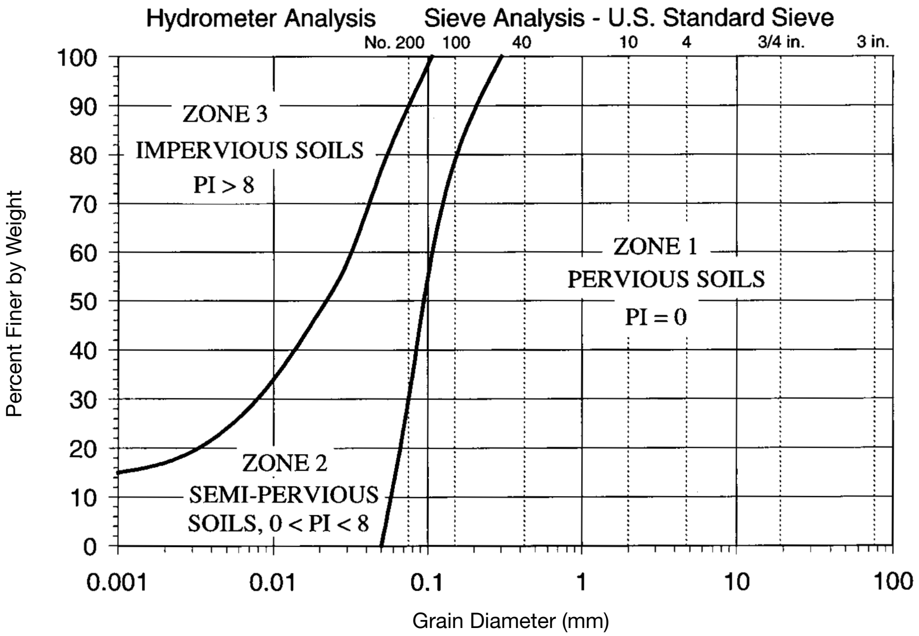


FIGURE 6A.153 Grouping of soils according to effectiveness of dynamic compaction (redrawn from Lukas 1986).

Air rather than water is squeezed from soils with a low degree of saturation because the viscosity of air is several orders of magnitude lower than the viscosity of water. A continuous air phase typically exists within the voids of a soil when the degree of saturation is less than about 70 to 95%, with the lower values usually representative of sandy soils and higher values representative of clayey soils. At higher degrees of saturation, the air in the voids occurs as bubbles trapped within water in the voids and the soil behaves as if it is saturated. (See Fig. 6A.85 and the related discussion for further clarification.)

Zone 1 materials are pervious, with typical values of coefficient of permeability (k) greater than 1×10^{-3} cm/sec (3 ft/day). Soil particle sizes range from boulders to sands. Also included within this category are deposits consisting of building rubble, construction debris, coarser mine spoils, some industrial waste fill such as slag and clinker, and decomposed refuse deposits. Dynamic compaction is effective in all Zone 1 deposits, even if saturated, because the permeability is sufficiently high to dissipate excess pore water pressures almost immediately. Hence densification is also nearly immediate.

Zone 3 materials are nearly impervious to water [generally, $k < 1 \times 10^{-6}$ cm/sec (3×10^{-3} ft/day)] and are generally not suitable for dynamic compaction if saturated or nearly saturated. Soils included in this category consist of clays or clayey soils with clay content higher than about 25% and plasticity index (PI) greater than 8. Induced excess pore pressures in these soils typically require long periods of time to dissipate, and densification is unduly delayed. Exceptions to this rule may occur when the drainage paths are very short, such as occurs in varved clays and where macroscopic features (animal burrows, holes resulting from decomposition of organic material, etc.) allow faster drainage. Some improvements have been achieved in partially saturated clays where the water content is less than the plastic limit. Densification will occur until the deposit becomes essentially saturated by the compaction process. No further improvement will be produced after saturation occurs, regardless of the amount of energy that is applied.

Zone 2 materials are semipervious with values of k usually ranging from 1×10^{-3} to 1×10^{-6} cm/sec (3×10^{-3} to 3×10^{0} ft/day). These materials include silts, sandy silts, silty sands, and clayey silts with PI < 8. Densification generally occurs readily in these materials when partially saturated. However, when saturated, induced excess pore water pressures may require days or weeks to dissipate completely. Therefore, tamping operations must be planned and conducted carefully. It is prudent to perform permeability tests beforehand to establish the suitability of these materials for dynamic compaction. Usually, the energy must be applied in multiple phases or multiple passes, with sufficient time in between to allow excess pore water pressures to dissipate. Alternately, wick drains can be used to facilitate drainage, but the cost-effectiveness of this combined method has not been established relative to other improvement techniques (Dise et al. 1994).

Obviously, there are soils that will not fall entirely within one of the three zones. In these cases, permeability tests should be conducted on the soil because permeability is a better indicator than grain-size distribution of the behavior of the soil during dynamic compaction.

Rollins et al. (1998) showed that the concept of optimum moisture content (OMC) is valid for dynamic compaction. This is to be expected, since dynamic compaction is an impact method similar to that used in Proctor tests. The following conclusions were drawn from their research:

1. OMC increases with depth because the compactive energy produced by the dynamic compaction decreases with depth.
2. The depth of improvement increased somewhat as the moisture content increased.
3. The crater depth increased as the moisture content increased. At moisture contents above optimum, crater depths became excessive and did not always reflect greater improvement.

These results suggest that the efficiency of dynamic compaction can be improved by controlling the moisture condition of the soils being treated. However, owing to the difficulty in controlling moisture content in the field for the large volumes of soils typically treated in any dynamic compaction, this may not be feasible for many dynamic compaction projects.

6A.4.1.4 Design Guidelines

The five steps shown below need to be undertaken during the design process for any dynamic compaction project (Lukas 1995). Other steps may be needed for particular projects.

1. Selection of tamper weight, shape, and drop height to achieve the desired depth of treatment
2. Determination of the applied energy required over the project site to obtain the desired improvement
3. Selection of the area(s) to be treated
4. Determination of the grid spacing along with the number and chronological sequence of phases, and the number of passes in each phase
5. Determining if a surface stabilizing layer is needed

Each of these steps is described in more detail in the following sections.

Tamper Characteristics and Drop Height. The weight of the tamper and the height of drop are not independent parameters, as shown in Fig. 6A.154. The theoretical energy applied by one drop of the tamper is equal to $W \times H$. However, there are energy losses associated with cable drag and wind resistance on the order of about 10% of the theoretical energy. Therefore, the actual applied energy is about 90% of the theoretical energy. The energy required to achieve a particular depth of treatment at a site can be achieved by infinite combinations of W and H.

Mayne et al. (1984) established an empirical trend between apparent maximum depth of improvement (D_{max}) and theoretical energy per blow (WH) shown in Fig. 6A.155 based on data from numerous projects. From this data, the following equation can be used to estimate D_{max}:

$$D_{max} = n\sqrt{\frac{WH}{f}} \tag{6A.62}$$

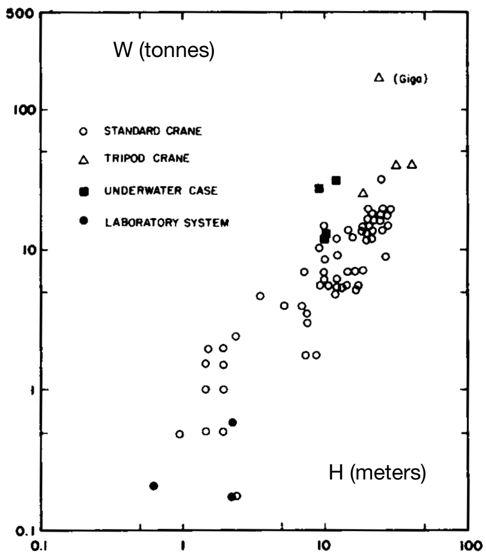


FIGURE 6A.154 Relationship between weight of tamper and drop height (from Mayne et al., 1984).

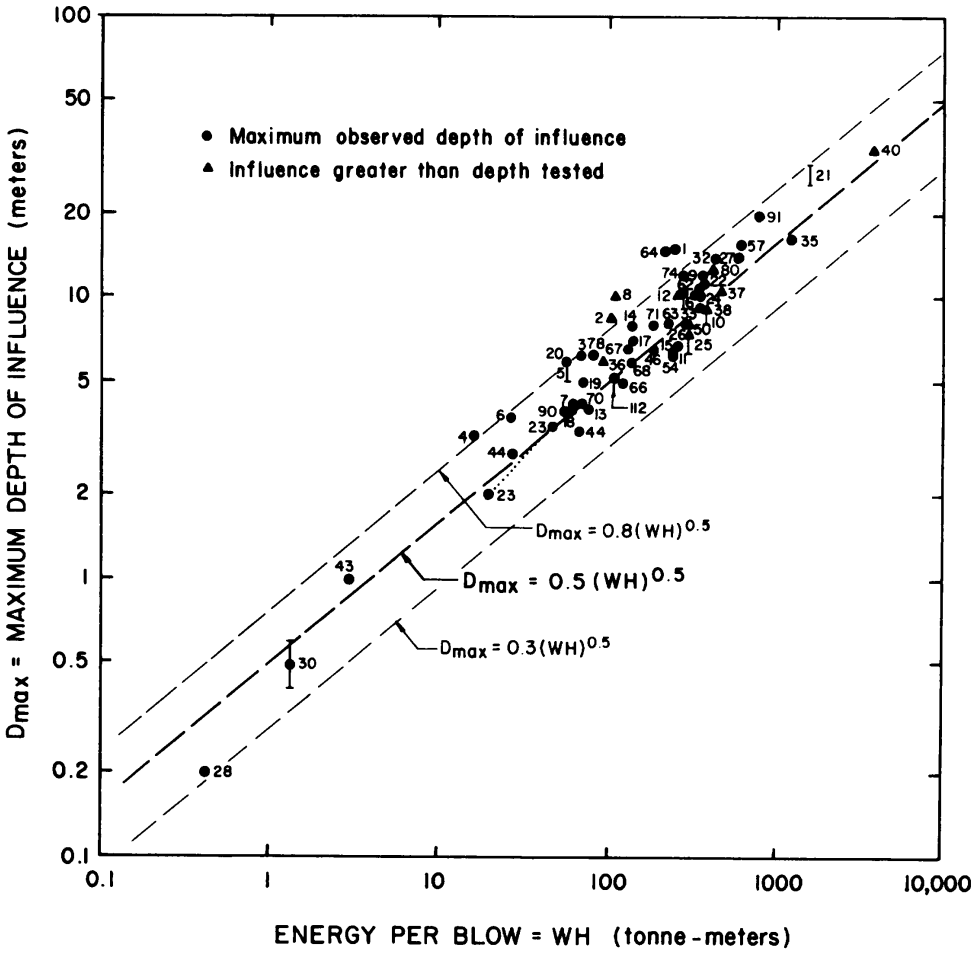


FIGURE 6A.155 Trend between apparent maximum depth of influence and theoretical energy per blow (from Mayne et al., 1984).

where n = empirical coefficient ranging from 0.3 to 0.8, averaging about 0.5
f = units factor = 1.0 tonne/m = 9.807 kN/m = 672 lb/ft

The variation in n can be attributed to the following factors (Lukas 1995):

1. Efficiency of the drop mechanism of the crane
2. Magnitude of total applied energy
3. Types and layering of soil being densified
4. Possible presence of energy absorbing layers
5. Presence of a hard layer below or above the material being treated
6. Contact pressure induced by the tamper on the ground

Lukas (1995) provided recommended values of n as a function of soil type and degree of saturation, which are summarized in Table 6A.13.

There is a critical depth at which the improvement is the greatest (D_c), which occurs at about one-half the maximum depth of improvement:

$$D_c \approx 0.5D_{\max} \tag{6A.63}$$

Use of values of n from Table 6A.13 in Eq. 6A.62 generally provide a reasonable estimate of $D_{\max}$. However, the following factors may affect the maximum depth of improvement actually achieved and, if present on a particular project, may require adjustment of the tamper characteristics and drop height to ensure the soil is densified to the desired depth:

1. The tamper used during the high energy phases generally should have a flat bottom and a contact pressure (tamper weight divided by area of base) in the range of 800 to 1,550 lb/ft^2 (40 to 75 kPa). If the contact pressure is significantly higher, the tamper could punch into the ground with little densification of the underlying soils. Smaller contact pressures generally produce densification of the surficial soil only and should only be used during the ironing phase.
2. Weak saturated clay layers tend to absorb energy. Depending on the thickness of these energy-absorbing layers and their location within the zone to be treated, some reduction in $D_{\max}$ may occur. If the weak layer is thick and located near the middle of the zone to be treated, the soils underlying this layer will not be improved. If the weak layer is thin and located near the ground surface, it is possible that the tamper will penetrate this layer and densify the underlying materials. The influence of a weak layer or layers on the depth of improvement is best determined by field tests wherein in situ tests are conducted after trial dynamic compaction to ascertain the effect of the weak layer(s).
3. Hard or cemented layers tend to distribute the impact load over a larger area, lessening the energy transmitted to underlying layers. If located near the ground surface, the harder layer will need to be loosened prior to tamping. A hard layer located immediately below the zone to be improved has the beneficial effect of increasing the effectiveness of the treatment by reflecting energy back into the loose materials.
4. If the groundwater table is within about 6 ft (2 m) of the ground surface, effectiveness of the tamping will be reduced. Water can rise up into the craters as a result of induced excess pore water pressures. Repeated tamping may cause liquefaction within the surficial soils. This weak upper zone will not densify and will not efficiently transmit energy beneath it. The water table can be controlled either by lowering it through pumping or by placing fill to raise the ground surface.

TABLE 6A.13 Recommended Values of Empirical Coefficient n for Different Types of Soil and Degrees of Saturation (from Lukas 1986)

Soil type	Degree of Saturation	Recommended value of n^a
Zone 1—Pervious granular soils	High	0.5
	Low	0.50 to 0.60
Zone 2—Semipervious primarily silts with PI < 8	High	0.35 to 0.40
	Low	0.40 to 0.50
Zone 3—Impervious primarily clays with PI > 8	High	Not recommended
	Low	0.35 to 0.40^b

[a]For an applied energy of 1 to 3 MJ/m^2 (69 to 206 ft-kips/ft^2) and a tamper drop using a single cable with a free spool drum.

[b]Soils should be at a water content less than the plastic limit.

Applied Energy. The energy applied per unit volume of treated soil (E) can be calculated from the following equation:

$$E = \frac{NWHP}{S^2_{d\max}} \tag{6A.64}$$

where N = number of drops at each drop point
P = number of passes
S = spacing of drop points on a square grid pattern

Guidelines for applied energy requirements as a function of type of deposit to be treated are given in Table 6A.14. The range in values in Table 6A.14 accounts for the initial relative density of the deposit to be treated and the required degree of improvement. The higher values in any range should be used for loose deposits or a high required degree of improvement and the lower values for dense deposits or a low required degree of improvement. The energy per unit area to be applied to the surface of the deposit can be obtained by multiplying the selected value of E by the thickness of the deposit to be densified.

Characteristics of Phases. Dynamic compaction undertaken on Zone 1 materials with any degree of saturation or Zone 2 materials with low degrees of saturation can generally be completed in one high-energy phase. Saturated Zone 2 materials and Zone 3 materials with low degrees of saturation usually required two or three high-energy phases. In the first phase, tamping is conducted at every second or third drop point. In the second or third phases, tamping is performed at intermediate points after the excess pore water pressures have dissipated significantly. Piezometers should be installed in the treated ground and monitored to determine when additional tamping can be conducted.

It is usually economical to use between about 7 and 15 drops at each impact point for any pass of a high-energy phase. The number of drops required at each impact point can be determined from Eq. 6A.64. If the required number of drops does not fall within the economical range, the spacing of the impact points can be adjusted.

The number of drops that can be applied at any impact point may be controlled by the development of excess pore water pressures or the depth of the craters. In general, the crater depth should be no more than the height of the tamper plus about 1 ft (0.3 m) for the following reasons (Lukas 1995):

1. Extracting the tamper from a deep crater is difficult because suction forces may develop and loose debris may fall on top of the tamper. This may result in cable damage and lost production time.
2. After extraction of the tamper, the sides of the crater may cave in, causing two types of problems: (a) the caved soil cushions additional impacts, and (b) the tamper may strike the caved material that accumulates along the sides, lessening the amount of energy that is transmitted deeper.
3. Applying the energy at a deeper level may generate higher excess pore water pressures.

TABLE 6A.14 Guidelines for Applied Energy Requirements (from Lukas 1986)

Type of deposit	Required E ft-kips/ft^3	Required E kJ/m^3
Zone 1	4.2 to 5.2	200 to 250
Zone 2; Zone 3 above the groundwater table	5.2 to 7.3	250 to 350
Landfills	12.5 to 23.0	600 to 1,100

4. The loosened soil above the base of the crater requires that a higher level of energy be used during the ironing phase.

Fig. 6A.156 can be used to estimate a range in expected crater depths (δ) as a function of the theoretical applied energy (*WH*) and the number of drops at each impact point.

In saturated Zone 2 and Zone 3 materials, which have moderate to low permeabilities, significant excess pore water pressures can be generated, which may required days or weeks to dissipate. These excess pore water pressures are sometimes sufficient to cause surface boils, and may result in water rising into the craters or ground heave adjacent to the craters. When high excess pore water pressures develop, additional applied energy primarily produces distortion of the soil mass (change in shape without change in volume). Hence, no significant densification occurs. Heaving of the ground surface, as illustrated in Fig. 6A.157, is an indication that distortion rather than densification is occurring and that tamping should be stopped at that location. In this case, the energy should be applied in multiple passes to allow dissipation of the excess pore water pressures prior to additional tamping at those impact points.

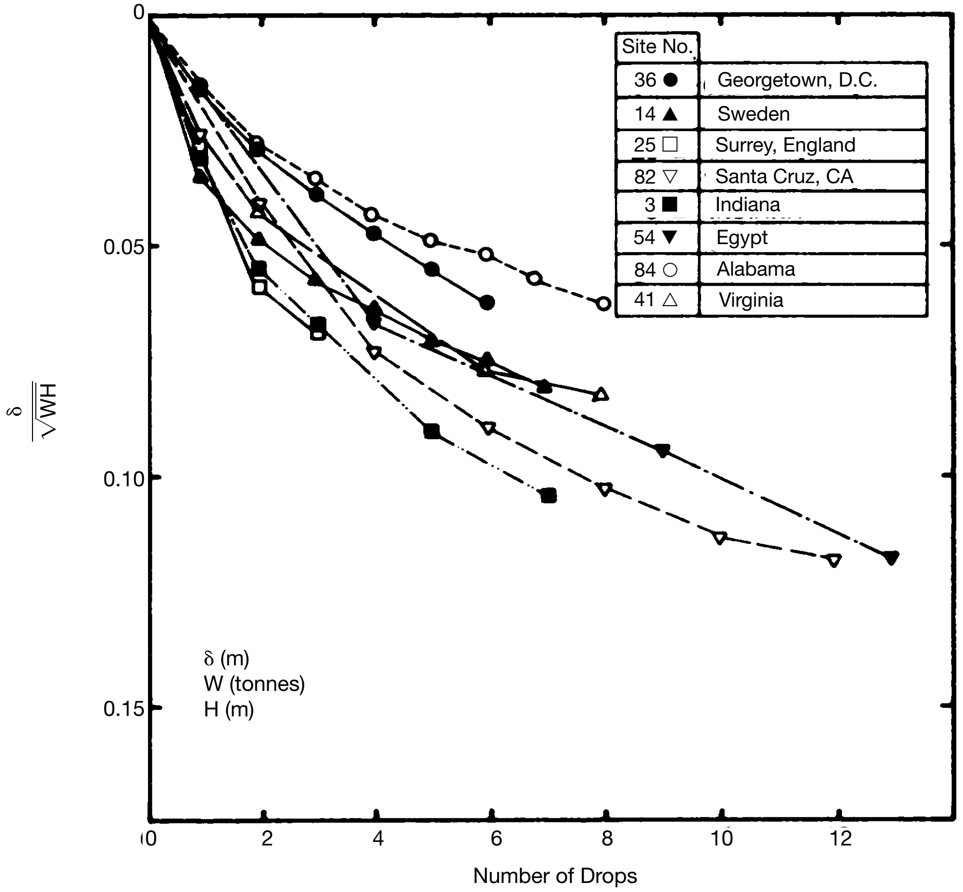


FIGURE 6A.156 Normalized crater depths (from Mayne et al., 1984).

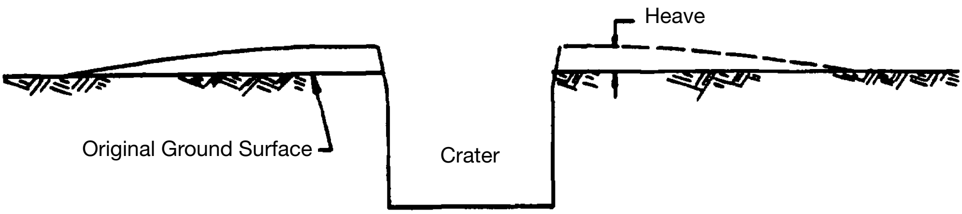


FIGURE 6A.157 Heaving of ground surface adjacent to a crater (from Lukas, 1995).

Treated Area. In many projects, the treated area is larger than the area used for foundation support. In general, the distance the treated area extends beyond the edges of the loaded area is between one-half to one times the thickness of the weak deposit being treated. In some cases, the treated area may need to be larger than this. For example, when an embankment is constructed on a dynamically compacted area where slope stability is a concern, it is necessary to densify the soil beyond the toe a distance greater than the distance at which the potential critical failure surface will intersect the ground surface. If some areas are to be more heavily loaded than others, additional tamping can be performed at these locations after the entire grid area has already been tamped. For example, additional tamping has been conducted on some building projects at the column locations after tamping of the entire treated area has been completed.

Surface Stabilizing Layer and Control of Groundwater. When the surficial soils are soft, a granular stabilizing layer should be placed on the surface to provide a working mat. This mat serves two primary purposes—it prevents sticking of the tamper and it limits the depth of penetration of the tamper into the soft material. The thickness of the stabilizing layer should be determined such that the mat provides adequate stability but does not significantly reduce the magnitude of energy transmitted to the soil to be treated.

When the groundwater table is less than about 6 ft (2 m) from the ground surface, the bottom of the craters may reach below the water table, reducing the effectiveness of the dynamic compaction. In this case, three methods of control can be used: (1) Raise the grade by adding a granular layer, (2) dewater the site to lower the groundwater table before tamping commences, or (3) pump water from the craters between tamps and then add granular material to keep the impact elevation higher.

6A.4.1.5 Ground Vibrations, Lateral Displacements, and Airborne Debris

When a tamper impacts the ground, vibrations are transmitted through the ground in all directions, as illustrated in Fig. 6A.147. Vibrations transmitted along and near the ground surface may travel large distances and affect nearby structures and occupants of adjacent buildings. The magnitude of the vibrations and the distance they travel increase with increasing impact energy. It has been determined that damage to structures and annoyance to people is more closely associated with particle velocity than either displacement or acceleration (Nicholls et al. 1971). Therefore, peak particle velocity (PPV) is the parameter generally used to estimate the magnitude of potential problems from vibrations. However, frequency of vibration also is an important factor in damage and annoyance. The frequency of ground vibrations from dynamic compaction is generally in the range of 6 to 10 Hz (Lukas 1995).

The expected magnitude of PPV from dynamic compaction should be estimated beforehand at any location where potential damage to adjacent structures or annoyance to occupants of nearby buildings may occur. Fig. 6A.158, which shows PPV as a function of scaled energy factor for various types of ground materials, can be used for this purpose. In addition, the following equation can be used to estimate a conservative upper limit of PPV as a function of energy and distance (Mayne et al. 1984):

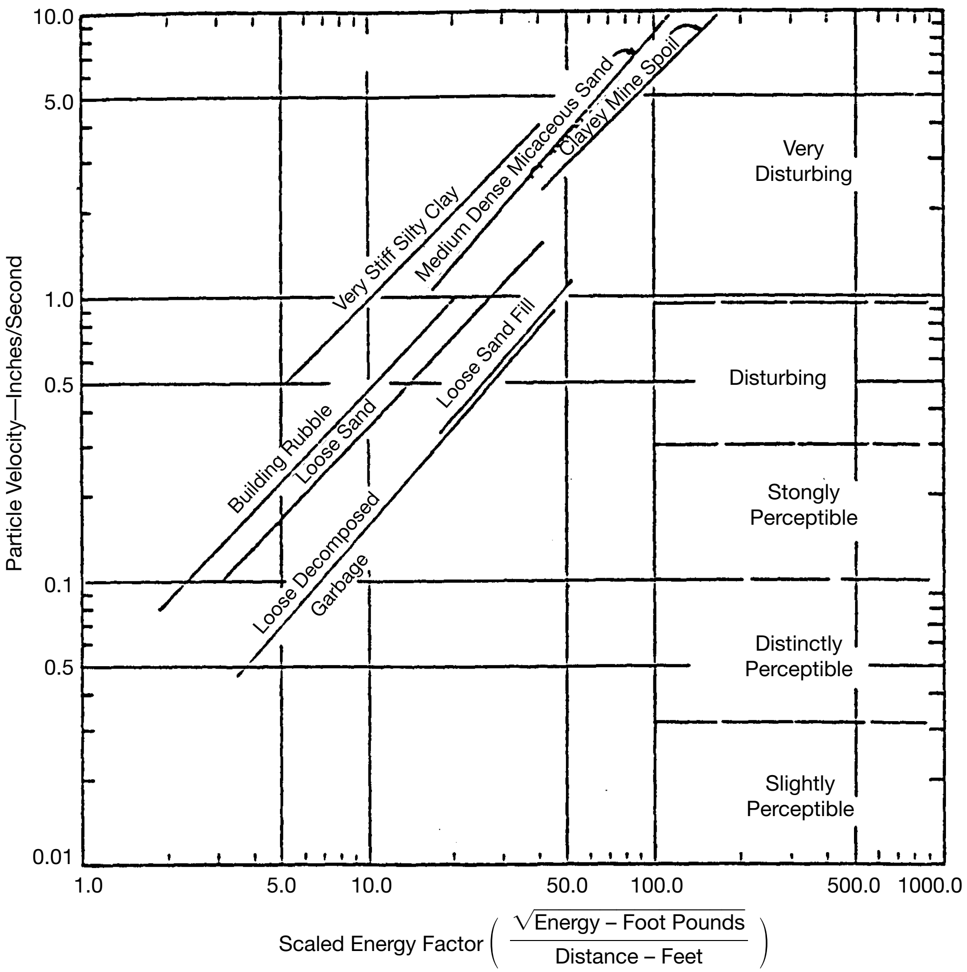


FIGURE 6A.158 PPV versus scaled energy factor for various types of ground materials (from Lukas 1986).

$$PPV\,(\text{in/sec}) \le 3.64\left(\frac{\sqrt{WH}}{d}\right)^{1.4} \qquad \text{for } W \text{ in kips, and } H \text{ and } d \text{ in ft} \qquad (6A.65a)$$

$$PPV\,(\text{mm/sec}) \le 70\left(\frac{\sqrt{WH}}{d}\right)^{1.4} \qquad \text{for } W \text{ in tonnes, and } H \text{ and } d \text{ in m} \qquad (6A.65b)$$

where d is the distance from the impact point. It has also been found that PPV tends to increase with the number of blows from a tamper at a particular location as the underlying materials densify.

Levels of annoyance to people are given in Fig. 6A.158 as a function of particle velocity, with categories ranging from slightly perceptible to very disturbing. Within the normal range of vibration frequencies produced by dynamic compaction, cracking of plaster and drywall in structures occur at particle velocities of about 0.5 in (13 mm) and 0.75 in (19 mm), respectively (Lukas 1995). Structural damage occurs at particle velocities greater than about 2 in (51 mm). In some cases, the ground vibrations can be substantially reduced by digging a trench to a depth of about 6 to 10 ft (2 to 3 m) between the impact point and the structure or area of concern (for example, see Thompson and Herbert 1978). An open trench is most effective in reducing vibrations but may be unacceptable for many reasons. In situations where an open trench is not feasible, the trench should be filled with loose granular soil or some other compressible material.

An assessment of preexisting cracking of adjacent houses and other structures should be conducted prior to beginning dynamic compaction, in conjunction with the owners of these structures, if it is expected that damage might occur. Actual PPVs produced by dynamic compaction should also be measured on the ground by adjacent structures using a portable field seismograph. If these steps are not taken, it may be impossible to determine whether cracking of these structures occurred before or during dynamic compaction and may cause unnecessary legal problems for the contractor.

Some permanent lateral displacements occur adjacent to the impact point. Although no reliable method has yet been established to predict the magnitude and distribution of lateral displacements with distance from the impact zone, some general relationships have been established (Lukas 1986, 1995):

1. Lateral displacements decrease with distance from the point of impact.
2. PPVs up to 3 in/sec (76 mm/sec) have not damaged buried utilities. Pressure pipelines have withstood PPVs of 10 to 20 in/sec (250 to 500 mm/sec) without distress.
3. For tampers in the range of 33 to 66 kips (147 to 294 kN), dynamic compaction should not be conducted within 25 ft (7.6 m) of any buried structure located within the upper 30 ft (9.1 m) of the ground mass if movement could cause damage.

Debris sometimes becomes airborne when the tamper impacts the ground. Rubble fills and landfills may produce substantial amounts of flying debris, owing to the large particles being impacted. Fine-grained soils may produce dust if dry or flying mud if wet. There are generally little or no problems with airborne debris in granular soils. Mitigation measures include the wearing of hardhats for all on-site personnel, maintaining a safe distance from the impact point, and erecting protective shields adjacent to the impact location.

6A.4.1.6 Verification of Improvement

Verification of the degree of improvement obtained by the dynamic compaction process is generally determined from in situ testing performed before and after the treatment. The most common methods are the standard penetration test (SPT), cone penetration test (CPT), and the pressuremeter test (PMT). Other methods that have been used include measurement of the increase in unit weight from samples, measurement of the deceleration of the tamper during impact using an accelerometer, load-deformation data from load tests, and measurement of increase in shear velocity from the cross-borehole seismic test.

The SPT and CPT are relatively insensitive to changes in stiffness of the soil compared to the PMT. Therefore, the PMT is generally a better test to provide information on the degree of improvement obtained by the treatment process. In many cases, the primary purpose of the dynamic compaction is to increase the stiffness of the soil, which is directly measured in the PMT. Another good method for determining the increase in stiffness of the soil is from load tests such as the plate load test (see section 6A.3.5.2 for additional details) and larger-scale load tests

Since dynamic compaction remolds the treated soil, the strength and stiffness of the soil will increase with aging after the treatment ceases. On average, an increase in strength and stiffness on the order of 50 to 100% owing to aging can be expected during engineering times. Further details on the effects of aging on compacted soils can be found in section 6A.3.4.3. Much of the aging-induced improvement occurs in a few weeks in many cases but can continue for months and years afterward. In moderate and low permeability saturated soils, improvement also occurs as excess pore water pressures dissipate. Thus, it is advisable to delay post-treatment testing as long as possible. If testing is performed immediately after treatment, additional testing should be performed a month or more later. The ideal posttreatment testing pattern is to test immediately after treatment, then 1, 2, 4, 8, 16, 32, etc. days afterward so that the long-term effects can be predicted from statistically reliable short-term data. Unfortunately, this is not always economically or logistically feasible.

6A.4.1.7 Case History

The following case history is condensed from Lukas (1995) to illustrate some important aspects of dynamic compaction. Additional details can be found in the reference.

A three-story structure was to be constructed on an 86,000 ft^2 (8,000 m^2) site in Florida. Although the structural loads were relatively light, the initial subsurface exploration indicated that sinkholes and voids caused by dissolution of limestone formations were present at the site. The geologic profile was relatively heterogeneous. The predominant soil type was a silty fine sand grading to a fine sand with seams of sandy clay. At some locations there was some cementation in the silty sand, making it substantially stiffer and stronger than the uncemented zones. Preliminary analyses indicated that settlement across the structure would range from 0.9 to 2.9 in. (23 to 74 mm), assuming that no voids or sinkholes were present within the bearing soils. The predicted differential settlement of 2.0 in. (51 mm) was considered to be unacceptable from a structural standpoint. Furthermore, the possible presence of large voids within the bearing materials was considered to be an unacceptable risk. Therefore, dynamic compaction was undertaken to improve the foundation bearing materials, eliminate voids to depths of 25 to 30 ft (7.6 to 9.1 m) below ground surface, reduce total settlements, and reduce differential settlements of the shallow foundations.

The dynamic compaction contractor used a 16.5 ton (147 kN) tamper and a drop height of 66 ft (20 m). From Eq. 6A.62, backcalulated values of n from 0.44 to 0.53 would be required to produce densification to the required depth of 25 to 30 ft (7.6 to 9.1 m). These values of n are reasonable. Since the soils at the site were predominantly Zone 2 materials, multiple passes and phases were needed to allow induced excess pore water pressures to dissipate. In addition, additional energy was applied at two locations where large ground depressions occurred, which was indicative of the presence of voids or sinkholes at those locations. Details of the dynamic compaction procedures and resulting induced settlements are summarized in Table 6A.15.

The total applied energy per unit area was approximately 111 ft-kips/ft^2 (1.63 MJ/m^2). Dividing the energy per unit area by the desired maximum depth of improvement of 30 ft (9.1 m) yields a value of energy per unit volume of 3.7 ft-kips/ft^3 (180 kJ/m^3), which falls below the recommended range of 5.2 to 7.3 ft-kips/ft^3 (250 to 350 kJ/m^3) given in Table 6A.14 for Zone 2 materials. Pressuremeter tests conducted before dynamic compaction and 21 days after the treatment was completed showed that the dynamic compaction produced substantial increases in both limit pressure and pressuremeter modulus at all depths (Fig. 6A.159).

TABLE 6A.15 Details of Dynamic Compaction Procedures Used in Florida Project

Phase	Pass	Spacing of impact points: ft	Spacing of impact points: m	Blows per point	Energy: ft-kips/ft^2	Energy: kJ/m^2	Induced settlement: in	Induced settlement: mm
1	1	30	9.1	8	19.4	284	4.8	121
1	2	30	9.1	9	21.8	320	3.9	98
1	3	30	9.1	9	21.8	320	2.8	70
2	1	30	9.1	9	21.8	320	3.8	97
2	2	30	9.1	10	24.2	355	3.7	93
Ironing		Overlapping	Overlapping	1	2.42	35.5	3.5	89
7	Void	Sinkhole locations	Sinkhole locations					

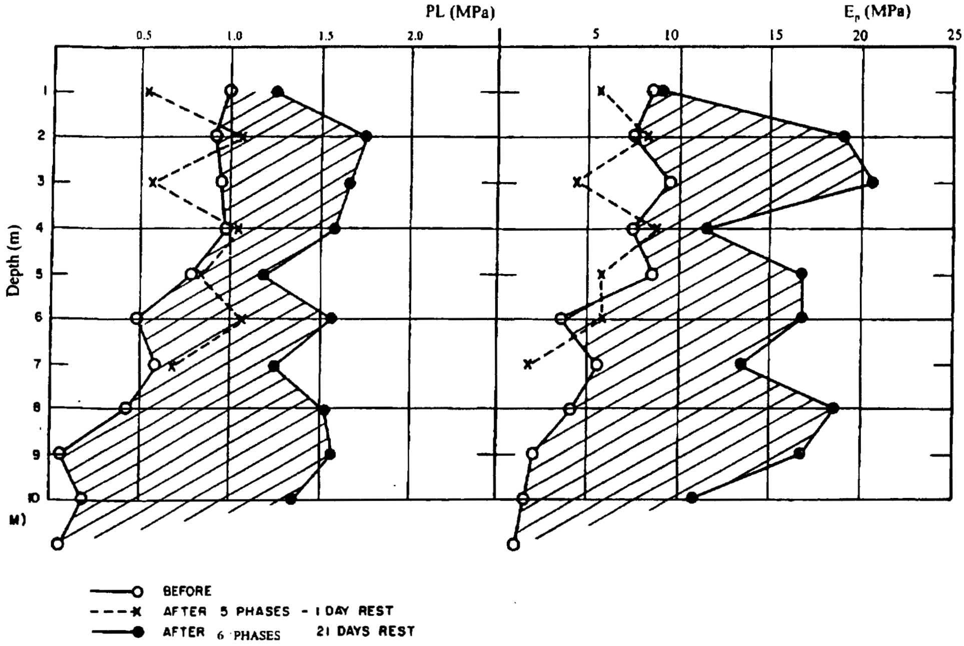


FIGURE 6A.159 PMT values before and after dynamic compaction (from Lukas 1995).

6A.4.2 Vibro-Compaction

In vibro-compaction, a vibratory probe is inserted into granular soils to densify them at depth below the ground surface. A schematic illustration of a typical vibro-compaction process is given in Fig. 6A.160. Typical depths of densification range from 10 to 50 ft (3 to 15 m), but can be as shallow as 3 ft (1 m) and as deep as 120 ft (36 m) (ASCE 1997). Horizontal spacing of the compaction points depends on the type of soil being densified, the amount of densification desired, and the characteristics of the probe. Usual horizontal spacings are in the range of 5 to 12 ft (1.5 to 4 m). Applications of this method include increasing bearing capacity, reducing settlement, increasing shear strength in foundation soils for embankments and hydraulic fills, and reducing liquefaction potential of loose saturated sands and silts.

6A.4.2.1 Equipment and Procedures

The probe vibrates either horizontally or vertically to density the granular soil. The most common type of vibro-compaction is called vibro-flotation and was developed in Germany in the 1930s and brought to the U.S. in the 1940s. Vibro-flotation involves the use of a torpedo-shaped probe called a vibroflot, which densifies the soil by horizontal motion from the probe while in the ground. The vibroflot is a hollow steel tube with an eccentric weight inside that rotates to develop large horizontal forces. Typical vibroflots are 12 to 18 in (300 to 460 mm) in diameter, 6 to 16 ft (2 to 5 m) in length, vibrate at frequencies of 1200 to 3000 rpms with amplitudes of 0.5 to 1.5 in (13 to 38 mm), and weigh about 3 to 5 kips (13 to 22 kN). Extension tubes are slightly smaller in diameter than the probe and allow the treatment process to be taken to greater depths. The vibroflot is normally suspended from a 60 to 100 ton (530 to 900 kN) crawler crane. A flexible coupling is used to isolate the vibroflot from the extension tubes so that the vibrations are not transmitted up the tubes to the crane.

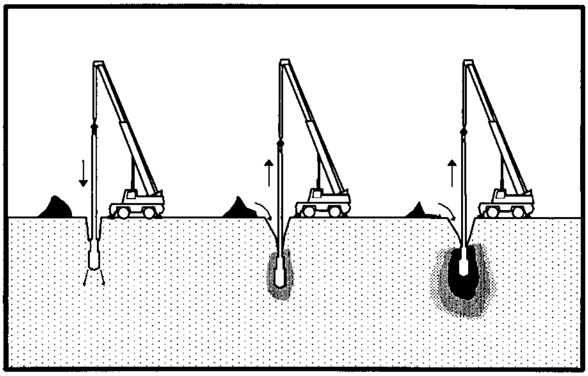


FIGURE 6A.160 Schematic illustration of typical vibro-compaction process (courtesy of Hayward Baker Inc., Odenton, Maryland).

The typical procedure used to densify the soil at depth is as follows:

1. The vibroflot is lowed to the desired depth by vibration, sometimes with the assistance of pressurized air or water jets located in the nose of the probe.
2. The vibroflot continues to vibrate at one depth, creating a cavity around the probe. This cavity is continuously filled with granular soil introduced either from the ground surface (top-feed method) as shown in Fig. 6A.160 or through feeder tubes directed to the tip of the probe (bottom-feed method). In the top-feed method, upper water jets can be used to facilitate getting the fill soil to the bottom of the cavity.
3. The vibroflot is retracted at about 1 ft (0.3 m) intervals, leaving a densified column of granular soil.
4. This procedure is repeated at other specified locations. The treated area can consist of isolated locations (for example to support individual footings) or full treatment of the area by compacting the soil on a grid pattern that allows overlapping of the densified zones.

Several other techniques use a top pile-driving vibrator that imparts vertical vibrations to the ground. Techniques utilizing this method include Terra-Probe, Vibro-Wing, and Tri-Star or Y-Probe (ASCE 1997). For example, the Terra-Probe method transmits vertical vibrations down an attached pipe with a diameter of about 30 in. (760 mm). Vertical vibration is generally less efficient that horizontal vibration, especially in finer-grained granular soils, and typically requires closer spacing of the compaction points.

6A.4.2.2 Soil Types and Technical Considerations

Vibro-compaction works best in loose granular soils, particularly clean sands and silty sands with less than about 20% fines and less than about 3% clay. The approximate range of grain-size distribution curves of granular soils suitable for densification by vibro-compaction is shown in Fig. 6A.161. The same procedures can be used in other types of soils with gravel as the backfill material, resulting in the creation of vibro-stone columns. Vibro-stone columns are described in Section 6A.5.

The radial extent of the densified soil depends on the characteristics of the soil being treated and the applied energy. The relative density of the treated material is highest at the periphery of the probe and decreases radially outward. Typical ranges of treated area per compaction probe as a function of the relative density obtained are shown in Fig. 6A.162. If treatment is conducted to the ground surface, the upper 3 to 6 ft (1 to 2 m) may require additional densification by surface rolling.

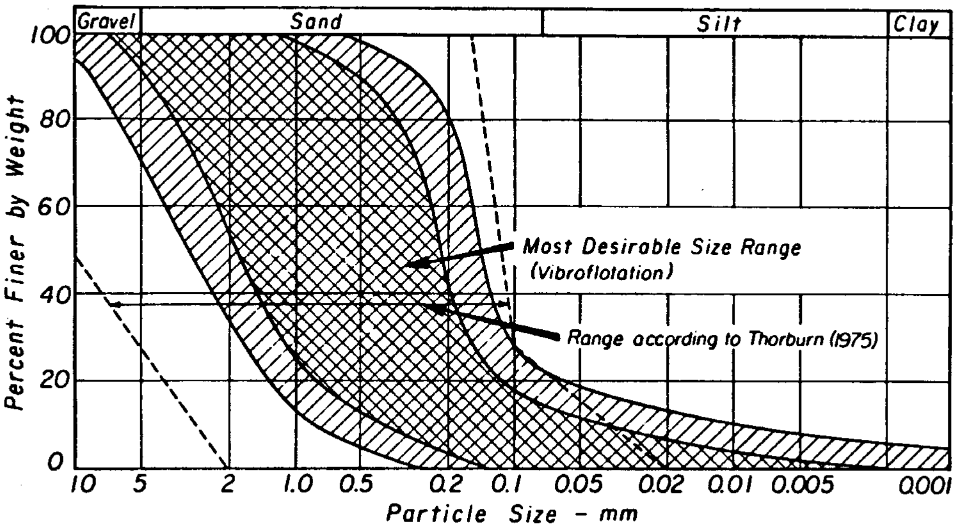


FIGURE 6A.161 Range of particle size distributions suitable for densification by vibro-compaction (from Mitchell 1981).

Verification of the effectiveness of the treatment is done by conducting pretreatment and posttreatment SPTs or CPTs.

6A.4.3 Blast Densification

The detonation of buried explosives can provide a rapid and sometimes economical means of densifying deep deposits of granular soils. This process is called blast densification or explosive compaction. Saturated loose sands and loessial soils are particularly suited to densification by blasting.

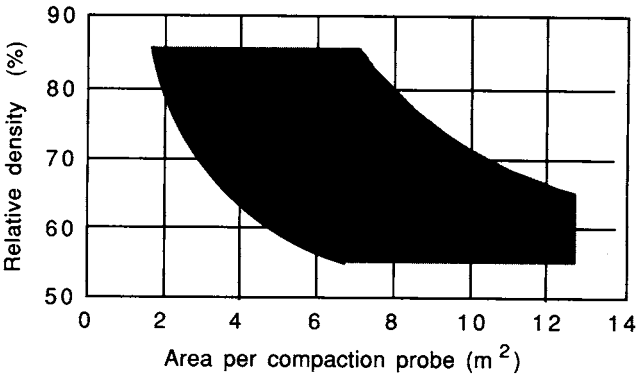


FIGURE 6A.162 Envelope for spacing of vibro-compaction centers in clean granular soils (from Hausmann 1990).

Although this technique has been used successfully for over 60 years in many different soil types under a variety of site and environmental conditions, it has not achieved general acceptance in civil engineering (Narin van Court and Mitchell 1995).

6A.4.3.1 Mechanisms of Explosion and Densification

To densify the soil effectively, the explosion must create sufficient energy to destroy the existing structure of the soil and rearrange the particles into a denser configuration. The explosion and densification process is summarized below from Narin van Court and Mitchell (1995). Additional details can be found in the reference.

The energy from the explosion is released in two forms—shock energy and gas energy. Shock energy is produced because the rate of reaction in the explosion is greater than the speed of sound in the soil, thereby forming a shock that impacts the surrounding soil. The stress applied to the soil by the shock wave is called detonation pressure. Gas energy is generated as the gaseous reaction products expand from their initial volume (volume of the charge) to an equilibrium volume dictated by the weight of the explosive charge and the confining pressure provided by the soil. The stress applied to the soil by the expanding gases is called the gas or explosion pressure. Gas energy typically accounts for 85% of the useful energy released by the explosive in rock blasting, and in soils the proportion may be higher.

Densification of granular soils occurs by many mechanisms, including compression, volumetric strains, shearing, and inducement of liquefaction resulting from generated excess pore water pressures. The generation of excess pore water pressures is an important part of the densification process. During liquefaction, the soil particles are suspended and then settle out into a denser configuration as the excess pore water pressures dissipate. Ivanov (1983) has indicated that full liquefaction (excess pore water pressures equal to the effective confining pressure) is necessary to achieve maximum densification. According to Barendsen and Kok (1983), the excess pore water pressure generated by the blasting should be greater than 80% of the effective overburden pressure for optimum densification. Vertical drains can be used to accelerate the rate of dissipation of excess pore water pressures (see Section 6A.6). The effect of shearing can be improved by setting off subsequent adjacent detonations while some excess pore water pressures remain. When adjacent charges are detonated while the soil is in this weakened state, the shear waves generated by subsequent blasts can readily cause additional particle movements and hence compaction.

6A.4.3.2 Procedures

The typical procedure used to densify soils by blasting is as follows

1. Pipes are installed at specified locations and depths by jetting, vibration, or other suitable means.
2. Charges are placed in the pipes.
3. The holes are backfilled.
4. The charges are detonated according to a pre-established pattern.

Three types of charges can be used: concentrated (point) charges, columnar charges that extend the full height of the soil layer, and short columnar charges called deck charges. The blasting procedure, layout, and timing of detonation of the charges are mostly empirical, owing to the many unpredictable factors involved.

Hansbo (1983) provided the following guidelines for the densification of loose, saturated sands by blasting:

1. The charges should be placed at approximately ½ to ¾ the desired depth of compaction, and in no case less than ¼ the depth.
2. The spacing between detonation holes should be about 15 to 50 ft (5 to 15 m), and in no case less than 10 ft (3 m).
3. The number of coverages is usually 2 to 3, which are separated by hours or days.

6A.4.3.3 Design Considerations

For blast densification to be effective, the soils must be cohesionless, loose, saturated, and freely draining. Narin van Court and Mitchell (1995) recommend using average normalized cone penetration tip resistance (q_{c1}) before treatment to determine the looseness of the soil and its suitability to be densified by blasting. q_{c1} is given by the following equation:

$$q_{c1} = q_{c,\mathrm{avg}} \cdot C_n \tag{6A.66}$$

where

$$C_n = \left(\frac{\sigma_{\mathrm{atm}}}{\sigma_{\mathrm{eff}}}\right)^{1/2} \leq 1.7 \tag{6A.67}$$

$q_{c,\mathrm{avg}}$ = average CPT tip resistance in 3 ft (1 m) intervals
σ_{atm} = atmospheric pressure in units consistent with σ_{eff}
σ_{eff} = effective confining stress at the middle of the interval

Estimation of the effectiveness of blast densification is based on initial (pretreatment) values of q_{c1}, designated q_{c1i}. Explosive compaction is nearly always effective when values of $q_{c1i} \leq 157$ ksf (7.5 MPa) and is generally ineffective when $q_{c1i} \geq 522$ ksf (25 MPa). At intermediate values of q_{c1i}, blast densification is sometimes effective and sometimes ineffective.

After determining that a site is suitable for blast densification, an engineer needs to develop a blast design that ensures the entire soil mass receives sufficient energy. The explosive energy input to the soil at any location is influenced by the strength of the charge, the distance from the detonation, and the confining pressure acting on the charge. The following equation can be used to estimate the magnitude of q_{c1} following treatment (designated q_{c1f}):

$$q_{c1_f} = 3.20 q_{c1i} (E_{i,\mathrm{total}})^{0.149} \tag{6A.68}$$

$$E_{i,\mathrm{total}} = \sum E_i \tag{6A.69}$$

$$E_i = \frac{W_e^{1/2}}{R_e \cdot \sigma_t} \tag{6A.70}$$

where E_i = explosive energy input for one charge
$E_{i,\mathrm{total}}$ = explosive energy input for all charges
W_e = weight of the charge, in equivalent weight of TNT
R_e = radial distance from the point of the explosion
σ_t = total confining stress at the depth of the charge, in kPa

Barendsen and Kok (1983) have indicated that the magnitude of generated excess pore water pressure and the degree of densification are related to Hopkinson's number (N_h) as follows:

$$N_h = 2.52 \frac{W_e^{1/3}}{R_e} \qquad \text{for } W_e \text{ in lb and } R_e \text{ in ft} \tag{6A.71a}$$

$$N_h = \frac{W_e^{1/3}}{R_e} \qquad \text{for } W_e \text{ in kg and } R_e \text{ in m} \tag{6A.71b}$$

Little or no liquefaction is expected to occur when N_h is less than about 0.09 to 0.15. This criterion and Eq. 6A.66 can be used to estimate the safe distance from the explosion. The following relationships can be used to estimate the the settlement of the ground surface (Δh) and the magnitude of the induced excess pore water pressure:

$$\frac{\Delta h}{h} = 2.73 + 0.9 \ln N_h \tag{6A.72}$$

$$\frac{u_e}{\sigma'_v} = 1.65 + 0.65 \ln N_h \tag{6A.73}$$

where h = height of the soil layer affected by the explosion
σ'_v = effective overburden stress

6A.4.3.4 Other Considerations

Other considerations in blast densification are summarized as follows:

1. Blasting can cause high very PPVs and therefore may damage adjacent structures or cause annoyance to occupants of adjacent buildings. Values of PPV higher than 12 in/sec (0.3 m/sec) have been reported from blast densification.
2. Zones of soil that are initially dense may be weakened by the blasting. However, if these dense zones are a small portion of the overall soil profile affected by the blasting, the resultant overall condition is likely to be satisfactory.
3. Settlement of the ground surface occurs almost immediately, with little additional settlement with time.
4. The surficial soil may be poorly compacted and may need to be excavated or compacted by some other method.
5. Gains in strength may continue with time after the blasting is complete, and even after all excess pore water pressures have dissipated. The majority of the gain in strength occurs within a few months after blasting, but may continue for several years after. This effect has not been observed at all sites treated by blast densification.

6A.5 GRANULAR COLUMNS

Several types of granular columns are currently used to improve bearing soils for shallow foundations in one or more of the following ways: (a) increase ultimate bearing capacity, (b) reduce compressibility, (c) increase the rate of settlement in saturated soils, (d) reduce liquefaction potential, (e) increase lateral resistance, and (f) increase uplift capacity. Granular columns can also be used to increase the stability of natural or fill slopes. The applicability of each type of granular column is summarized in Table 6A.16.

Although the methods of installation and types of columnar materials can vary significantly among the different types of granular columns, all types have the following basic features:

1. A single vertical, cylindrical column or group of columns consisting of granular or chemically stabilized granular material are created within the ground. Typically, about 10 to 40% of the volume of the native soil is replaced or displaced by the granular columns within the reinforced zone.
2. The columns are usually stronger, stiffer, and more permeable than the preexisting natural or fill soil into which they are installed (hereinafter called *matrix* or *native soil*).
3. When large areal coverage is provided, such as beneath a long embankment or a mat foundation, a triangular pattern is typically used (Fig. 6A.163). Sometimes a square pattern is used.

TABLE 6A.16 Comparison of Engineering Properties Improved by Various Granular Columns

Type of granular column	Type of improvement: Increase ultimate bearing capacity	Reduce magnitude of settlement	Increase rate of settlement	Provide uplift resistance	Increase lateral resistance	Reduce liquefaction potential	Increase slope stability
Stone column	Significant	Significant	Significant	None	Moderate	Significant	Significant
Sand column	Significant	Significant	Significant	None	Some	Significant	Significant
Geopier	Significant	Significant	Significant	Significant	Significant	Significant	Significant
Gravel drain	Some	Some	Significant	None	Not applicable	Significant	Not applicable
Sand drain	Some	Some	Significant	None	Not applicable	Significant	Not applicable

4. For support of individual footings, a variety of patterns can be used, depending on the shape and size of the footing, as well as the types and degrees of improvement needed in the soil. Some common patterns for individual footings are illustrated in Fig. 6A.164.

6A.5.1 Types of Granular Columns

The primary types of granular columns in common use today are *stone columns, sand columns, geopiers, gravel drains,* and *sand drains*. The important characteristics of each of these types are described in the following sections.

6A.5.1.1 Stone Columns

Stone columns are installed using either vibratory, rotary, or ramming techniques. The conventional stone column method was developed in Germany in the 1950s as an extension of the vibro-compaction process and is the most predominant technique in current use.

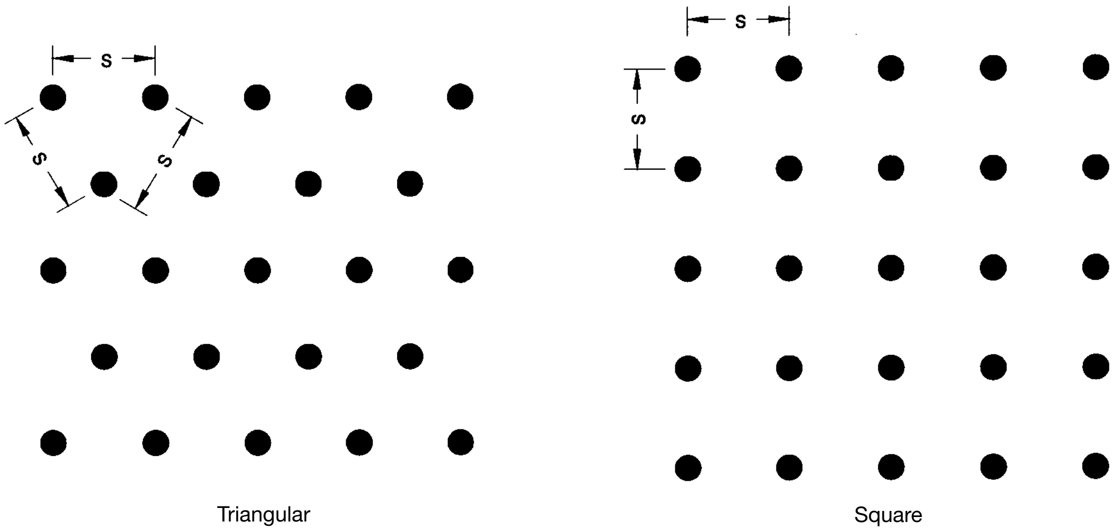


FIGURE 6A.163 Triangular and square patterns of columnar reinforcement.

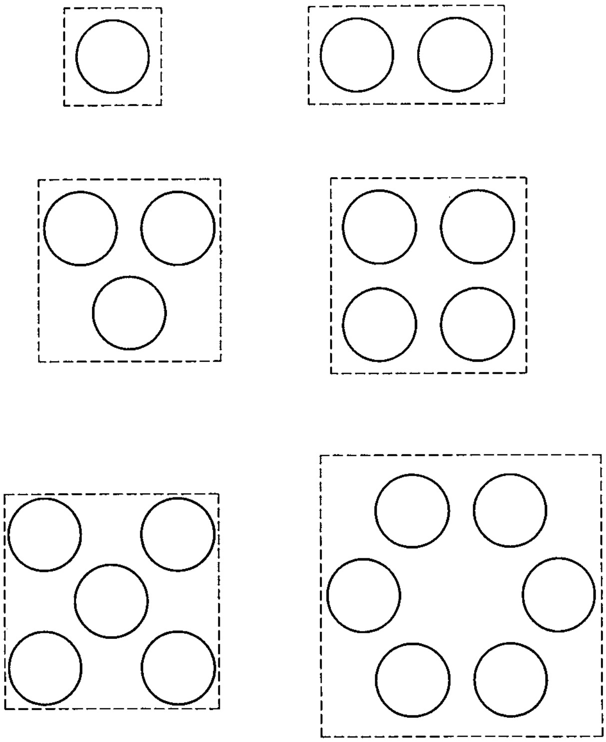


FIGURE 6A.164 Some common patterns of granular columns to support individual footings (from Geopier Foundation Company 1998).

Conventional stone columns are installed using the same type of horizontally vibrating probe used in vibro-compaction (see Section 6A.4.2). Either replacement (wet) or displacement (dry) techniques can be used. In the *vibro-replacement* method (Fig. 6A.165a), a hole is created in the ground to the desired depth by water jetting from the vibratory probe (ASCE 1987). The uncased hole is flushed out and stone is added in increments through the annular space between the probe and the enlarged hole. The stone is compacted in about 12 to 48 in (0.3 to 1.2 m) thick lifts by a combination of vibration from the probe and ramming the probe into the stone. During this process, soft matrix soils may collapse into the hole. If so, the continuing water upflow carries the collapsed material to the ground surface, allowing the stone to expand farther outward until equilibrium is reached. The diameter of the column varies with depth, generally being larger at the top, bottom, and at softer soil layers. Vibro-replacement is best suited for sites with soils having undrained shear strengths in the range of about 300 to 1000 psf (15 to 50 kPa) and a high groundwater table. In the 1970s and early 1980s, vibro-replacement was the only method used to construct stone columns in the United States, although dry methods were used elsewhere (Barksdale and Bachus 1983, Goughnour 1997). However, since that time environmental constraints have complicated the disposal of the large amounts of silt and clay-laden effluents generated during the wet process. An additional problem is the ponding of water on the ground surface, which can disrupt work and slow production. In response to these constraints and problems, the use of dry techniques is now common in the United States.

In the vibro-displacement technique, the vibrating probe displaces the soil laterally as it is ad-

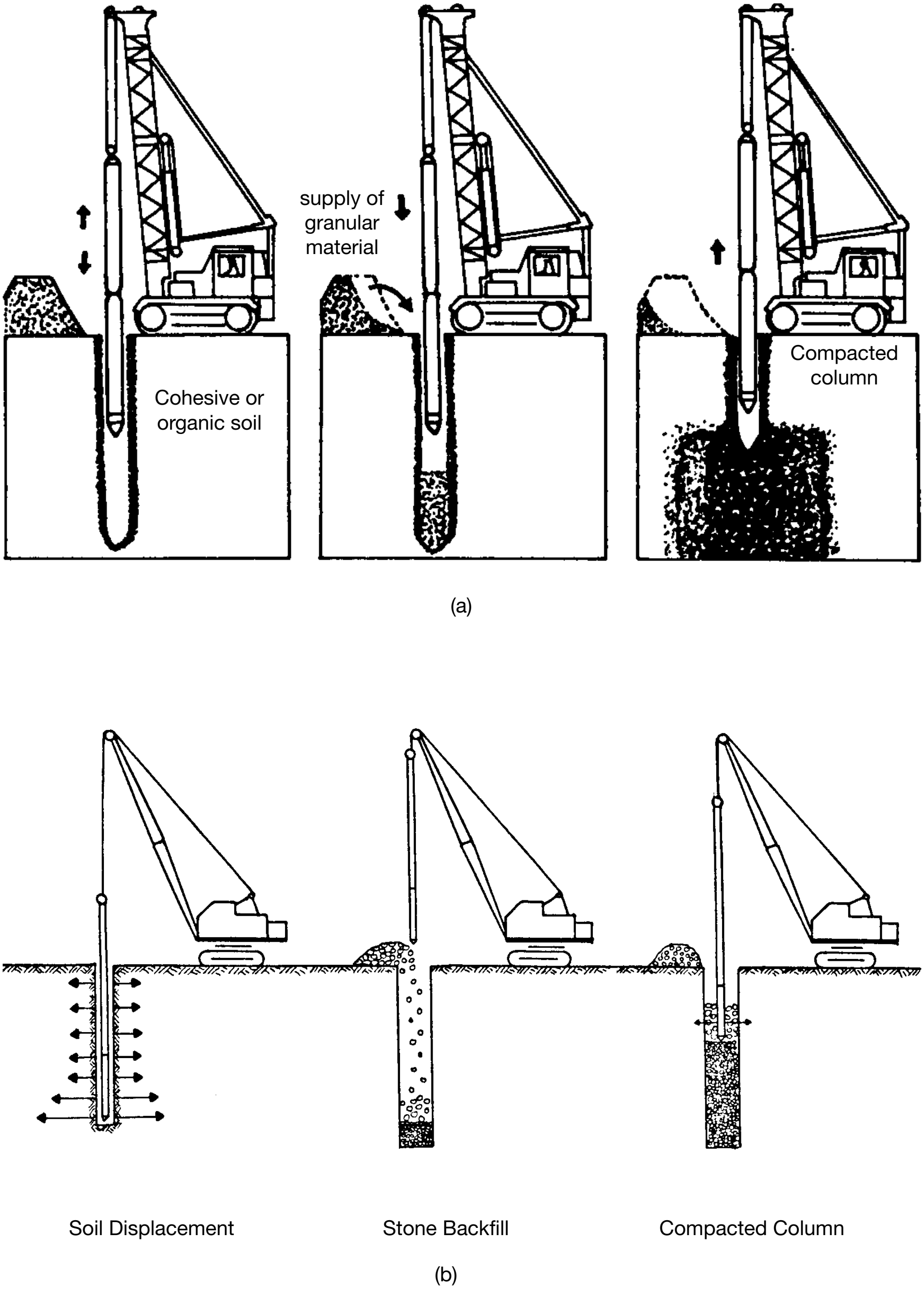


FIGURE 6A.165 Construction of a stone column by top-feed (a) vibro-replacement (from Baumann and Bauer 1974), and (b) vibro-displacement (from Barksdale and Dobson 1983).

vanced into the ground, usually with the aid of compressed air through the tip of the probe (ASCE 1987). In the top-feed method illustrated in Fig. 6A.165b, the probe is removed from the hole after reaching the desired depth, backfill is dropped in the annular space between the probe and enlarged hole, and the probe is lowered again to displace the stone laterally and downward. This process is repeated in lifts to create the compacted stone column. This top-feed dry method is best suited for sites with a deep groundwater table and firm soils with undrained shear strengths from about 600 to 1200 psf (30 to 60 kPa) and low sensitivity. Beginning about 1976, bottom-feed equipment and methods were developed to extend the use of vibro-displacement methods to soft and loose saturated soils (Jebe and Bartels 1983). In the bottom-feed methods, the probe remains in the hole while the stone is discharged through the probe. The columns created by the dry techniques are usually smaller in diameter than those created by the wet process because no matrix material is removed from the hole in the dry process.

The rotary method of installing stone columns was developed as an alternate technique for use in soft cohesive soils and loose silty or clayey sands (Goughnour 1997). This method was developed to reduce the problems associated with contamination of the stone by intermixing with matrix soil that occur in vibratory installation methods. The heart of this system is an impeller that consists of two symmetrically located logarithmic spiral sections (Fig. 6A.166). The impeller is driven by a centrally placed drive shaft and fits closely beneath the bottom of a feed pipe. Stone is fed down the annular space between the feed pipe and the shaft. During rotation of the impeller, stone is thrown radially outward while additional stone falls from the feed pipe into the pockets behind the logarithmic spiral sections. The main components of the machine used to install rotary stone columns (Fig. 6A.167a) are a carrier (crane) and mast (construction leads), a hopper and winch arrangement for stone delivery, a probe that includes a vibratory driver/extractor, a rotary hydraulic motor, an airlock and chute, and an impeller fitted at the bottom. The top portion of the probe is shown in Fig. 6A.167b.

The normal procedures used to build a rotary stone column are as follows (Goughnour 1997):

1. The feed pipe and the impeller are positioned over the location where the stone column is to be installed.
2. The feed pipe is lowered into the ground. During this process air pressure is applied to the in-

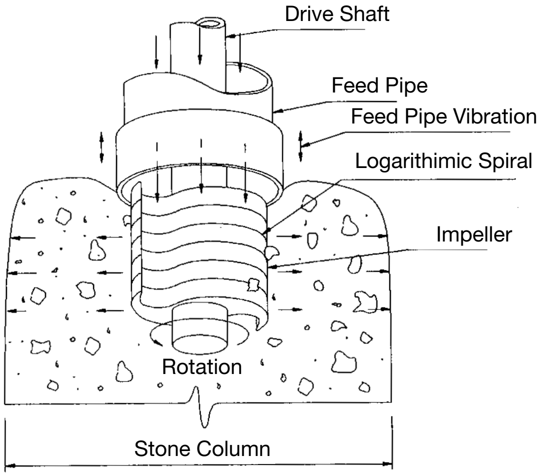
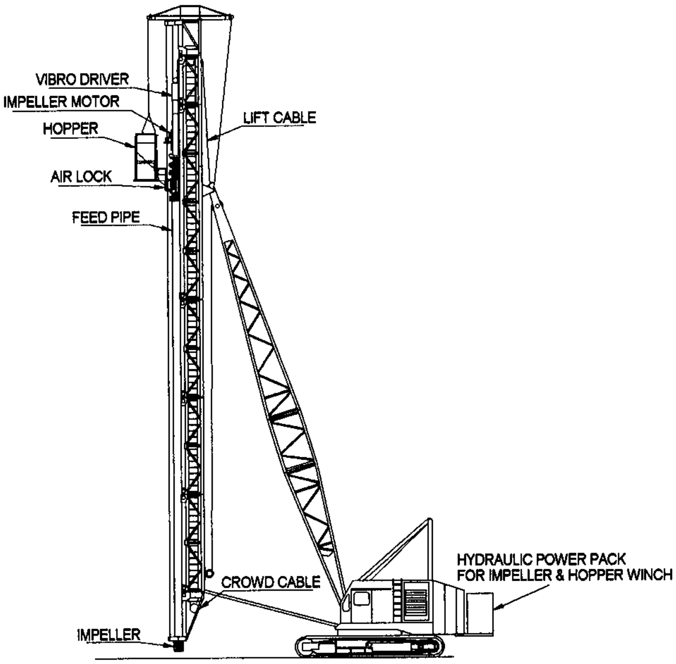
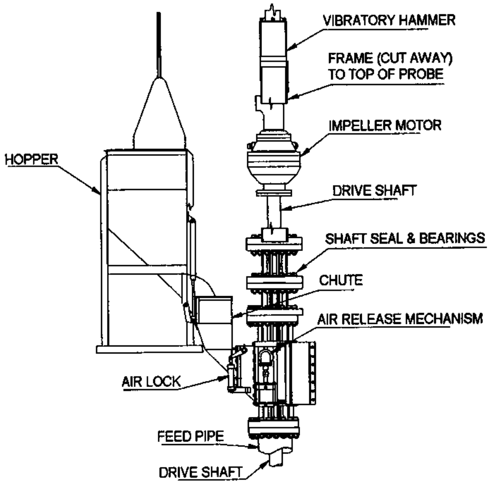


FIGURE 6A.166 Perspective view of impeller used in construction of rotary stone columns (from Goughnour 1997)



(a)



(b)

FIGURE 6A.167 Equipment used to construct rotary stone columns (from Goughnour 1997): (a) Overall view of rig, and (b) detail of top part of probe.

terior of the empty feed pipe, the impeller is rotating, and the vibratory driver/extractor is turned on.

3. When the desired depth is reached, the penetration, vibration, and impeller rotation are all stopped.
4. The loaded hopper is positioned for discharge into the airlock.
5. The air pressure from the feed pipe is released, the airlock is opened, and stone is fed from the hopper through the airlock into the feed pipe.
6. The airlock is closed and air pressure is re-established within the feed pipe.
7. With the impeller rotating and vibration applied as necessary, the feed pipe is raised in response to hydraulic pressure on the impeller motor. The pressure must be maintained as closely as possible to the target pressure, which is site-specific and depends on field conditions and desired column diameter. The target pressure may be to be varied if field conditions change on site.
8. When the feed pipe is empty, the hydraulic pressure on the impeller motor drops and does not rebuild. At this point the pipe is no longer lifted, and the vibration and impeller are stopped.
9. Steps 4 through 8 are repeated until the column is constructed to the desired elevation.

Stone columns have also been installed using a technique called *dynamic replacement*, which is a combination of stone columns and dynamic compaction, as shown in Fig. 6A.168 (Gambin 1984, Liausu 1984). In this method, stone is placed in a layer on the ground surface and compacted using a heavy tamper in the manner described for dynamic compaction (section 6A.4.1). Soil improvement occurs not only at the location of the stone columns, but also between column locations, owing to horizontal densification of the soil.

6A.5.1.2 Sand Columns

Sand columns—also known as sand compaction piles—have been used extensively in Japan and other parts of Asia to reinforce soft and loose soils. The methods of installation used to create sand columns can be classified into the following three primary types (Japanese Geotechnical Society 1998):

1. Vertical vibration and compaction with enlargement of the column diameter by repeatedly supplying sand and then pulling and repenetrating the casing pipe (Fig. 6A.169a).
2. Vertical vibration and compaction with enlargement of the column diameter by a hydraulic compaction device at the end of the casing pipe (Fig. 6A.169b).
3. Vertical vibration and compaction with enlargement of the column diameter by horizontal vibration of the end of the casing pipe (Fig. 6A.169c).

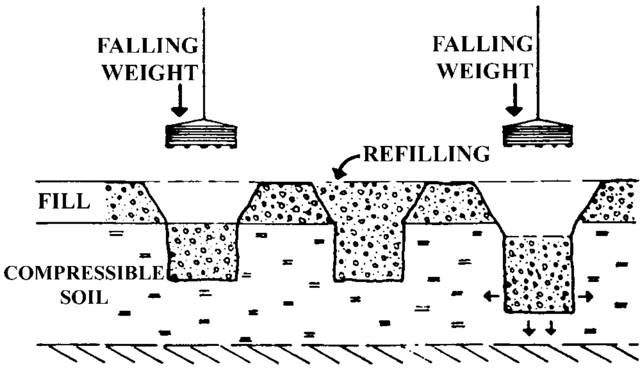


FIGURE 6A.168 Construction of stone columns by dynamic replacement (modified from Liausu 1984).

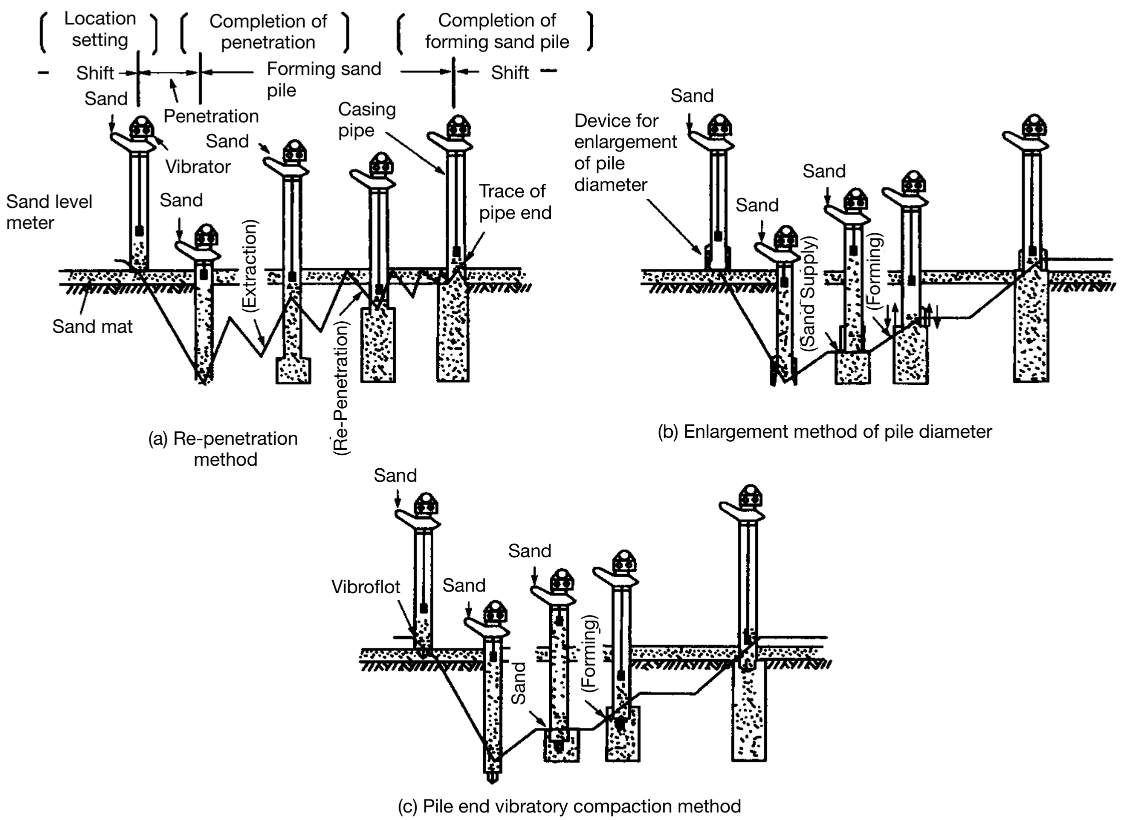


FIGURE 6A.169 Common methods used to install sand columns (from Japanese Geotechnical Society 1998).

6A.5.1.3 Geopiers

There are two primary types of geopiers—compressive and uplift. The major steps used to construct compressive geopiers are illustrated in Fig. 6A.170 and summarized as follows:

1. A cylindrical cavity is formed in the soil using an auger (Fig. 6A.170a). The diameter of the cavity is typically in the range of 24 to 36 in (0.61 to 0.91 m).
2. Aggregate is placed at the bottom of the hole and is compacted by repeated ramming using a specially designed tamper with a beveled head (Fig. 6A.170b,c). High-frequency, low-amplitude energy for this process is supplied by a skid loader, a backhoe, or an excavator (Fig. 6A.171). The "bulb" created from this process provides a firm foundation on which to construct the remainder of the geopier and is especially important in soft and loose soils.
3. The main body of the geopier is then constructed in a similar manner by placing loose aggregate in the hole and compacting it in 12 in (0.30 m) or thinner lifts to the desired height (Fig. 6A.170d). The height of a geopier is typically two to five times its diameter.

An uplift geopier is constructed in a similar manner with the following additional steps. After the bottom bulb is constructed (step 2), an uplift assembly consisting of a horizontal steel plate and attached vertical threaded steel bars (Fig. 6A.172) is set in hole and rests on the bottom bulb. A spac-

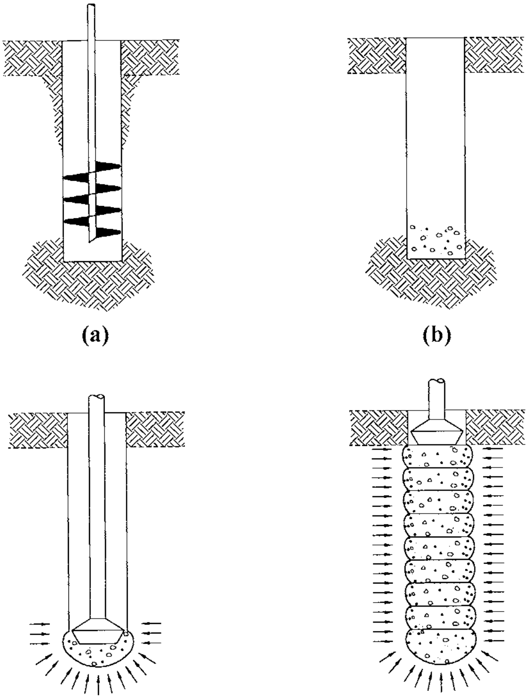


FIGURE 6A.170 Steps in construction of a compressive geopier.

er is used to hold the threaded bars apart during construction of the main body of the geopier in the same manner as described is step 3 above. In permanent applications, the uplift plate and threaded bars are galvanized to reduce long-term corrosion of the steel. The uplift bars or dowels spliced to the uplift bars extend upward into a reinforced concrete footing and are bonded to the concrete, forming an integral foundation system with the footing. Uplift geopiers also provide substantial resistance to compressive and lateral forces and displacements.

Well-graded gravelly sand (normally highway base course material) is typically used as the geopier aggregate above the groundwater table, with open-graded gravel normally used below the groundwater table. Owing to the confinement provided by the adjacent matrix soil and the high energy used to compact the geopiers, high densities are achieved within the compacted geopier aggregate (typically more than 100% of modified Proctor maximum dry density). In addition, the adjacent matrix soils are substantially prestressed and prestrained. Measurements in matrix soils adjacent to geopiers have shown that installation of a geopier can increase the horizontal stresses as far away as 10 ft (3 m) or more, and that the horizontal stresses immediately adjacent to the geopiers can reach the limiting passive condition.

Linear geopiers can also be constructed by first excavating a trench using a backhoe or excavator. Aggregate is then compacted to fill the trench in thin lifts using the same procedures described above for columnar geopiers. Linear geopiers are used to support walls and long, thin rectangular foundations and typically have nominal widths of 18 to 30 in (0.46 to 0.76 m).

FIGURE 6A.171 Installation of a geopier with an excavator supplying the energy.

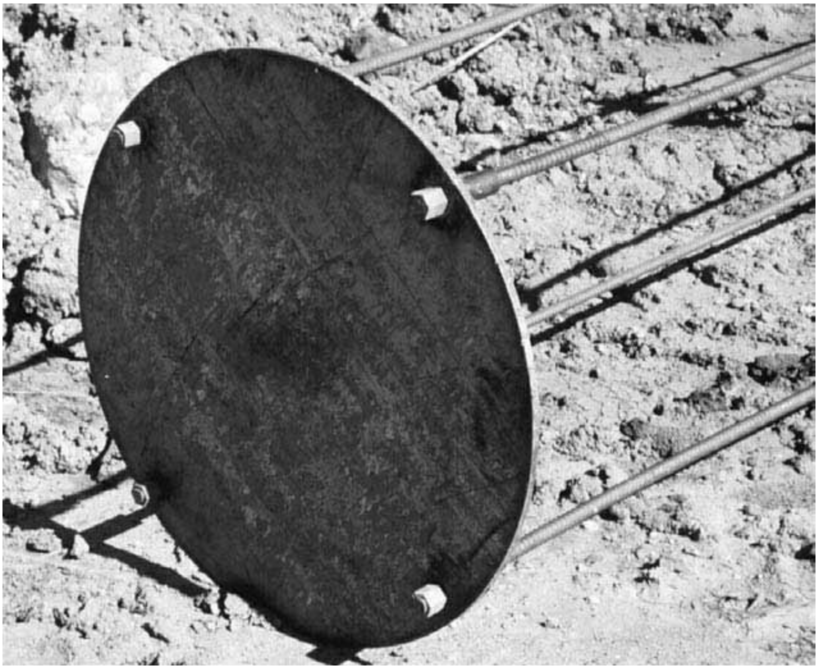


FIGURE 6A.172 Anchor plate with threaded steel bars used in uplift geopiers.

6A.5.1.4 Sand and Gravel Drains

The main purpose of sand and gravel drains is to provide rapid drainage of a saturated soil during precompression (section 6A.6), earthquakes, or other phenomena in which significant excess pore water pressures may develop. Sand and gravel drains are typically 8 to 20 in (200 to 500 mm) in diameter and are spaced anywhere from 5 to 20 ft (1.5 to 6.0 m). Since drainage is the primary desired engineering characteristic, compaction of the sand or gravel is not a priority; on the contrary, excessive compaction may reduce the permeability and hence reduce drainage capability of the drain. Therefore, only sufficient compaction necessary to ensure continuity of the drain is typically undertaken.

Many methods of installation are used to construct sand and gravel drains, which are generally classified into two groups (displacement and nondisplacement). The equipment used to install these types of drains include closed-ended mandrel, screw-type auger, continuous flight hollow stem auger, internal jetting, rotary jet, and Dutch jet-bailer. Additional details on the installation can be found in Ladd (1986).

6A.5.2 Engineering Characteristics, Response, and Behavior

6A.5.2.1 Stress Concentration

When compressive loads are applied to a bearing soil reinforced with granular or chemically stabilized columns, the vertical stress induced on top of the substantially stiffer columns is much greater than on the more compressible matrix soil at the same level. This concept is illustrated using a spring analogy in Fig. 6A.173 for a centric vertical load applied to a rigid foundation. In this analogy, each spring represents the force generated over the same amount of contact area. It is well known from basic physical laws that the force induced in a linearly elastic spring is equal in magnitude to the spring constant times the amount of deflection of the spring and acts in the direction opposite to the direction of the spring deflection ($F = -K\delta$). For the conditions shown in Fig. 6A.173, the footing will settle the same amount at all points. Hence the force induced in the stiff spring representing a granular column will be substantially greater than the forces generated in the springs representing the matrix soil. The sum of these spring forces must equal the applied load P. If these forces are divided by the same contact area, it is concluded that the stresses induced on the columns

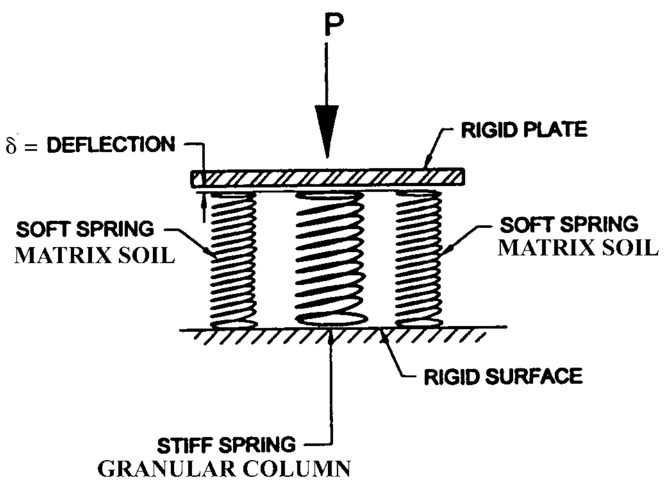


FIGURE 6A.173 Spring analogy illustrating stress concentration on granular columns (modified from Geopier Foundation Company 1998).

are greater than those induced on the matrix soil. Stress concentration also occurs beneath flexible foundations but to a lesser degree. Concentration of stresses on the stiffer columns is a key factor in controlling foundation settlement, increasing lateral resistance of footings, and stabilizing slopes with columnar reinforcement.

The concepts of unit cell, area replacement ratio, and stress concentration are illustrated in Fig. 6A.174 for columnar reinforcement arranged in a large square array at a center-to-center spacing of s. The unit cell is comprised of a single column and the corresponding tributary matrix soil. The boundaries of the unit cells are shown with dashed lines in Fig. 6A.174a. The area of a column is designated A_c, the area of the matrix soil within the unit cell A_m, and the total area of the unit cell A. The area replacement ratio (R_a) and the stress concentration ratio (R_s) are given by the following equations:

$$R_a = \frac{A_c}{A} \tag{6A.74}$$

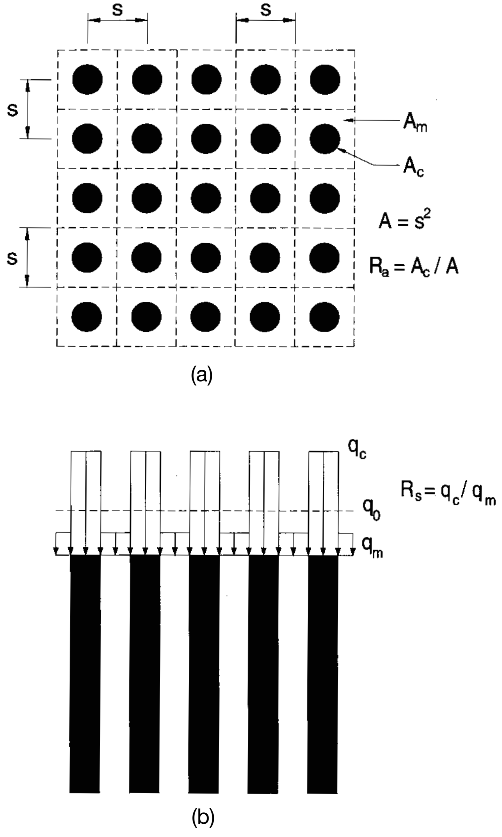


FIGURE 6A.174 Illustration of (a) unit cell and area replacement ratio (horizontal section), and (b) stress concentration and stress concentration ratio (vertical section).

$$R_s = \frac{q_c}{q_m} \tag{6A.75}$$

For an individual footing, A_c is the area of all the columns supporting the footing and A is the total area of the footing. If the footing is rigid, as is the case for most individual footings composed of reinforced concrete, $R_s = k_{sc}/k_{sm}$, where k_s is vertical subgrade modulus (Lawton et al. 1994). Since k_s varies as a function of the width of the loaded area (see Eqs. 6A.54–6A.57), the values of k_{sc} and k_{sm} used must correspond to the appropriate loaded areas of the columns and the matrix soil. k_{sc} can be obtained by performing a plate load test on top of a test or production column, with the diameter of the plate the same as the nominal diameter of the column. k_{sm} can be estimated by performing a similar plate bearing test on the matrix soil at the elevation of the top of the columns and using established scaling laws to calculate k_{sm} for the width and shape of the actual footing. k_{sm} can also be estimated from the results of in situ or laboratory tests on the matrix soil (see Bowles 1975, pp. 516–518).

The following equations for the stresses induced on the top of columns (q_c) and the matrix soil (q_m) as a function of the average applied stress (q_0) can be obtained by summing forces in the vertical direction and satisfying static equilibrium (Aboshi et al. 1979):

$$q_c = q_0 \frac{R_s}{R_a(R_s - 1) + 1} = q_0 \cdot \mu_c \tag{6A.76}$$

$$q_m = q_0 \frac{1}{R_a(R_s - 1) + 1} = q_0 \cdot \mu_m \tag{6A.77}$$

Both q_c and q_m depend on R_s, which must be estimated. The following theoretical equations from Aboshi et al. (1979) can be used to estimate the stress concentration ratio at yield (R_{sy}).

For friction-only columnar and matrix materials (ϕ', $c' = 0$):

$$\tan^2(45° + \phi_c'/2) \le R_{sy} \le \tan^2(45° + \phi_c'/2) \cdot \tan^2(45° + \phi_c'/2) \tag{6A.78}$$

For friction-only columnar material (ϕ_c', $c_c' = 0$) and a saturated cohesive matrix soil in the unconsolidated-undrained condition ($\phi_m = 0$, $c_m = s_{um}$):

$$R_{sy} \le \tan^2(45° + \phi_c'/2) \cdot (1 + 2s_{um}/q_m) \tag{6A.79}$$

In a design situation where settlement is to be estimated, these equations are of limited value because the actual state of stress should be well below the level at which failure occurs.

Typical values of R_s vary depending on the type of granular columns, applied stress level, duration of the load, stiffness of the matrix soil, and flexibility of the foundation applying the load to the columns. Measured values of R_s for stone columns and sand columns generally range from 2.5 to 5.0 for typical values of applied stress. In design, R_s is usually conservatively assumed to be about 2 or 3. There are less data available for measured values of R_s for geopiers. In recent tests with rigid foundations, measured stress concentration ratios varied from about 10 to 40 at typical applied stress levels (Lawton 1999). R_s for the design of geopier foundations is usually conservatively estimated to be about 10 to 15. It appears that R_s increases with increasing applied load up to a critical value of applied stress where it reaches a maximum (Han and Ye 1991). At stresses higher than the critical value, R_s decreases with increasing load. Furthermore, R_s usually increases with time at constant applied stress, probably owing to greater secondary compression within the matrix soil than the granular columns.

6A.5.2.2 Settlement

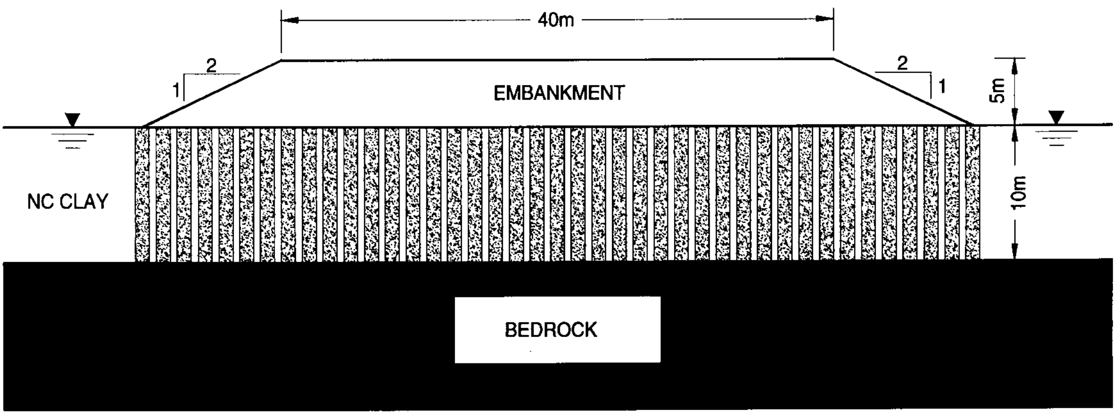
The inclusion of columnar reinforcement reduces the magnitude of settlement and generally increases the rate of settlement compared to the unreinforced soil. Numerous methods varying in complexity from simple approximations to sophisticated numerical analyses have been used to estimate the magnitude of settlement for foundations bearing on columnar-reinforced soil. Some of these methods are used exclusively for certain types of columnar reinforcement. Only a general overview of some of the most commonly used methods will be described here. Discussions of additional methods can be found, for example, in ASCE (1987) and Barksdale and Bachus (1983).

One of the simplest methods for estimating the settlement of structures founded on column-reinforced bearing soil is the *equilibrium method.* Although this method was originally developed to estimate primary consolidation settlement of saturated clays reinforced with sand columns (Aboshi et al. 1979), it can be applied to any type of matrix soil and drainage conditions. However, the equilibrium method is valid only where the columnar reinforcement extends throughout the entire depth of the compressible material or throughout the depth where most of the strain occurs (approximately two times the width of a circular or a square foundation and about four times the width of a strip foundation). The steps in the equilibrium method are summarized as follows:

1. Estimate the stress induced on the matrix soil at the bearing level (q_m) using Eq. 6A.77.
2. Calculate the settlement of the structure as if there is no columnar reinforcement and the average stress at the bearing level is q_m rather than q_0. Use the actual dimensions and shape of the foundation.

The following example is given to illustrate the method.

Example Problem 6A.2 A proposed highway embankment will be 40 m (131 ft) wide at its crest and 5 m (33 ft) tall (H_e) with 2H:1V side slopes and an average unit weight (γ_e) of 20 kN/m^3 (127 pcf). The embankment will be constructed on the surface of a normally consolidated clay stratum that is 10 m (33 ft) thick with an average buoyant unit weight (γ') of 7.0 kN/m^3 (45 pcf), an existing average void ratio of 1.30, and a virgin compression index (C_c) of 0.30. The clay stratum is underlain by hard and impervious bedrock. The properties of the clay stratum will be assumed constant throughout the clay layer to simplify the calculations. Reinforcing columns 1.0 m (3.3 ft) in diameter will be arranged in a square array at a spacing (s) of 1.5 m (4.9 ft) and will extend the entire height of the clay stratum. The stress concentration ratio (R_s) is estimated to be 5. Estimate the ultimate primary consolidation settlement (S_c) of this embankment along its centerline both without and with the columnar reinforcement.



Solution: S_c without columnar reinforcement S_c for the clay layer without reinforcement will be calculated by subdividing the clay stratum into ten 1.0-m (3.3-ft) thick sublayers and calculating and summing S_c for each sublayer. In equation form:

$$S_c = \sum_{i=1}^{i=n} S_{ci}$$

where n is the total number of sublayers (ten in this case) and i identifies individual sublayers. The solution is shown in tabular form below. It should be noted that in real practice, a greater number of sublayers should be used, which can be easily accomplished using a spreadsheet program. Only ten sublayers are used here because of space limitations. For simplicity, buoyancy effects produced by submergence of the lower portion of the embankment as settlement occurs will be ignored (these effects are usually minor).

For a normally consolidated clay:

$$S_{ci} = \frac{C_{ci}}{1 + e_{0i}} \cdot H_{0i} \cdot \log \frac{\sigma'_{vf}}{\sigma'_{v0}}$$

where H_{0i} = initial height of sublayer i
σ'_{v0i} = initial effective vertical stress at the midheight of sublayer $i = \gamma' \cdot z_i$
σ'_{vf} = final effective vertical stress at the midheight of sublayer $i = \sigma'_{v0i} + \Delta\sigma_{vi}$
$\Delta\sigma_{vi}$ = total vertical stress induced by load from embankment
z_i = depth from top of the clay layer to the midheight of sublayer i

$\Delta\sigma_v$ will be calculated using the following equation (Osterberg 1957):

$$\Delta\sigma_v = \frac{2q_0}{\pi}\left[\frac{m+n}{m} \cdot \tan^{-1}(m+n) - \frac{n}{m}\tan^{-1} n\right]$$

where $m = a/z$
$n = b/z$
a = horizontal width of one slope = 10 m (33 ft)
b = half the width of the embankment crest = 20 m (66 ft)
q_0 = applied stress at the bottom of the embankment = $\gamma_e \cdot H_e$ = (20)(5) = 100 kPa (2.09 ksf)

i	H_{0i} (m)	z_i (m)	σ'_{v0i} (kPa)	m	n	$\Delta\sigma_{vi}$ (kPa)	σ'_{vfi} (kPa)	S_{ci} (m)
1	1	0.5	3.5	20.00	40.00	100.00	103.50	0.1919
2	1	1.5	10.5	6.67	13.33	99.99	110.49	0.1333
3	1	2.5	17.5	4.00	8.00	99.95	117.45	0.1078
4	1	3.5	24.5	2.86	5.71	99.88	124.38	0.0920
5	1	4.5	31.5	2.22	4.44	99.74	131.24	0.0808
6	1	5.5	38.5	1.82	3.64	99.54	138.04	0.0723
7	1	6.5	45.5	1.54	3.08	99.26	144.76	0.0656
8	1	7.5	52.5	1.33	2.67	98.89	151.39	0.0600
9	1	8.5	59.5	1.18	2.35	98.44	157.94	0.0553
10	1	9.5	66.5	1.05	2.11	97.89	164.39	0.0513
								ΣS_{ci} = 0.9103

Therefore, S_c = 0.91 m (3.0 ft) without reinforcement.

Solution: S_c with columnar reinforcement S_c for the clay layer with reinforcement will be calculated in the same manner except that q_m will be used in place of q_0. First, the area replacement ratio (R_a) will be calculated using the unit cell concept because the columnar reinforcement is of large areal extent.

$$A = s^2 = (1.5)^2 = 2.25 \text{ m}^2 \text{ (24.2 ft}^2\text{)}$$

$$A_c = 0.25\pi(1.0)^2 = 0.7854 \text{ m}^2 \text{ (8.45 ft}^2\text{)}$$

$$R_a = A_c/A = 0.7854/2.25 = 0.3491$$

Now calculate q_m using Eq. 6A.77:

$$q_m = 100 \cdot \frac{1}{0.3491(5-1)+1} = 100(0.4173) = 41.73 \text{ kPa (872 psf)}$$

The solution is set up in the same tabular form as before.

i	H_{0i} (m)	z_i (m)	σ'_{v0i} (kPa)	m	n	$\Delta\sigma_{vi}$ (kPa)	σ'_{vfi} (kPa)	S_{ci} (m)
1	1	0.5	3.5	20.00	40.00	41.73	45.23	0.1450
2	1	1.5	10.5	6.67	13.33	41.73	52.23	0.0909
3	1	2.5	17.5	4.00	8.00	41.71	59.21	0.0690
4	1	3.5	24.5	2.86	5.71	41.68	66.18	0.0563
5	1	4.5	31.5	2.22	4.44	41.62	73.12	0.0477
6	1	5.5	38.5	1.82	3.64	41.54	80.04	0.0415
7	1	6.5	45.5	1.54	3.08	41.42	86.92	0.0367
8	1	7.5	52.5	1.33	2.67	41.27	93.77	0.0329
9	1	8.5	59.5	1.18	2.35	41.08	100.58	0.0297
10	1	9.5	66.5	1.05	2.11	40.85	107.35	0.0271
								$\Sigma S_{ci} = 0.5767$

Therefore, using columnar reinforcement would result in an estimated 58% reduction in stress induced in the matrix soil. The ultimate primary consolidation settlement is expected to be reduced from 0.91 m (3.0 ft) to 0.58 m (1.9 ft), a 36% reduction. Note that the reduction in S_c is less than the reduction in stress because S_c is proportional to the reduction in the logarithm of induced stress rather than the reduction in induced stress. For very small values of induced stress and very high values of initial effective stress (deep compressible layers), the percent reduction in settlement approaches the average percent reduction in induced stress (Barksdale and Bachus 1983). Note also that the magnitude of settlement for thick embankments constructed on soft clays is significant even for substantial reinforcement. Generally the columnar reinforcement will also substantially increase the rate at which the settlement occurs. This increase in the rate of settlement is especially important in embankment and fill construction wherein most of the settlement must be allowed to occur before any structures (pavement systems, bridge abutments, buildings, etc.) are founded on or within the embankment or fill.

Priebe (1988) has developed design charts for estimating the reduction in settlement for long granular columns. Charts were provided for both one-dimensional and three-dimensional settlement (Figs. 6A.175 to 177). The charts for one-dimensional settlement (Figs. 6A.175 and 6A.176) are appropriate when the width of the loaded area (B) is very large in comparison to the height of the com-

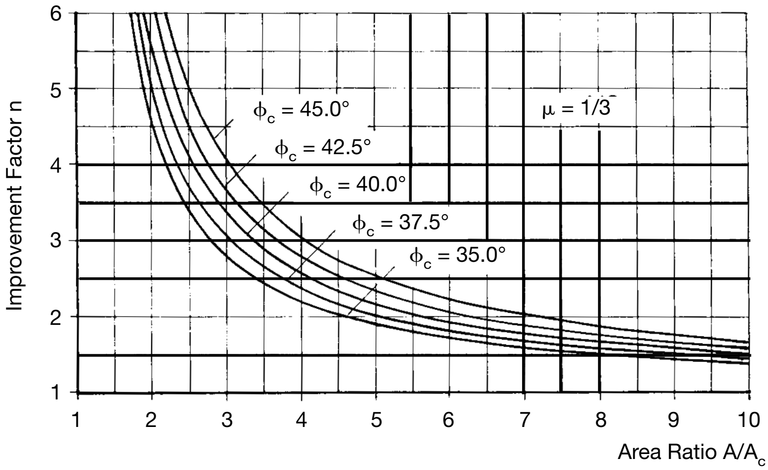


FIGURE 6A.175 Priebe's chart for reduction in settlement for an infinite array of infinitely stiff granular columns (from Moseley and Priebe 1993).

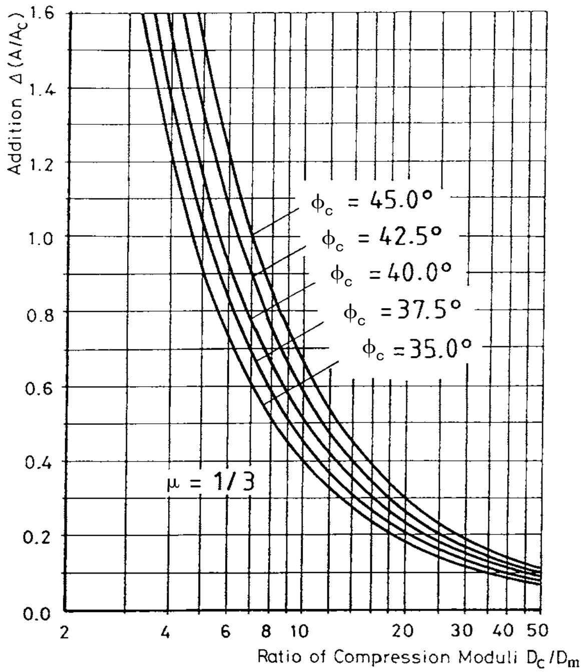


FIGURE 6A.176 Priebe's chart for additional area ratio to account for compressibility of the granular columns (from Moseley and Priebe 1993).

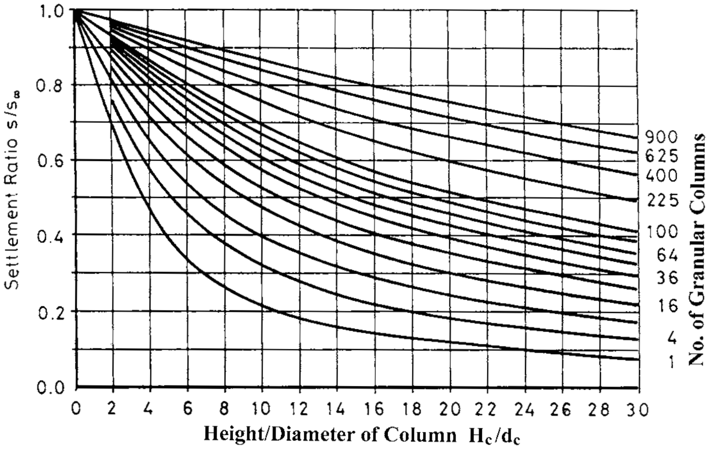


FIGURE 6A.177 Priebe's chart to estimate settlement of an isolated footing supported by granular columns (modified from Moseley and Priebe 1993).

pressible layer, and the columns extend throughout the height of the compressible layer. Fig. 6A.177 applies to isolated footings supported by granular columns.

One-Dimensional Settlement. One-dimensional settlement occurs when the width of the loaded area is infinitely wide. One-dimensional settlement is also approximated when the width of the loaded area is very large in comparison to the height of the compressible layer. The results in Fig. 6A.175 are based on the assumption that the granular columns are incompressible, which is obviously not the case. Fig. 6A.176 is used to correct for the compressibility of the granular columns. The procedure for estimating settlement for this case is as follows:

1. Determine the ratio of one-dimensional compression moduli for the columnar material to the matrix soil (D_c/D_m). This value can be estimated (it is approximately equal to the stress concentration ratio, R_s), or one-dimensional compression tests can be conducted on specimens of the two materials.
2. From Fig. 6A.176 with D_c/D_m and ϕ_c, determine the additional area ratio $\Delta(A/A_c)$.
3. Add the value of $\Delta(A/A_c)$ calculated in step 2 to the true area ratio (A/A_c).
4. With the combined value of A/A_c calculated in step 3 and ϕ_c, determine the settlement improvement factor (n) using Fig. 6A.175. n is defined as the ratio of the settlement without granular columns to the settlement with granular columns.
5. Divide the value of settlement for no granular columns (calculated using any appropriate method) by the value of n found in step 4. This value is the estimated settlement with granular columns.

Three-Dimensional Settlement. Three-dimensional settlement occurs when the width of the loaded area is small in comparison to the height of the compressible layers. This commonly occurs when individual footings are supported by granular columns. The procedure for estimating settlement for this case is as follows:

1. Determine the number of granular columns in the group and the height-to-diameter ratio of the columns (H_c/d_c). With these values, find the settlement ratio S/S_∞ using Fig. 6A.177. S/S_∞ is de-

fined as the settlement of the finite group (3-D settlement) divided by the settlement for an infinitely wide group (1-D settlement) under the same conditions.

2. Estimate S_∞ using the method described above for one-dimensional settlement.
3. Multiply S/S_∞ from step 1 by S_∞ from step 2 to obtain an estimate of the settlement for the finite group.

6A.5.2.3 Ultimate Bearing Capacity

In most instances, the bearing soil for a foundation is supported by a group of reinforcing columns. The mechanisms by which failure occurs for a group of columns is quite complex and involves several types of interaction. Sometimes a foundation is supported by a single column, for which the mechanisms of failure are more straightforward. Thus, ultimate bearing capacity for single columns will be discussed first, which will then provide the basis to understand the more complex situation for groups of columns.

Single Columns. There are three possible mechanisms of failure for a single reinforcing column in a homogeneous soil mass, as illustrated in Fig. 6A.178. The type of failure that would occur in any situation depends on characteristics of the columnar material and the matrix soil, and the area over which the load is applied (Fig. 6A.179).

Bulging: Failure of a single granular column in weak matrix soil such as a soft clay will occur by bulging. Granular columns have little or no internal cohesion and therefore depend on lateral resistance provided by the matrix soil. As load is applied to the top of the column, the column will tend to push outward within the upper portion of the column. If the matrix soil is weak, bulging will occur within a height of about two to three times the diameter of the column if the matrix soil is relatively homogeneous within this zone. If the upper portion of the matrix soil consists of layers of relatively strong and weak soils, bulging may occur only in the weaker layers. If the load is applied over an area greater than the area of the column (Fig. 6A.179a), the ultimate bearing capacity is increased in two ways: (a) Some of the load is carried by the matrix soil, lessening the load carried by the column; and (b) the stress carried by the matrix soil increases the confinement on the upper portion of the column.

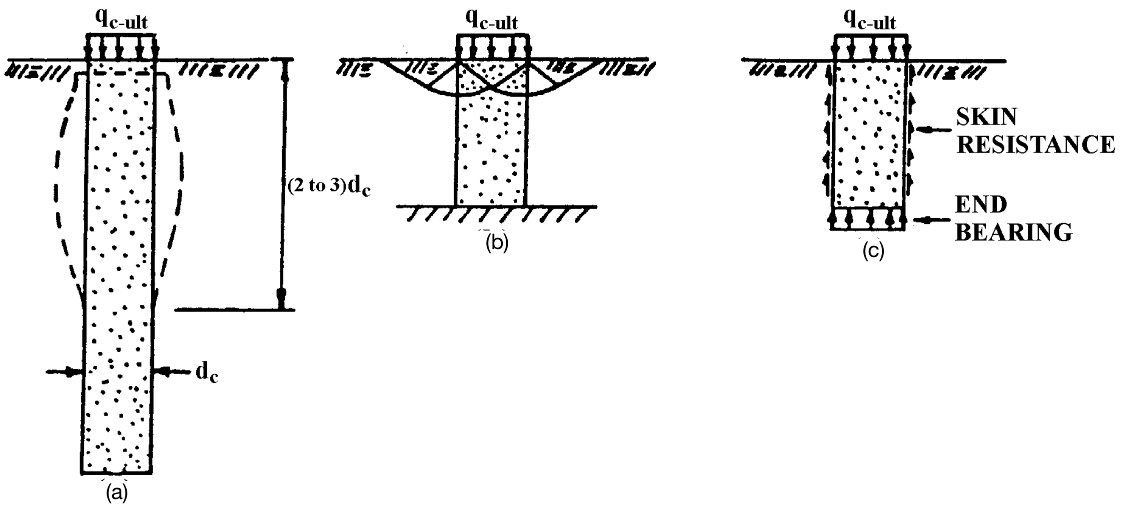


FIGURE 6A.178 Mechanisms of failure for single reinforcing columns: (a) Bulging, (b) general or local shear, and (c) punching.

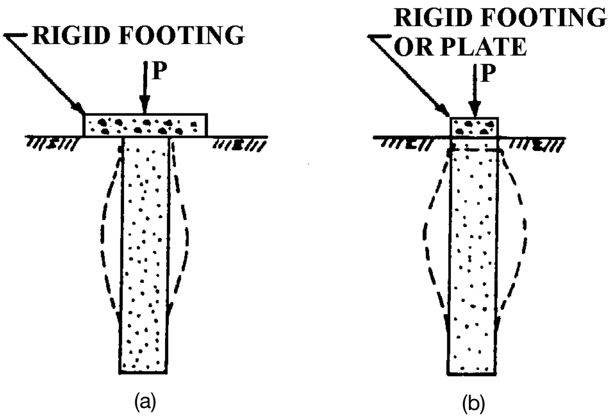


FIGURE 6A.179 Types of loading for single reinforcing column: (a) Footing larger than column, (b) footing same size as column.

General or Local Shear: It is unlikely that a single column will fail by general or local shear failure, but it may occur under the following conditions: (a) a very short column [H_c < (2 to 3)d_c] bearing on a rigid base, or (b) the columnar material is not significantly stronger than the matrix soil. Neither situation occurs very often in practice. These types of failure are similar to those that occur for shallow foundations bearing on homogeneous unreinforced soils.

Punching: Punching or pile-like failure will occur in columns with substantial internal cohesion, such as lime-cement, jet grouted, vibratory concrete, and rammed cement columns. This mechanism of failure may also predominate in very short granular columns [H_c < (2 to 3)d_c] floating in the matrix soil (not bearing on a rigid base). Resistance to the applied load develops as skin resistance (shearing stresses) along the interface of the column and matrix soil, end bearing (normal stresses) at the bottom of the column, or a combination of both.

The best method to determine the ultimate bearing capacity of a single column is to load a prototype to failure. This is generally accomplished by conducting a plate load test (see Section 6A3.5.2 for details). However, this is an expensive and time-consuming process that typically requires generating about 100 to 500 kips (450 to 2200 kN) of reactive force. Thus, ultimate bearing capacity is usually estimated.

The following empirical formula can be used to estimate the ultimate bearing capacity of a single granular column within a matrix of soft clay (bulging failure):

$$q_{c\text{-ult}} = N_{sc} \cdot s_{um} \tag{6A.80}$$

where N_{sc} = bearing capacity factor for a single column
s_{um} = undrained shear strength of the clay matrix

For vibro-replacement stone columns, N_{sc} has been found to range from about 18 to 25 (Barksdale and Bachus 1983, Mitchell 1981).

Many theories and analytical methods have been developed for the ultimate compressive capacity of a single granular column in saturated clay under undrained conditions. Summaries of many of these methods can be found in ASCE (1987), Barksdale and Bachus (1983), and Brauns (1978). Two theories will be described here for illustrative purposes.

Bell's (1915) method is the simplest and most conservative and is illustrated in Fig. 6A.180. In

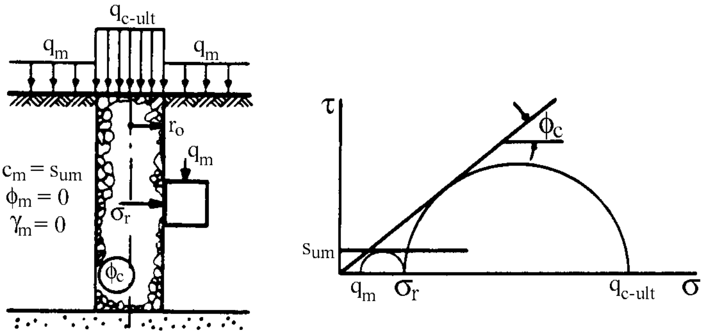


FIGURE 6A.180 Bell's (1915) Method for the ultimate bearing capacity of a single granular column within undrained clay matrix (modified from Brauns 1978).

this method, the weight of the matrix soil and the shear stresses that develop along the column–matrix interface are ignored, as well as the distribution of stress within the columnar and matrix soils. Hence, the principal stresses in both the column and matrix soil adjacent to the interface are assumed to act in the vertical and horizontal directions. In the column adjacent to the interface, the major principal stress is in the vertical direction ($q_{c\text{-ult}}$) owing to the high stress concentration on the column. The minor principal stress therefore acts in the radial direction and is designated σ_r. For a granular column, the cohesion intercept (c_c) is zero, and the following equation can be written based on the geometry of the Mohr's circle at failure in the column:

$$q_{c\text{-ult}} = \sigma_r \cdot \tan^2(45° + \phi_c/2) \tag{6A.81}$$

The corresponding equation at failure in the clay adjacent to the interface is as follows:

$$\sigma_r = q_m + 2s_{um} \tag{6A.82}$$

Inserting Eq. 6A.82 into Eq. 6A.81 gives:

$$q_{c\text{-ult}} = (q_m + 2s_{um}) \cdot \tan^2(45° + \phi_c/2) \tag{6A.83}$$

Normalized values of ultimate bearing capacity ($N_{sc} = q_{c\text{-ult}}/s_{um}$) are plotted versus ϕ_c for Bell's method assuming $q_m = 0$ in Fig. 6A.181. Note that Bell's method is very conservative compared to the other methods shown and compared to the typical range of values of N_{sc} reported in the literature (see previous discussion).

Hughes and Withers (1974) considered bulging failure of a single column to be similar to the expansion of a pressuremeter probe against the sides of a borehole. In a pressuremeter test, the radial resistance of the soil reaches a limiting value (σ_{rl}) at which indefinite expansion occurs. If the soil is idealized as an elasto-plastic material, the limiting radial stress can be estimated from the following equation (Gibson and Anderson 1961):

$$\sigma_{rl} = \sigma_{r0} + s_{um}\left[1 + \ln\frac{E_m}{2s_{um}(1 + \nu_m)}\right] \tag{6A.84}$$

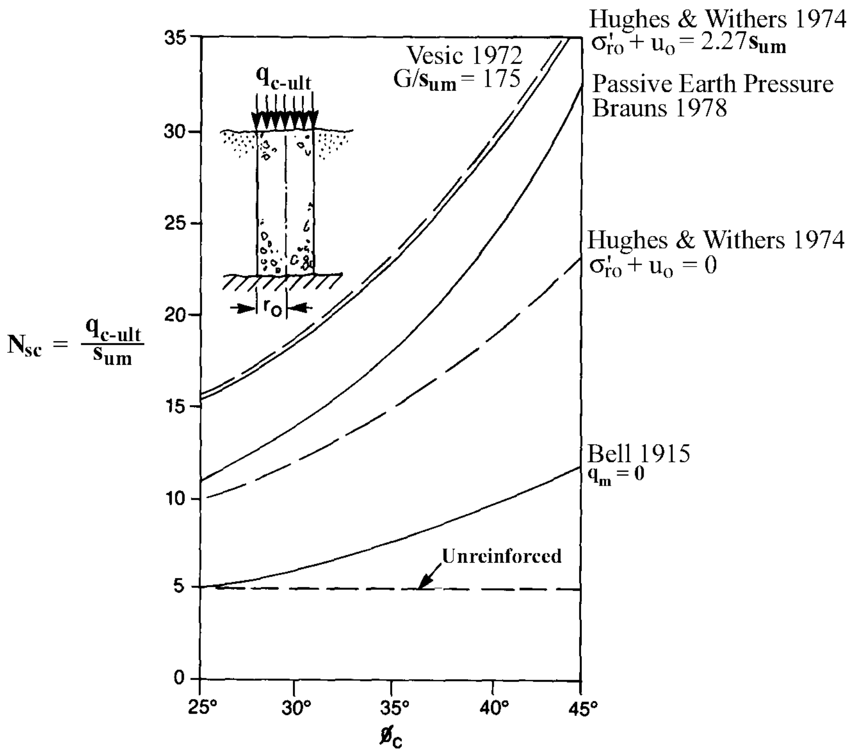


FIGURE 6A.181 Comparison of predicted values by various methods of ultimate bearing capacity for a single granular column within an undrained clay matrix (modified from Brauns 1978).

where ln is the natural logarithm, σ_{r0} is the initial radial stress in the matrix soil (prior to conducting the pressuremeter test), and s_{um}, E_m, and ν_m are the undrained shear strength, elastic modulus, and Poisson's ratio of the matrix soil, respectively. The following empirical equation for σ_{rl} determined from the results of many quick-expansion pressuremeter tests can also be used:

$$\sigma_{rl} = \sigma'_{r0} + 4s_{um} + u_0 \tag{6A.85}$$

where σ'_{r0} is the initial effective radial stress and u_0 is the initial excess pore water pressure. Because the granular columnar material acts as a drain, u_0 can be reasonably taken as zero with little or no error introduced. Of course, σ_{rl} can also be determined from pressuremeter tests conducted within the range of depths where bulging is expected occur (upper two to three diameters of the column).

Ignoring the shearing stresses that develop along the interface of the columnar and matrix soils and any stress carried by the matrix soil at the bearing level (q_m), the ultimate bearing capacity can be calculated as follows:

$$q_{c\text{-ult}} = \sigma_{rl} \cdot \tan^2(45° + \phi_c/2) \tag{6A.86}$$

Curves of $q_{c\text{-ult}}/s_{um}$ versus ϕ_c are shown in Fig. 6A.181 using Eqs. 6A.85 and 6A.86 and two values of $(\sigma'_{r0} + u_0)$: zero and $2.27s_{um}$.

Madhav and Vitkar's (1978) method can be used to estimate the ultimate bearing capacity for general shear failure within a long granular trench (rectangular prism) and an undrained clay matrix supporting a foundation. This method is described in Section 6A.2.1. So far as the author knows, there is no method available for general shear failure for a single reinforcing column. Established methods for estimating the ultimate bearing capacity of piles can be extended to single reinforcing columns that fail by punching. These methods can be found in any standard foundation engineering book (for example, see Section 4C) and will not be discussed here.

Groups of Columns. Hughes and Withers (1974) indicated that groups of granular columns within soft cohesive soils act independently when the center-to-center spacing of the columns is greater than about $2.5d_c$. This criterion also seems reasonable for other types of columns and matrix soils. For this case, the average ultimate stress that can be carried by the group (q_{ult}) is given by the following expression:

$$q_{\text{ult}} = \frac{q_{c\text{-ult}} \cdot A_c + q_m \cdot A_m}{A} = q_{c\text{-ult}} \cdot R_a + q_m(1 - R_a) \tag{6A.87}$$

It should be noted that many of the methods developed for single columns ignore the confining pressure provided by q_m that in a group of columns helps to resist bulging. This contribution may be significant in some instances and should not be arbitrarily ignored.

For larger spacings the columns act with the matrix soil as a composite material, and the group action of the reinforced zone must be considered. It is useful in some instances to consider this composite zone to have a single set of Mohr–Coulomb strength parameters calculated as follows:

$$\phi_{\text{comp}} = \tan^{-1}[\mu_c \cdot R_a \cdot \tan \phi_c + \mu_m(1 - R_a)\tan \phi_m] \tag{6A.88}$$

$$c_{\text{comp}} = c_c \cdot R_a + c_m(1 - R_a) \tag{6A.89}$$

Note that the effects of stress concentration are included in Eq. 6A.88 for ϕ_{comp}. If stress concentration does not occur, such as when columnar reinforcing is used to stabilize natural slopes (to be discussed subsequently), values of $\mu_c = \mu_m = 1$ must be used in Eq. 6A.88.

If the height of the columns is relatively short compared to the width of the foundation ($H_c \leq 1.5B$), q_{ult} for the group can be estimated using the composite strength parameters and the method developed for overexcavation/replacement (Section 6A.2.1). For taller columns ($H_c > 1.5B$), the failure mechanism suggested by Barksdale and Bachus (1983), as illustrated in Fig. 6A.182, may be appropriate. In this method, the major principal plane at the bearing level is assumed to be horizontal, with the result that failure is initiated at one corner of the foundation and the failure surface extends underneath the foundation at an angle of $\beta = 45° + \phi_{\text{comp}}/2$ relative to horizontal to a point directly beneath the opposite corner of the foundation. This sliding wedge is resisted by passive lateral resistance within the adjacent matrix soil. This passive resistance is assumed to be the major principal stress in the matrix soil (σ_{1m}) and the minor principal stress in the composite reinforcing zone (σ_{3c}), and q_{ult} is the major principal stress for the composite reinforced zone. If the weight of the sliding wedge is ignored, which is conservative, the corresponding equations are as follows:

$$\sigma_{1m} = \sigma_{3c} = (\overline{q} + 0.5\gamma_m B \tan \beta)\tan^2(45° + \phi_m/2) + 2c_m \tan(45° + \phi_m/2) \tag{6A.90}$$

$$q_{\text{ult}} = \sigma_{3c} \tan^2 \beta + 2c_{\text{comp}} \tan \beta \tag{6A.91}$$

where $\overline{q}$ is the effective vertical surcharge pressure at the bearing level. The term $(\overline{q} + 0.5\gamma_m B \tan \beta)$ corresponds to the average vertical normal stress acting along the vertical face of the sliding wedge

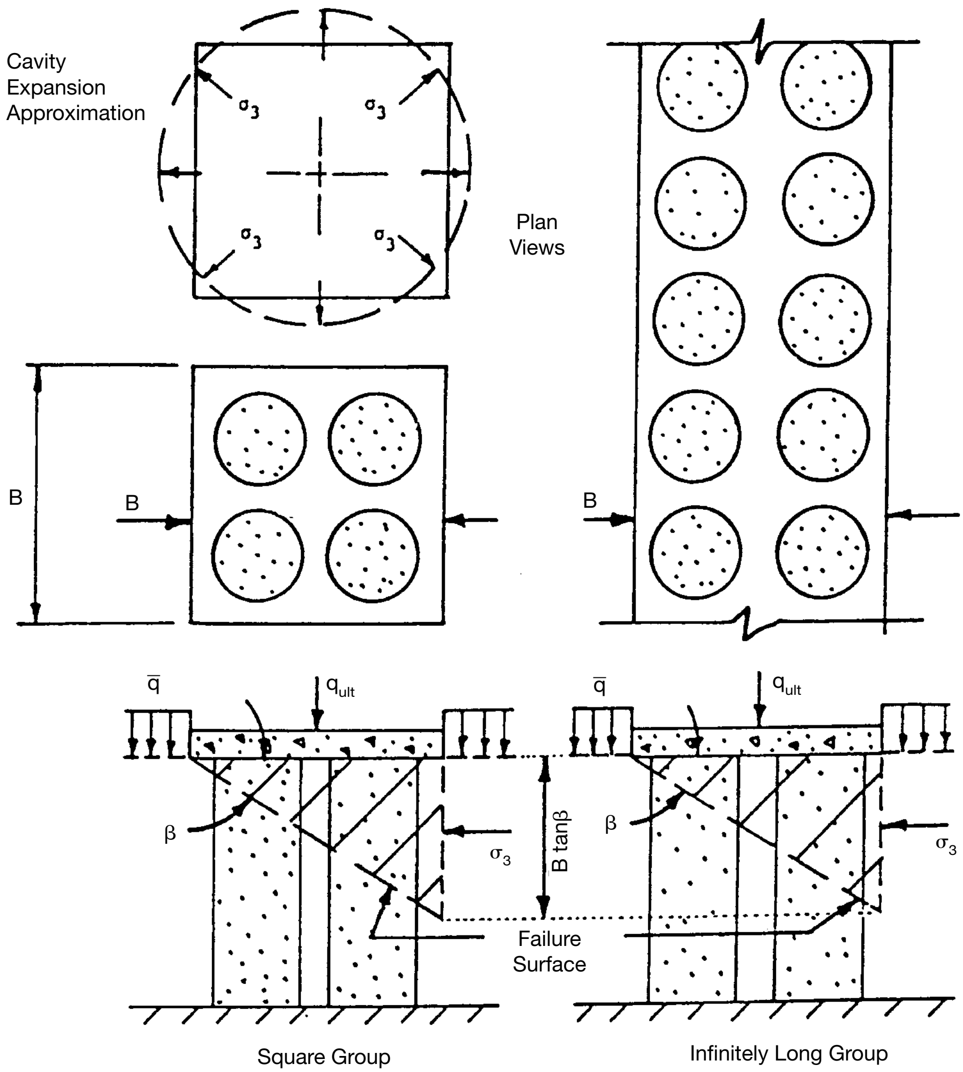


FIGURE 6A.182 Barksdale and Bachus' (1983) method to estimate the ultimate bearing capacity of a foundation supported by a group of reinforcing columns.

and is valid for either a shallow or deep groundwater table. The value of γ_m used in these cases should correspond to the appropriate groundwater and drainage conditions. That is, the saturated unit weight should be used for a shallow groundwater table and undrained (short-term) conditions; the effective (buoyant) unit weight should be used for a shallow groundwater table and drained (long-term) conditions; and the total (wet) unit weight should be used for a deep groundwater table. If the groundwater table is located within the height of the sliding wedge, the appropriate value of average vertical stress along the vertical face owing to the weight of the matrix soil below the bearing level should be used in place of $0.5\gamma_m B\tan\beta$ and added to $\bar{q}$. Total vertical stress should be used for undrained conditions and effective vertical stress for drained conditions, with the exception that $\bar{q}$ should be the *effective* surcharge pressure for undrained and drained conditions.

6A.5.2.4 Liquefaction Potential

Columnar reinforcement can mitigate the potential for liquefaction of saturated granular matrix soils in the following four ways (Baez 1995):

1. Columns act as drainage wells to reduce the buildup of excess pore water pressures
2. Increase the density the matrix soil adjacent to the columns
3. Increase the stress levels in the matrix soil adjacent to the columns
4. Reduce the shear stress carried by the matrix soil

The effectiveness of each type of granular column in reducing liquefaction potential by the four possible ways described above is summarized in Table 6A.17.

Granular columns must meet three requirements if they are to reduce significantly the excess pore water pressures generated in potentially liquefiable saturated granular soils during an earthquake (Seed and Booker 1976, Sonu et al. 1993): (1) To act as a free drain, the columnar material must have a permeability of at least 200 times that of the matrix soil; (2) the columns must effectively filter the matrix soil to prevent clogging by intrusion of finer particles into the columns; and (3) the columns must have sufficient void space to handle the volume of inflowing water required to keep the excess pore water pressures to an acceptably low level.

The increase in density of the matrix soil adjacent to stone columns can be significant. This also occurs in other types of granular columns to a lesser extent (see Table 6A.17). Baez (1995) evaluated this effect for vibratory stone columns at 18 sites based on an evaluation of nearly 400 sets of pretreatment and posttreatment data from standard penetration and cone penetration tests. Empirical correlations were developed for posttreatment normalized SPT blowcounts as a function of pretreatment values for area replacement ratios from 5 to 20%. Results of the empirical study are presented graphically in Fig. 6A.183 and are valid for uniform to medium silty sands with less than 15% fines and little or no clay content. When used in conjunction with Seed's method, this design chart can be used to estimate the area replacement ratio needed to achieve a desired factor of safety against liquefaction (see discussion of liquefaction potential in Section 6A.3.4.1).

The increase in horizontal stresses within the matrix soil adjacent to geopiers can be significant. This also occurs in other types of granular columns to a much lesser extent (see Table 6A.17). There

TABLE 6A.17 Comparison of effectiveness of various types of granular columns in controlling liquefaction potential

	Method of controlling liquefaction potential			
Type of granular column	Reduce buildup of excess pore water pressures by providing radial drainage	Increase density of adjacent matrix soil	Increase stress levels in adjacent matrix soil	Reduce shear stresses in matrix soil
Stone column	Significant[a]	Significant	Moderate	Moderate
Sand column	Significant[a]	Moderate	Some	Moderate
Geopier	Significant[a]	Some	Significant	Significant
Gravel drain	Significant[a]	Slight	Slight	Some
Sand drain	Significant[a]	Slight	Slight	Some

[a]Significant so long as the permeability of the column is greater than 200 times the permeability of the matrix soil, the column material acts as a filter (prevents intrusion of finer particles into the column), and there is sufficient void space in the columns to handle the volume of inflowing water required to keep the excess pore water pressures at a sufficiently low level.

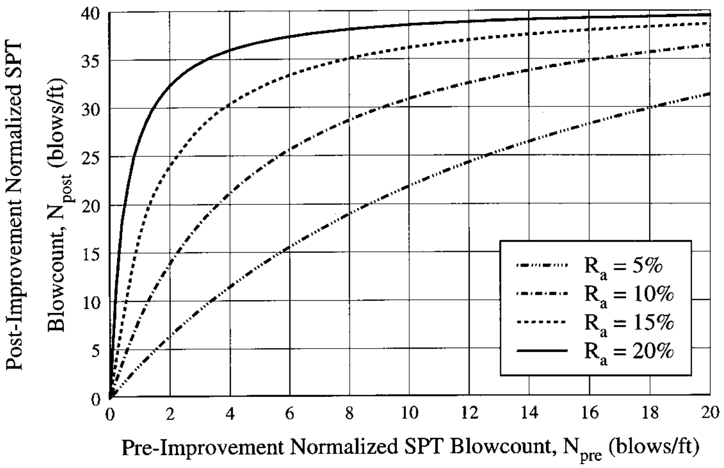


FIGURE 6A.183 Prediction of postimprovement normalized SPT blowcount as a function of preimprovement normalized SPT blowcount for uniform fine to medium silty sands with less than 15% fines and little or no clay content (from Baez 1995).

may also be some minor increases in the vertical stresses. This increase in stress level helps to reduce the potential for liquefaction by increasing the confining stresses, particularly for vertical shaking (see Fig. 6A.64).

During horizontal ground shaking, lateral shear stresses are generated in the soil. Within a columnar reinforced soil, a concentration of shear stresses on the columns occurs, similar to the concentration of normal stresses for vertical loads discussed previously. This results in a reduction in the shear stresses within the potentially liquefiable matrix soil. The magnitude of this reduction depends on the area replacement ratio and the ratio of the shear modulus for the granular columns to that for the matrix soil ($R_G = G_c/G_m$), according to the following equation:

$$\tau_m = \tau \frac{1}{R_a(R_G - 1) + 1} \tag{6A.92}$$

where τ is the average shear stress generated by the earthquake. Eq. 6A.92 can be rearranged to solve for area replacement ratio:

$$R_a = \frac{1}{R_G - 1}\left(\frac{\tau}{\tau_m} - 1\right) \tag{6A.93}$$

The factor of safety against liquefaction is generally defined as the available cyclic shear strength divided by the cyclic shear stress expected to be generated by the earthquake (Eq. 6A.20). The preimprovement factor of safety (FS_{pre}) can be introduced into Eq. 6A.93 as τ_m/τ, resulting in the following equation for the area replacement ratio required to produce a posttreatment factor of safety of one:

$$R_a = \frac{1}{R_G - 1}\left(\frac{1}{FS_{pre}} - 1\right) \tag{6A.94}$$

Eq. 6A.94 is shown graphically in Fig. 6A.184.

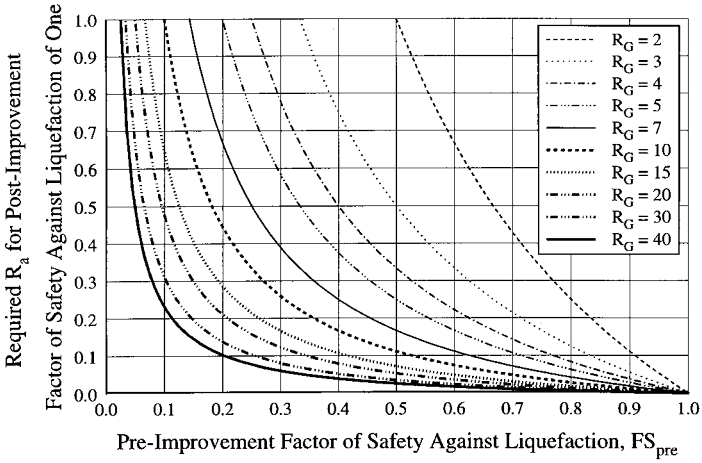


FIGURE 6A.184 Required area replacement ratio to achieve a posttreatment factor of safety of one against liquefaction considering shear stress redistribution only (from Baez 1995).

6A.5.2.5 Slope Stability

Columnar reinforcement can be used to increase the stability of man-made and natural slopes. In situations where fill or other load is placed on top of the columns, such as occurs when stabilizing a soil deposit and then placing an embankment on top of the reinforced soil (for example, see Example Problem 6A.2), the concentration of stress on the columns provides a very efficient and cost-effective reinforced zone. When stabilizing natural slopes or landslides, however, stress concentrations generally do not occur and the method is less efficient but still technically effective.

Most slope stability analyses conducted in engineering practice involve the use of a two-dimensional computer program. Although all slope stability problems are three-dimensional, two-dimensional analyses are usually conducted for three primary reasons:

1. Many slopes are long in comparison to their height and width and thus are reasonably represented as two-dimensional problems.
2. Three-dimensional analyses required about an order or magnitude more time to perform than two-dimensional analyses.
3. Use of a two-dimensional analysis is nearly always conservative.

Two methods for performing slope stability analyses involving columnar reinforcement are the *composite strength method* and the *profile method*. Either method can be used with standard slope stability computer programs. In the composite strength method, which is valid only when no stress concentration is present, the reinforced zone is treated as a single composite material with one set of Mohr–Coulomb strength parameters (ϕ_{comp} and c_{comp}) and a composite unit weight (γ_{comp}). Some judgment is required to determine how far outside the edge of the outermost row of columns the reinforced zone should extend. c_{comp} can be calculated from Eq. 6A.89 and ϕ_{comp} from Eq. 6A.88 with $\mu_c = \mu_m = 1$. γ_{comp} can be calculated from the following equation:

$$\gamma_{comp} = \gamma_c R_a + \gamma_m (1 - R_a) \tag{6A.95}$$

The values used for γ_c and γ_m should correspond to the location of the groundwater table and type of strength analysis being conducted. Use total unit weights for soils above the groundwater and saturated unit weights for undrained conditions below the water table. Use effective (buoyant) unit weights for drained conditions below the groundwater table. The composite strength method should not be used when $c_c = 0$ and $\phi_m = 0$, which occurs for static loads, granular columns, and saturated clay as the matrix soil in the unconsolidated–undrained condition.

The profile method is preferred by the author for most cases. In this method, each row of columns and the matrix soil between the columns within that row are converted into three fictitious two-dimensional strips of materials: a strip of columnar material in the center sandwiched between a strip of matrix material on each side. This conversion is illustrated in Fig. 6A.185. The corresponding equations for the width of the fictitious columnar strip (W_{fc}) and the width of each fictitious matrix strip (W_{fm}) are as follows:

$$W_{fc} = R_a d_c \tag{6A.96}$$

$$W_{fm} = 0.5(1—R_a) d_c \tag{6A.97}$$

When stress concentrations are present, they are accounted for by placing a zone of fictitious strips between the fill and the underlying soil as illustrated in Fig. 6A.186. The fictitious strips above the columns have a positive unit weight to account for the additional stress carried by the columns

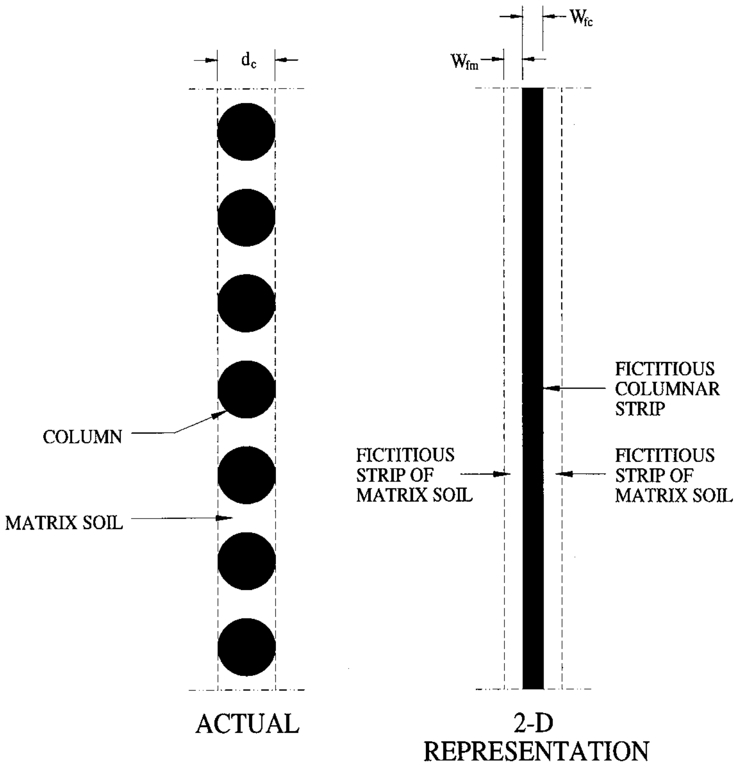


FIGURE 6A.185 Plan view of long row of columns with matrix soil and two-dimensional representation using profile slope stability method.

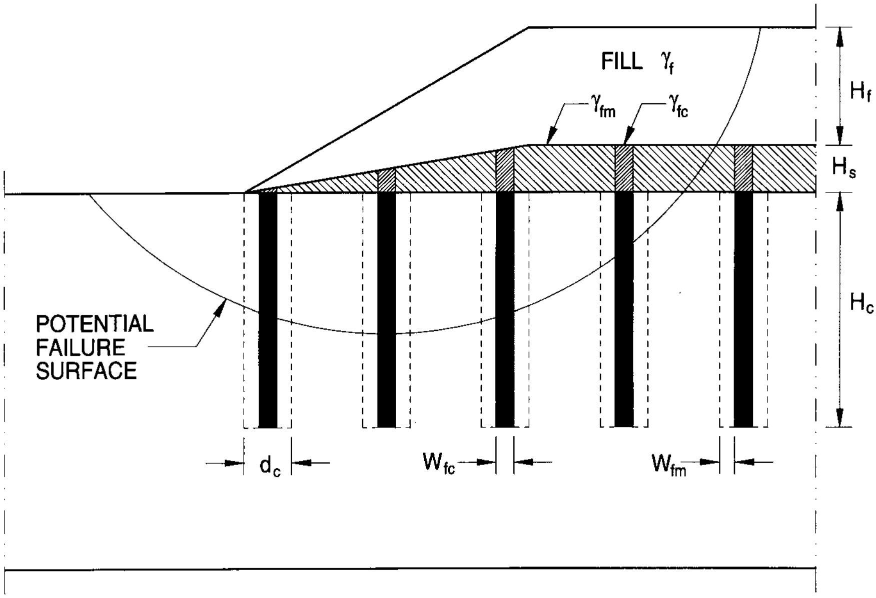


FIGURE 6A.186 Accounting for stress concentration in the profile method.

($\Delta\sigma_c$). The fictitious strips above the matrix soil have a negative unit weight to account for the corresponding reduction in stress on the matrix soil ($\Delta\sigma_m$). Equations for $\Delta\sigma_c$ and $\Delta\sigma_m$ can be written in the following form:

$$\Delta\sigma_c = \sigma_c - \sigma = \mu_c\sigma - \sigma = \sigma(\mu_c - 1) \tag{6A.98}$$

$$\Delta\sigma_m = \sigma_m - \sigma = \mu_m\sigma - \sigma = \sigma(\mu_m - 1) \tag{6A.99}$$

The corresponding unit weights of the fictitious strips above the columns (γ_{fc}) and the fictitious strips above the matrix soil (γ_{fm}) are given as follows:

$$\gamma_{fc} = \frac{\Delta\sigma_c}{H_s} = \frac{\sigma(\mu_c - 1)}{H_s} = \frac{\gamma_f H_f(\mu_c - 1)}{H_s} \tag{6A.100}$$

$$\gamma_{fm} = \frac{\Delta\sigma_m}{H_s} = \frac{\sigma(\mu_m - 1)}{H_s} = \frac{\gamma_f H_f(\mu_m - 1)}{H_s} \tag{6A.101}$$

where γ_f = unit weight of the fill
H_f = height of the fill
H_s = height of the fictitious strip

The thickness of the fictitious strips should be small to avoid changing the geometry of the problem. Barksdale and Bachus (1983) recommended using a maximum value of H_s in the range of 0.25 to 0.5 ft (75 to 100 mm). The ratio H_s/H_f should be constant for all the fictitious strips, tapering to zero at the edge of an embankment as shown in Fig. 6A.186. In addition, the fictitious strips must be assigned zero shear strength ($\phi = 0$ and $c = 0$) or else very small values if zeroes are not allowed by the computer program. Furthermore, if allowed by the program, limits should be placed on the generated failure surfaces so that the critical failure surface is not controlled by the weak, fictitious strips. If limits are not possible, a final check of the critical failure surface should be made to ensure that a significant part of it does not go through the zone of fictitious strips. Potential failure surfaces not meeting this criterion should be eliminated from consideration because they are not realistic.

6A.5.2.6 Lateral Sliding Resistance

For most types of columnar reinforcement, the resistance to lateral sliding of footings bearing on soil reinforced with columns comes from two sources: shearing resistance along the bottom of the footing and passive resistance of the soil adjacent to the side of the footing. The passive resistance of the adjacent soil is unaffected by the reinforcement unless the columns extend outside the footprint of the footing and above the bearing level. Traditional methods can be used to estimate this passive resistance. The shearing resistance along the bottom of the footing is significantly enhanced by the concentration of stress on the stronger columns. The maximum horizontal resisting force that can be developed from shearing along the bottom of the footing ($R_{h\text{-max}}$) can be estimated from the following equation:

$$R_{h\text{-max}} = A_c\,(a_c + q_c \tan \delta_c) + A_m\,(a_m + q_m \tan \delta_m) \tag{6A.102}$$

where A_c = area of columns at the bearing level within the footprint of the footing
A_m = area of matrix soil at the bearing level within the footprint of the footing
a_c = maximum adhesive stress for sliding along the columnar material
a_m = maximum adhesive stress for sliding along the matrix soil
δ_c = angle of friction for sliding of the footing along the columnar material
δ_m = angle of friction for sliding of the footing along the matrix soil

If the bottom of the footing is rough, which is generally the case for concrete cast directly against the ground, δ_c, δ_m, a_c, and a_m are approximately equal to ϕ_c, ϕ_m, c_c, and c_m, respectively. Stronger materials generally achieve peak lateral resistance at smaller displacements of the footing than do weaker soils. If the columnar material is dilative at the expected level of q_c, the columnar adhesive and frictional resistance may reach a peak and drop into the residual range before peak resistance is achieved in the matrix soil. Therefore, Eq. 6A.102 may be somewhat unconservative for these conditions. However, in many cases the contribution of the matrix soil to the total sliding resistance is small so that this error is insignificant. So long as a reasonable factor of safety is applied to Eq. 6A.102 to obtain an allowable or design value of R_h, use of this equation is acceptable. If desired, laboratory direct shear tests can be conducted on the columnar and matrix materials at appropriate vertical stress levels to determine if stress-displacement incompatibility is a potential problem.

When uplift geopiers are used, additional lateral resistance results from passive pressure generated on the uplift bars as they are pushed into the geopier material. The results from full-scale tests and numerical analyses conducted on footings supported by uplift geopiers indicate that this phenomenon may produce as much as 40% of the total lateral resistance at large displacements of the footings (Lawton 1999).

6A.5.2.7 Uplift Capacity of Geopiers

The results of a study on the behavior of single geopiers during uplift has been reported by Hsu (2000). Based on the results of a series of uplift tests conducted on geopiers, methods to predict the maximum pullout force ($T_{\max}$) of a single geopier have been developed. These methods can also be extended to groups of uplift geopiers supporting a footing.

The following key factors were found to affect the uplift capacity of geopier foundations:

1. During installation of geopiers, the horizontal stresses in the adjacent matrix soil are increased to limiting passive pressures to a depth of about 6 ft (2 m) below the top of the geopier. Below this depth, the horizontal stresses are increased but not to full limiting passive values.
2. In cohesive soils, remolding of the matrix soil occurs, resulting in a complete loss of the cohesion intercept. It is believed that in the long-term, the cohesive soil will regain strength owing to thixotropy and the full cohesion intercept will be redeveloped. However, additional research is needed to verify that this occurs.
3. During uplift loading, the horizontal stress along the interface of the geopier and the matrix soil decreases by about 1 psi (7 kPa) from the initial condition (just prior to loading) to failure.
4. Possible shapes of the potential failure surfaces can take two forms: (a) the surface of a cylinder, or (b) the surface of a cylinder in the lower portion transitioning to the surface of a truncated cone in the upper portion.

Based on these factors and assuming the failure surface occurs along the surface of the geopier, the following equations can be used to estimate T_{max} for a single geopier when the Mohr–Coulomb strength parameters (ϕ and c or ϕ' and c') are known for all layers of the matrix soil adjacent to the geopiers.

For a height of geopier (H_g) ≤ 6 ft (1.8 m):

$$T_{max}(\text{single}) = W' + \pi d_g \sum_{i=1}^{i=l} (K_{pi} \cdot \sigma'_{vi} - \Delta\sigma_h) H_i \cdot \tan \phi'_i \tag{6A.103}$$

For H_g > 6 ft (1.8 m):

$$T_{max}(\text{single}) = W' + \pi d_g \left[\sum_{j=1}^{j=m} (K_{pj} \cdot \sigma'_{vj} - \Delta\sigma_h) H_j \cdot \tan \phi'_j + \sum_{k=1}^{k=n} [0.5(K_{0k} + K_{pk})\sigma'_{vk} - \Delta\sigma_h] H_k \cdot \tan \phi'_k \right] \tag{6A.104}$$

where W' = effective weight of the geopier = $0.25\pi d_g^2(\gamma_g H_{ga} + \gamma'_g H_{gb})$
d_g = nominal diameter of the geopier
γ_g = total unit weight of the geopier above the groundwater table
γ'_g = effective unit weight of the geopier below the groundwater table
H_{ga} = height of the geopier above the groundwater table
H_{gb} = height of the geopier below the groundwater table
H_i = height of the ith layer of matrix soil
i, j, k = summation variables referring to the numbering of the matrix soil layers
l = number of layers of matrix soil adjacent to a geopier when H_g < 6 ft (1.8 m)
m = number of layers of matrix soil adjacent to a geopier for the upper 6 ft (1.8 m) when H_g > 6 ft (1.8m)
n = number of layers of matrix soil adjacent to a geopier for the portion below 6 ft (1.8 m) from the top of the geopier when H_g > 6 ft (1.8 m)
K_{pi} = Rankine's coefficient of passive earth pressure for the ith layer of matrix soil = $\tan^2(45° + \phi'_i/2)$
K_{0k} = coefficient of lateral earth pressure at rest for the kth layer of matrix soil
ϕ'_i = coefficient of internal friction for the ith layer of matrix soil
σ'_{vi} = average vertical overburden pressure for the ith layer of matrix soil
$\Delta\sigma_h$ = reduction in horizontal stress during uplift loading = 1 psi = 0.144 ksf = 7 kPa

If there is soil above the top of the geopier and it will remain throughout the design life of the structure, the weight of this soil can be added into the equations for T_{max} (single). However, generally this weight is insignificant compared to the other components. It is recommended that the borehole shear test (Handy and Fox 1967) be used to obtain the strength parameters of the matrix soil layers because the shearing action during this test simulates the shearing action in the matrix soil that occurs during uplift of a geopier.

T_{max} can also be estimated from CPT data when it is available using the following equations.

For all soils:

$$T_{max}(\text{single}) = W' + 1.6\pi d_g \sum_{i=1}^{i=n} f_{si} H_i \tag{6A.105}$$

where f_{si} is the average CPT sleeve resistance for the height increment H_i.

For cohesionless soils:

$$T_{max}(\text{single}) = W' + \frac{\pi d_g}{47} \sum_{i=1}^{i=n} q_{ci} H_i \tag{6A.106}$$

For cohesive soils:

$$T_{max}(\text{single}) = W' + \frac{\pi d_g}{23} \sum_{i=1}^{i=n} q_{ci} H_i \tag{6A.107}$$

where q_{ci} is the average CPT tip resistance for the height increment H_i.

This method can be extended to calculate the uplift capacity of a group of geopiers supporting a footing by assuming that the failure surface is vertical and, in plan view, takes the shape of the geometric figure determined by drawing straight lines between the outermost points on the geopier group. This technique is illustrated in Fig. 6A.187 for 2 by 2 and 3 by 4 groups of geopiers. The capacity determined in this manner cannot be more than the total uplift capacity of the geopiers in the group assuming that each geopier behaves individually. A similar technique is commonly used in the analysis of the uplift capacity of groups of piles. This method is described in equation form as follows.

For $H_g \leq 6$ ft (1.8 m):

$$T_{max}(\text{group}) = W' + W_f + p \sum_{i=1}^{i=l} (K_{pi} \cdot \sigma'_{vi} - \Delta\sigma_h) H_i \cdot \tan \phi'_i \leq W_f + N_g \cdot T_{max}(\text{single}) \tag{6A.108}$$

For $H_g > 6$ ft (1.8 m):

$$T_{max}(\text{group}) = W' + W_f + p\left[\sum_{j=1}^{j=m} (K_{pj} \cdot \sigma'_{vj} - \Delta\sigma_h) H_j \cdot \tan \phi'_j + \sum_{k=1}^{k=n} [0.5(K_{0k} + K_{pk})\sigma'_{vk} - \Delta\sigma_h] H_k \cdot \tan \phi'_k\right]$$
$$\leq W_f + N_g \cdot T_{max}(\text{single}) \tag{6A.109}$$

where W' = effective weight of geopiers and matrix soil within group mass
W_f = weight of footing
p = perimeter length of group mass

T_{max}(single) is the maximum uplift capacity for a single geopier, from Eqs. 6A.103 through 6A.107

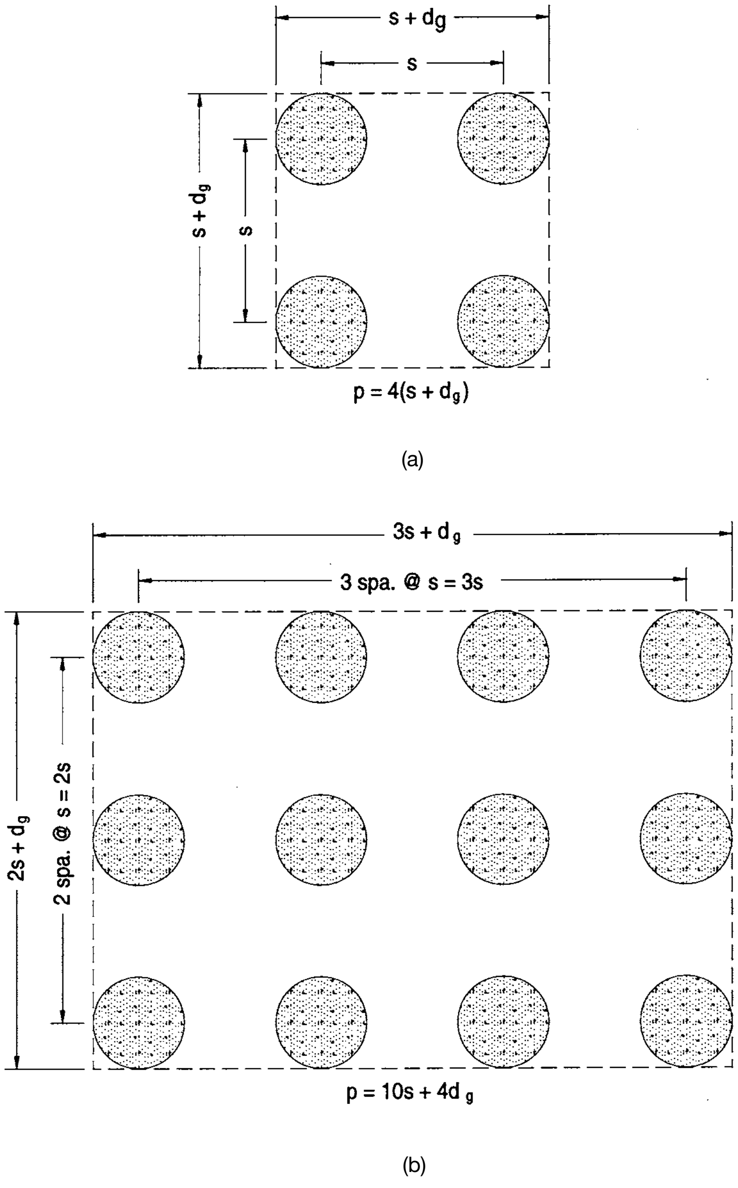


FIGURE 6A.187 Determination of geopier group perimeter length for (a) 2 by 2 group, and (b) 3 by 4 group.

For all soils:

$$T_{\max}(\text{group}) = W' + W_f + 1.6p\sum_{i=1}^{i=n} f_{si}H_i \tag{6A.110}$$

For cohesionless soils:

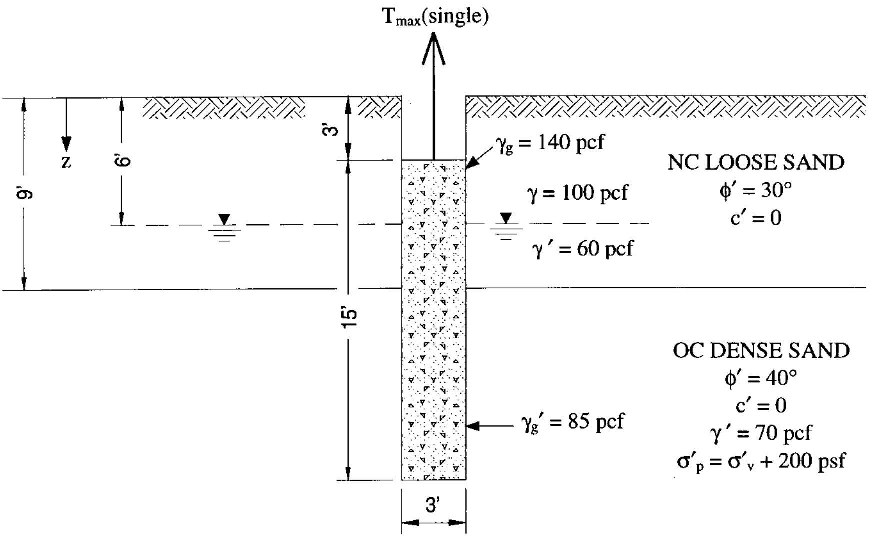
$$T_{\max}(\text{group}) = W' + W_f + \frac{p}{47}\sum_{i=1}^{i=n} q_{ci}H_i \tag{6A.111}$$

For cohesive soils:

$$T_{\max}(\text{group}) = W' + W_f + \frac{p}{23}\sum_{i=1}^{i=n} q_{ci}H_i \tag{6A.112}$$

If there is soil above the top of the footing and it will remain throughout the design life of the structure, the weight of this soil can be added into the equations for $T_{\max}$ (group). However, generally this weight is insignificant compared to the other components. The use of the equations to estimate $T_{\max}$ (single) and $T_{\max}$ (group) are illustrated in the following examples.

Example Problem 6A.3 Estimate the maximum uplift capacity of the single geopier shown in the diagram below. The geopier is 3 ft (0.91 m) in diameter, 15 ft (4.57 m) tall, has a total unit weight above the groundwater table (γ_g) of 140 pcf (22.0 kN/m^3), and an effective (buoyant) unit weight below the groundwater table (γ_g') of 85 pcf (13.4 kN/m^3). The top of the geopier is 3 ft (0.91 m) below the ground surface. The soils at the site consist of two layers. The upper layer consists of normally consolidated, loose sand that is 9 ft (2.74 m) thick with a total unit weight (γ) of 100 pcf (15.7 kN/m^3), an effective unit weight (γ') of 60 pcf (9.43 kN/m^3), an effective friction angle (ϕ') of 30°, and an effective cohesion intercept (c') of zero. This upper layer is underlain by a deep, overconsolidated, dense sand layer with $\gamma' = 70$ pcf (11.0 kN/m^3), $\phi' = 40°$, and $c' = 0$. The dense sand layer is overconsolidated owing to removal of overburden at some time in the past, with the preconsolidation pressure (σ_p') equal to the effective vertical stress (σ_v') plus 200 psf (9.6 kPa). The groundwater table is 6 ft (1.83 m) below the ground surface. z is defined as the depth below the ground surface.



Solution: For purposes of analysis, the matrix soils will be divided into three layers: Layer 1 is loose sand from the top of the geopier to the groundwater table (z = 3 to 6 ft = 0.91 to 1.83 m); Layer 2 is loose sand from the groundwater table to the bottom depth where full passive pressures are assumed to develop (z = 6 to 9 ft = 1.83 to 2.74 m); and Layer 3 from the top of the dense sand to the bottom of the geopier (z = 9 to 18 ft = 2.74 to 5.49 m). Since H_g = 15 ft (4.6 m) > 6 ft (1.8 m), Eq. 6A.104 will be used to calculated T_{max} (single).

Layer 1: This layer is completely within the zone where full passive pressures are assumed to develop in the matrix soils adjacent to the geopiers.

$K_{p1} = \tan^2(45° + \phi_1'/2) = \tan^2(45° + 30°/2) = 3$

$\sigma_{v1}'(z = 4.5 \text{ ft}) = \gamma z = (100)(4.5) = 450$ psf = 0.45 ksf (21.5 kPa)

$\Delta\sigma_h = 0.144$ ksf (7.0 kPa)

$H_1 = 3$ ft (0.91 m)

Maximum uplift shearing force = $\pi d_g (K_{p1}\sigma_{v1}' - \Delta\sigma_h) H_1 \tan \phi_1' = \pi(3)[(3)(0.45) - 0.144](3) \tan 30°$ = 19.7 kips (87.6 kN)

$W'_1 = 0.25\ \pi\, d_g^2\, \gamma_g\, H_1 = 0.25\pi(3)^2(0.140)(3) = 3.0$ kips (13.2 kN)

T_{max} (single) for Layer 1 = 19.7 + 3.0 = 22.7 kips (101 kN)

Layer 2: This layer is also completely within the zone where full passive pressures are assumed to develop in the matrix soils adjacent to the geopiers.

$K_{p2} = K_{p1} = 3$

$\sigma_{v2}'(z = 7.5 \text{ ft}) = (100)(6) + (60)(1.5) = 690$ psf = 0.69 ksf (33.0 kPa)

$H_2 = 3$ ft (0.91 m)

Maximum uplift shearing force = $\pi(3)[(3)(0.69) - 0.144](3) \tan 30°$ = 31.4 kips (140 kN)

$W_2' = 0.25\ \pi d_g^2\, \gamma_g'\, H_2 = 0.25\pi(3^2)(0.085)(3) = 1.8$ kips (8.0 kN)

T_{max} (single) for Layer 2 = 31.4 + 1.8 = 33.2 kips (148 kN)

Layer 3: This layer is within the zone where the pressures are assumed to be halfway between at-rest and full passive conditions.

$\sigma_{v3}'(z = 13.5 \text{ ft}) = (100)(6) + (60)(3) + (70)(4.5) = 1{,}095$ psf = 1.095 ksf (52.4 kPa)

$\sigma_{p3}'(z = 13.5 \text{ ft}) = \sigma_{v3}' + 200 = 1{,}095 + 200 = 1{,}295$ psf = 1.295 ksf (62.0 kPa)

Overconsolidation Ratio, OCR = $\sigma_{p3}'/\sigma_{v3}' = 1.295/1.095 = 1.183$

$K_{03} = (1 - \sin \phi_3')\, \text{OCR}^{0.5} = (1 - \sin 40°)(1.183)^{0.5} = 0.388$

$K_{p3} = \tan^2(45° + 40°/2) = 4.599$

$H_3 = 9$ ft (2.74 m)

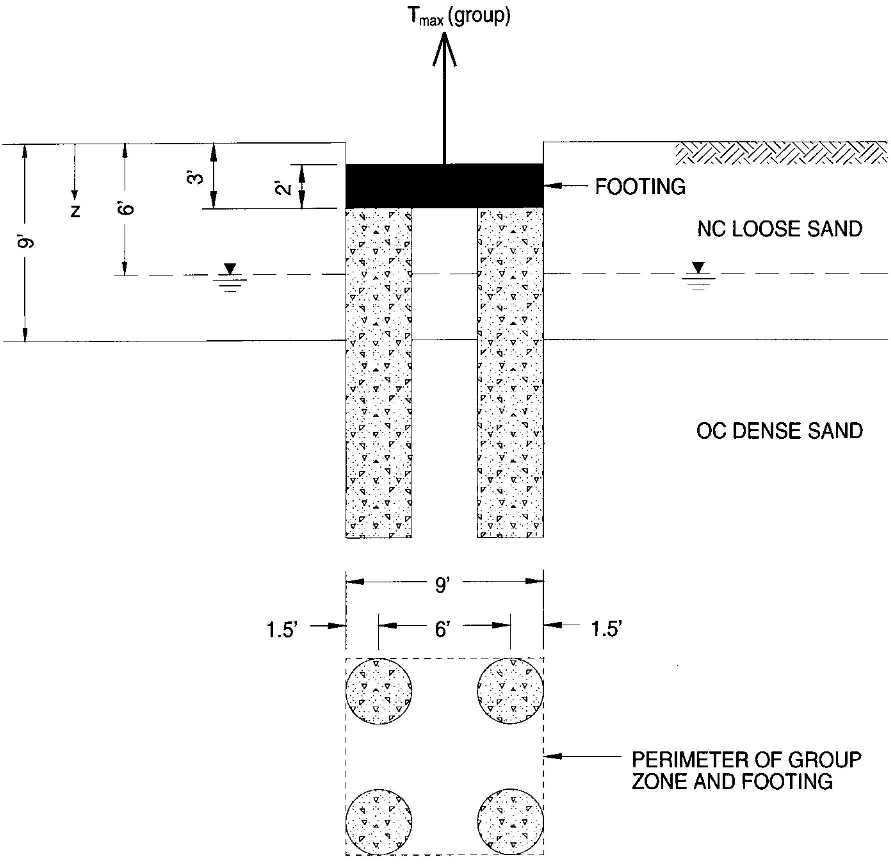
Maximum uplift shearing force = $\pi d_g[0.5(K_{03} + K_{p3})\ \sigma_{v1}' - \Delta\sigma_h)\ H_3 \tan \phi_3' = \pi(3)[0.5(0.388 + 4.599)(1.095) - 0.144](9) \tan 40°$ = 184.1 kips (87.6 kN)

$W_3' = 0.25\pi(3^2)(0.085)(9) = 5.4$ kips (24.1 kN)

T_{max} (single) for Layer 1 = 184.1 + 5.4 = 189.5 kips (843 kN)

All Three Layers: T_{max} (single) = 22.7 + 33.2 + 189.5 = 245 kips (1,090 kN)

Example Problem 6A.4 Estimate the maximum uplift capacity of a group of four geopiers with the same dimensions and soil conditions as in Example Problem 6A.3. The four geopiers will support a 9-ft (2.74-m) square footing with a thickness of 2 ft (0.61 m) as shown in the figure below. The unit weight of the footing (γ_f) is assumed to be 150 pcf (23.6 kN/m^3).



Solution: Since $H_g > 6$ ft (1.8 m), Eq. 6A.109 will be used to calculated T_{max} (group). The same three layers used in Example Problem 6A.3 will be used for the group.

Layer 1:

$p = 4(s + d_g) = 4(6 + 3) = 36$ ft (11.0 m)

Maximum uplift shearing force $= p(K_{p1}\sigma'_{v1} - \Delta\sigma_h)\, H_1 \tan \phi'_1 = 36[(3)(0.45) - 0.144](3) \tan 30° =$ 75.2 kips (335 kN)

Area of geopiers, $A_g = N_g\,(0.25\,\pi\, d_g^2) = 4[0.25\,\pi\,(3)^2] = 28.27$ ft² (2.63 m²)

Area of matrix soil, $A_m = 9^2 - 28.27 = 52.73$ ft² (2.63 m²)

$W'_1 = H_1(A_g\gamma_g + A_m\gamma) = 3[(28.27)(0.140) + (52.73)(0.100)] = 27.7$ kips (123 kN)

T_{max} (group) for Layer 1 = 75.2 + 27.7 = 103 kips

Layer 2:

Maximum uplift shearing force = 36[(3)(0.69) – 0.144](3) tan 30° = 120.1 kips (534 kN)

$W'_2 = H_2(A_g\gamma'_g + A_m\,\gamma') = 3[(28.27)(0.085) + (52.73)(0.060)] = 16.7$ kips (123 kN)

T_{max} (group) for Layer 2 = 120.1 + 16.7 = 137 kips (657 kN)

Layer 3:

Maximum uplift shearing force = $p[0.5(K_{03} + K_{p3})\ \sigma'_{v3} - \Delta\sigma_h)\ H_3 \tan \phi'_3$ = 36[0.5(0.388 + 4.599)(1.095) – 0.144](9) tan 40° = 703.2 kips (3,130 kN)

W'_2 = 9[(28.27)(0.085) + (52.73)(0.070)] = 54.8 kips (244 kN)

T_{max} (group) for Layer 3 = 703.2 + 54.8 = 758 kips (3,370 kN)

Weight of Footing:

$W_f = (9)^2(2)(0.150)$ = 24.3 kips (108 kN)

Total Uplift Capacity for Group:

T_{max} (group) = 103 + 137 + 758 + 24 = 1,022 kips (4,550 kN) $\leq W_f + N_g\ T_{max}$ (single) = 24 + 4(245) = 1,004 kips (4,470 kN)

So single action controls and T_{max} (group) = 1,004 kips (4,470 kN)

6A.6 PRECOMPRESSION

Precompression or *preloading* of weak and compressible soils to increase their strength and stiffness is one of the oldest and most widely used methods of soil improvement (Mitchell 1981). In its simplest form, the technique involves increasing the effective stress within the bearing soils at a site to the anticipated final level under the structure to be built there. In some instances, the time required for the precompression to occur may be longer than desired or acceptable, in which case the process can be accelerated using *surcharging, vertical drains,* or both. Surcharging involves the application of an additional load beyond that necessary to achieve the final effective stresses. Vertical drains are used in saturated, fine-grained soils to hasten the rate at which primary consolidation occurs. Details of these procedures are provided in subsequent sections.

Precompression is especially well suited for use when the bearing soils will undergo large volume decreases and strength increases under sustained static loads, and when there is adequate time for the required compressions to occur. The types of soils that meet these requirements are generally normally consolidated or lightly overconsolidated, loose granular or soft cohesive soils.

6A.6.1 Types of Preloads

The effective stresses in the bearing soils can be increased by either increasing the total stress, decreasing the pore air or water pressures, or a combination thereof. Generally, the increase in effective stress is accomplished by adding weight to the ground surface to increase the total stress within the underlying soils. The added weight is usually in the form of an earthen embankment, but can also be in many other forms including water in tanks, water in lined ponds, concrete highway barriers, and rubble. The total stress can also be increased by jacking against an anchored reaction system. If the groundwater table is at or near the ground surface, a drainage medium is usually needed on the ground surface to provide surface drainage for the water that is squeezed from the underlying soils. The drainage medium typically consists of a geotextile or granular drainage layer. These total stress methods are generally simple in execution but can cause stability problems in the underlying soils, especially in soft, saturated, cohesive soils if the loads are applied quickly.

Lowering the groundwater table at a site increases the effective stresses by reducing the pore water pressures and reducing slightly the total stress owing to a decrease in unit weight of the soils above the lowered groundwater table. Applying a vacuum beneath an impervious membrane—called *vacuum preloading*—reduces the pore air pressure in an unsaturated soil or reduces the pore water pressure in a saturated soil, which results in an increase in effective stress. Additional details on this technique can be founded in Holtz and Wager (1975), Pilot (1977), and Cognon et al. (1994).

In electro-osmosis, water is removed from saturated, fine-grained soils by application of a direct current field, producing increases in effective stress with the treated soil mass. If stabilizing chemicals are injected into the soil while water is being removed, the process is called electro-kinetic injection. Further information on electro-osmosis and electro-kinetic injection can be found in Mitchell (1981, 1993), Esrig (1968), Wan and Mitchell (1976), Mitchell and Wan (1977), and Pilot (1977). Although preloading by lowering the groundwater table, vacuum, or electro-osmosis eliminates the problem with stability of the bearing soils, these methods are substantially more complex to design, execute, and monitor than the total stress methods.

6A.6.2 Simple Precompression

The term *simple precompression* is used for the case where a preload equal to a future site load is applied (Hausmann 1990). When primary compression or consolidation is nearly complete, the preload is removed and the new structure is built. This process is illustrated in Fig. 6A.188 for the case

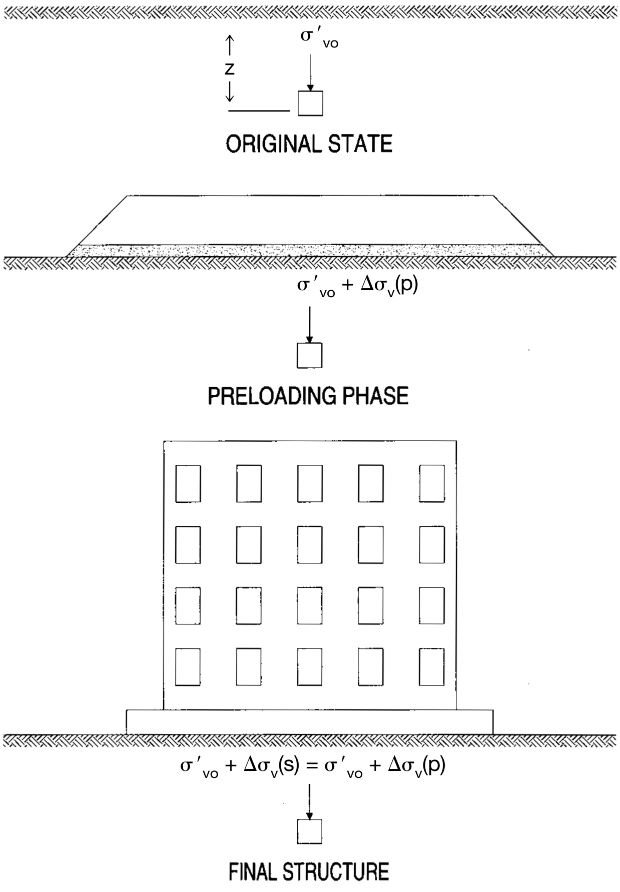


FIGURE 6A.188 Illustration of simple precompression (modified from Hausmann 1990).

of preloading with an earthen fill at a site where a building supported on a mat foundation will be constructed. In this case, the increase in total stress at any location within the bearing soil caused by the preload [$\Delta\sigma_v(p)$] is equal to the expected increase in total stress to be caused by the structure at the same point [$\Delta\sigma_v(s)$].

If the preload could be removed instantaneously and the structure built instantaneously, the primary compression settlement (S_c) under the load of the structure [$\Delta\sigma_v(s)$] would theoretically be zero. In reality, the magnitude of S_c resulting from $\Delta\sigma_v(s)$ is approximately equal to the amount of primary heave that occurs upon removal of the preload. The magnitude of primary heave depends on the rate at which it occurs and how long it takes to remove the preload and build the structure. The rate of primary heave is primarily a function of the moisture condition and permeability of the soil, and the lengths of the drainage drainage paths if the soil is saturated. For example, a coarse sand is highly permeable regardless of its moisture condition and primary heave will be complete within a few days after the preload is removed. On the opposite extreme, the primary heave for a saturated, low-permeability clay with long drainage paths may require many years to complete, and thus the magnitude of primary heave would be very small for typical removal and construction times.

Soils that are overconsolidated behave essentially elastically when loaded to stresses up to the preconsolidation pressure (σ'_p). Precompression is ineffective in overconsolidated soils when the final stress under the load of the structure [$\sigma'_{vo} + \Delta\sigma_v(s)$] is less than the preexisting σ'_p and full primary heave occurs upon removal of the preload. That is, the magnitude of S_c (and the strength of the soil) will be about the same whether or not precompression is performed. Therefore, precompression should only be considered when at least one of the following conditions is present:

1. The soil is normally consolidated
2. The soil is overconsolidated but $\sigma'_{vo} + \Delta\sigma_v(s)$ is somewhat greater than the preexisting σ'_p.
3. The amount of rebound that will occur between removal of the overburden and completion of the structure will be small.

The total settlement or heave (S_t) that the structure will undergo throughout its design life is given by the following equation:

$$S_t = S_i + S_c + S_s + S_m \tag{6A.113}$$

where S_i = immediate distortion settlement
S_s = settlement from secondary compression
S_m = settlement or heave caused by changes in moisture condition (swelling, shrinkage, or collapse)

Precompression is generally not a viable treatment method for soils with high potential for S_m. Highly expansive clays generally are chemically stabilized to reduce problems from swelling and shrinkage. Highly collapsible soils are generally densified using dynamic compaction or other densification techniques, or chemically stabilized, to reduce collapse potential. Thus, for precompression, Eq. 6A.113 is generally reduced to one of the following equations, depending on the time required for primary consolidation to be nearly complete (t_{pr}) in relation to the design life of the structure (t_d).

For $t_{pr} \leq t_d$:

$$S_t = S_i + S_c(t_{pr}) + S_s(t_d) \tag{6A.114a}$$

For $t_{pr} > t_d$:

$$S_t = S_i + S_c(t_d) \tag{6A.114b}$$

S_i results primarily from distortion of the soil mass (change in shape without change in volume) at the edges of the loaded area, and generally increases in significance as the width of the loaded area decreases in relation to the height of the compressible layer. S_i is theoretically equal to zero for true one-dimensional compression.

Although secondary compression occurs during primary consolidation, it is difficult to determine the magnitudes of the two phenomena. Therefore, it is generally assumed that secondary compression does not occur until primary consolidation is essentially complete. S_s is usually calculated from the following equation:

$$S_s = \frac{C_{\alpha e}}{1 + e_{pr}} H_{pr} \log \frac{t_d}{t_{pr}} \tag{6A.115}$$

where $C_{\alpha e} = -\dfrac{\Delta e}{\Delta(\log t)}$

e_{pr} = void ratio at t_{pr}
H_{pr} = height of the compressible layer at t_{pr}

e_{pr} can be difficult to determine for three-dimensional loading conditions, so it can be assumed that $H_{pr}/(1 + e_{pr}) \approx H_0/(1 + e_0)$, with little error generally introduced by this assumption. One of the additional benefits of precompressing a normally consolidated clay is that it reduces $C_{\alpha e}$ to about 10 to 50% of the normally consolidated value (Hausmann 1990).

S_c in saturated, fine-grained soils is typically estimated from one of the following equations.

For virgin compression:

$$S_c \sum_{i=1}^{i=n} = \frac{C_{cei}}{1 + e_{1i}} H_{1i} \log \frac{\sigma'_{v2i}}{\sigma'_{v1i}} \tag{6A.116a}$$

For rebound or recompression to $\sigma'_{v2} \leq \sigma'_p$:

$$S_c \sum_{i=1}^{i=n} = \frac{C_{rei}}{1 + e_{1i}} H_{1i} \log \frac{\sigma'_{v2i}}{\sigma'_{v1i}} \tag{6A.116b}$$

For recompression to $\sigma'_{v2} > \sigma'_p$:

$$S_c \sum_{i=1}^{i=n} = \frac{C_{rei}}{1 + e_{1i}} H_{1i} \log \frac{\sigma'_{pi}}{\sigma'_{v1i}} + \sum_{i=1}^{i=n} \frac{C_{cei}}{1 + e_{pi}} H_{pi} \log \frac{\sigma'_{v2i}}{\sigma'_{pi}} \tag{6A.116c}$$

where i = integer summation variable
n = number of sublayers into which the compressible layer is divided
C_{cei} = virgin compression index based on void ratio for the ith sublayer
C_{rei} = recompression/rebound index based on void ratio for the ith sublayer
H_{1i} = height of the ith sublayer when loading begins
H_{pi} = height of the ith sublayer when $\sigma'_{vi} = \sigma'_{pi}$
e_{1i} = void ratio of the ith sublayer when loading begins
e_{pi} = void ratio of the ith sublayer when $\sigma'_{vi} = \sigma'_{pi}$
σ'_{pi} = preconsolidation pressure of the ith sublayer
σ'_{v1i} = effective vertical stress at the midheight of the ith sublayer when loading begins
σ'_{v2i} = effective vertical stress at the midheight of the ith sublayer when the soil comes to equilibrium under the load from the structure
log = logarithm to the base 10

The rate at which S_c occurs is usually predicted beforehand using Terzaghi's one-dimensional consolidation theory. Unfortunately, there are a number of assumptions in this theory that invalidate its use in some preloading situations. Details of some of these assumptions and limitations can be found in Duncan (1993). The following assumptions most often invalidate its use in practice:

1. All compression occurs in the direction the load is applied or removed; that is, there is no lateral compression. This assumption is reasonably correct only if the applied stress is relatively uniform; the bearing soil consists of nearly horizontal, uniform layers; and the width of the loaded area is much greater than the thickness of the compressible layer.
2. The strains resulting from primary consolidation are small. In precompression, the soils being treated are frequently soft clays that undergo large strains when precompressed, which invalidates this assumption.
3. The change in void ratio of the consolidating soil is directly proportional to the change in effective stress: $\Delta e \propto \Delta\sigma'_v$. For one-dimensional compression, this is tantamount to assuming that the strains are constant throughout the compressible layer. It has been found from experience that for most saturated, fine-grained soils, $\Delta e \propto \Delta(\log \sigma_v')$, as indicated by the log terms in Eq. 6A.116. So in reality, the strains are greatest in the upper portions of the compressible layer and decrease with depth within the layer.

According to Terzaghi's theory, the primary consolidation settlement at any time (t) after a load has been applied can be estimated from the following equation:

$$S_c(t) = U_v \cdot S_c(t = \infty) \tag{6A.117}$$

where U_v is called the average degree of vertical consolidation within the compressible layer and can be determined from the following equation:

$$U_v = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \cdot e^{-M^2 T_v} \tag{6A.118}$$

where m = integer summation variable starting at zero

$$M = \frac{\pi}{2}(2m + 1)$$

$$T_v = \frac{c_v t}{H_{dp}^2}$$

c_v = coefficient of consolidation (usually obtained from 1-D consolidation tests)

H_{dp} = height of the longest vertical drainage path

In actuality, U_v is the average degree of dissipation of excess pore water pressure within the compressible layer rather than the average degree of primary consolidation settlement. This discrepancy is related to the inconsistency of assumption number 3 discussed above. Duncan (1993) gives an example where the use of Eq. 6A.117 to estimate $S_c(t)$ gives unrealistic results. **Hence, Eq. 6A.117 is not correct and should not be used in practice.**

Terzaghi's theory can be used to give reasonable estimates of $S_c(t)$ in many cases if the following procedure is used:

1. The compressible layer should be subdivided into at least ten sublayers and $S_c(t)$ calculated using Eq. 6A.116.
2. The increase in total stress at the midheight of each sublayer ($\Delta\sigma_{vi}$) should not be assumed to be equal to the applied stress unless one-dimensional compression conditions are closely approximated. Instead, $\Delta\sigma_{vi}$ should be calculated using a method that accounts for the finite width of the loaded area, embedment of the foundation (if any), and layering of the bearing soils.

3. σ'_{v2i} should be replaced in Eq. 6A.116 with $\sigma'_{vi}(t)$, with $\sigma'_{vi}(t)$ calculated as follows:

$$\sigma'_{vi}(t) = \sigma'_{v1i} + U_{zi} \cdot \Delta\sigma'_{vi} \tag{6A.119}$$

where U_{zi} is the consolidation ratio at the midheight of the ith sublayer, which can be calculated as

$$U_z = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \cdot \sin\frac{Mz_i}{H_{dp}} \cdot e^{-M^2 T_v} \tag{6A.120}$$

where z_i is the depth from the top of the clay layer to the midheight of the ith sublayer.

It is sometimes difficult to determine H_{dp} reliably, especially in highly stratified soils containing alternating layers of cohesive and cohesionless soils, such as varved clays. In these types of soils, it must be determined whether or not the cohesionless layers are continuous across the site. This determination requires that extensive boring and sampling be conducted. If the cohesionless layers are continuous, they will act as drainage layers and H_{dp} will be considerably reduced. Otherwise their effect on H_{dp} will be minimal unless vertical drains are installed.

6A.6.3 Precompression with Additional Surcharge

The precompression process can be achieved much quicker in saturated clays if additional surcharge is used above that required for simple precompression. This method is especially attractive when the structure to be built on the site is a permanent embankment, such as for highway and railroad applications. Three possible ways to construct the additional surcharge are illustrated in Fig. 6A.189.

The critical factor in successful application of this technique is determining the appropriate time when the surcharge should be removed (t_{sr}). Proper determination of t_{sr} depends on a thorough understanding how the pore pressures and effective stresses change in the clay layer as the permanent and surcharge embankments are constructed, while the additional surcharge is in place, and after the surcharge is removed.

Total vertical stress (σ'_v), pore water pressure (u), and effective vertical stress (σ'_v) profiles in the clay layer at various times are illustrated in Fig. 6A.190 for the idealized case of perfect one-dimensional compression of a normally consolidated clay layer, instantaneous construction and removal times, the top of the clay layer and the groundwater table both coinciding with the ground surface, and double drainage. The applied stress at the ground surface from the permanent embankment is designated Δp, and the corresponding stress from the additional surcharge is designated Δs. Just before construction of the permanent and surcharge embankments (t_0^-), σ'_v, u, and σ'_v are zero at the top of the clay layer and increase linearly with depth (z) in proportion to the saturated unit weight of the clay (γ_{sat}), the unit weight of the water (γ_w), and the effective unit weight of the clay (γ'), respectively. Just after the permanent and surcharge embankments are constructed instantaneously (t_0^+), the combined applied stress ($\Delta p + \Delta s$) is carried entirely by the pore water as excess pressure. Therefore, at time t_0^+ the effective vertical stresses in the clay layer have not changed from their initial values and no primary consolidation settlement has occurred. As time elapses after construction, the excess pore water pressures dissipate. At the drainage boundaries, the excess pore pressures dissipate completely in a short time. The rate of dissipation decreases with distance from the boundaries and is slowest at the elevation farthest from the drainage boundaries (midheight of the clay layer in this case). Since the total stresses have not changed once construction was completed, the effective stresses increase by the same amounts that the excess pore water pressures decrease ($\Delta\sigma'_v = -\Delta u$). At time t_1, corresponding to some time after construction, the stress profiles take the shapes shown in Fig. 6A.190. Profiles at later times t_2 and t_3 are also shown. As time elapses with the additional surcharge, the pore water pressures are decreasing toward their equilibrium condition (same

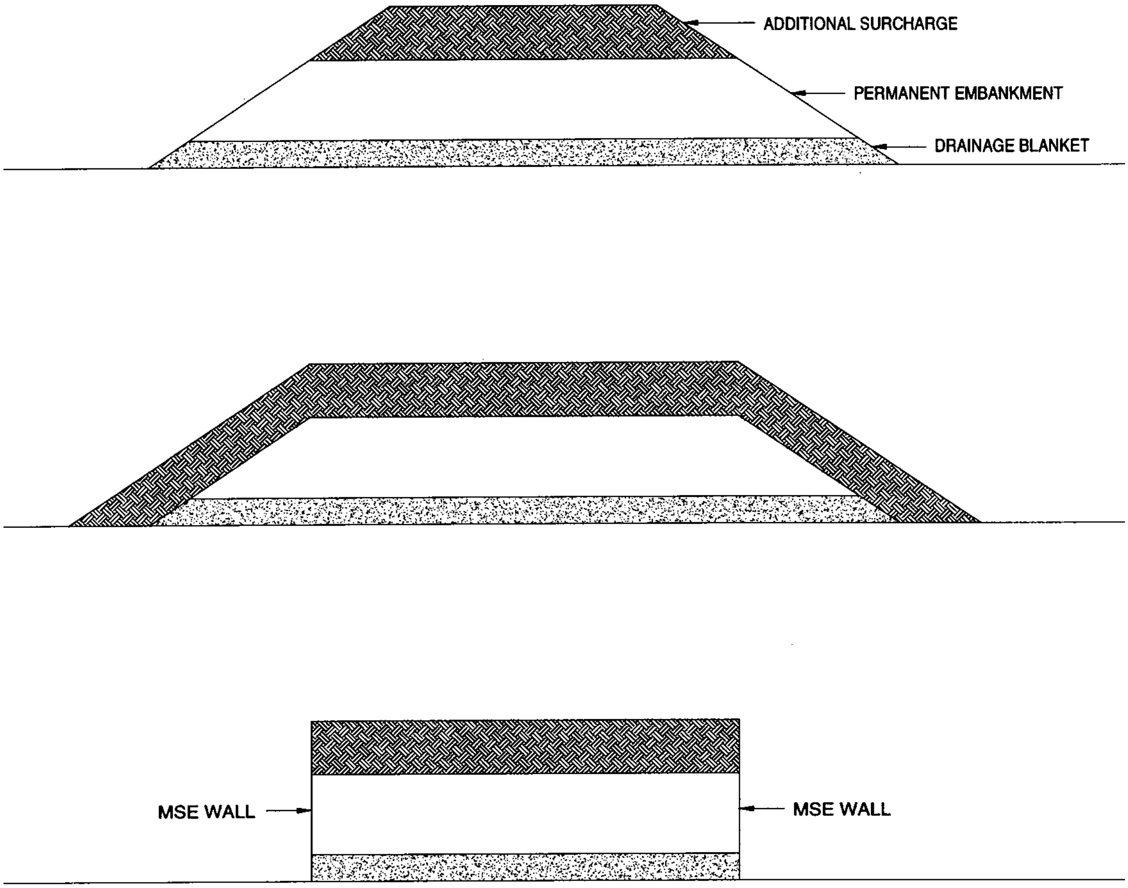


FIGURE 6A.189 Three possible configurations of permanent embankment with additional surcharge.

as initial condition assuming the groundwater table stays at the same elevation), while the effective vertical stresses are increasing toward their equilibrium condition ($\sigma'_{v0} + \Delta p + \Delta s$).

At the instant the additional surcharge is removed, σ_v and u at all depths decrease by $-\Delta s$, while σ'_v at all depths does not change. The equilibrium condition for u remains the same, while the equilibrium condition for σ'_v decreases by Δs and is now equal to $\sigma'_{v0} + \Delta p$. If the surcharge is removed at time t_1, the pore water and effective stress profiles are as shown in Fig. 6A.191. t_1^- denotes just before the surcharge is removed, t_1^+ denotes just after the surcharge is removed, and t_∞ denotes the equilibrium condition under the permanent embankment. The central portion of the clay layer is *normally consolidated–underconsolidated* (NC–UC, normally consolidated and still compressing but now only under the permanent embankment load), while the top and bottom portions of the clay layer are *overconsolidated–underconsolidated* (OC–UC, precompressed and now rebounding owing to removal of the additional surcharge). The excess pore water pressures are indicated by the hatched regions. The excess pore water pressures in the upper and lower portions are negative, and in the middle portion they are positive. At equilibrium under the permanent embankment, the net vertical movement of the clay layer from t_{sr} to t_∞ may be either settlement (downward) or heave (upward) depending on the magnitude of the compression or rebound that occurs within each portion of

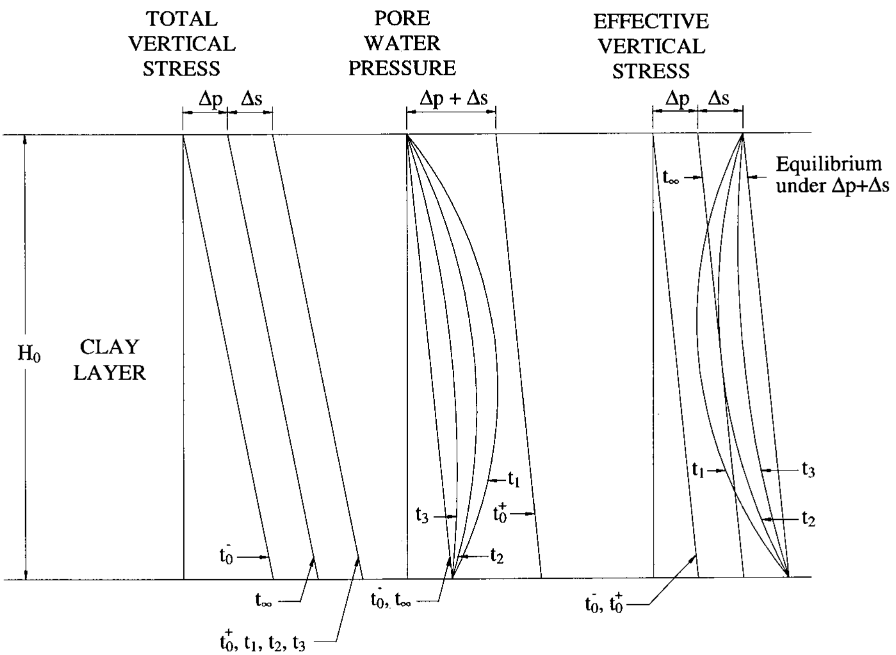


FIGURE 6A.190 Stress profiles within a doubly drained clay layer before, during, and after precompression.

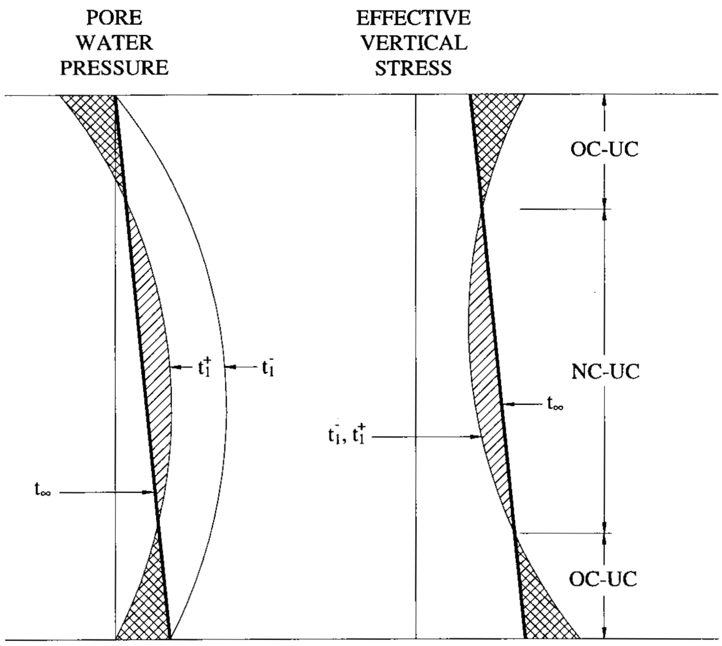


FIGURE 6A.191 Pore water and effective vertical stress profiles within a doubly drained clay layer if additional surcharge is removed at time t_1.

the clay layer. It is theoretically possible to time the surcharge removal such that the net vertical movement of the clay layer at equilibrium is zero, but this is difficult to do in practice. Furthermore, the rates at which virgin compression and rebound occur are different. Virgin compression is typically much slower than either recompression or rebound. Hence, there would be net vertical movement of the clay layer between t_{sr} and t_∞ even if the net vertical movement at equilibrium is zero.

If the additional surcharge is removed at time t_2, the pore water and effective stress profiles are as shown in Fig. 6A.192. The entire clay layer is precompressed (OC–UC), except at its midheight, which is the location where the water has the longest vertical distance to drain in this case. This critical location is in equilibrium (primary consolidation is complete). If the clay layer is singly drained (drainage at the top only), the critical location will be at the bottom of the layer. When controlling the magnitude of primary consolidation settlement is the primary goal of the precompression process, this method of determining when the surcharge should be removed is preferred by most engineers, and a small amount of primary consolidation heave will occur after the surcharge is removed. The procedure used to determine t_{sr} for zero excess pore water pressure at the critical location is as follows:

1. Calculate $\Delta\sigma_v$ for the load from permanent embankment only (Δp) at the critical depth (z_{cr}) using an appropriate method. If true one-dimensional compression conditions exists, $\Delta\sigma_v(z_{cr}, \Delta p) = \Delta p$.
2. Calculate $\Delta\sigma_v$ for the load from permanent embankment plus the additional surcharge ($\Delta p + \Delta s$) at the critical depth (z_{cr}) using an appropriate method. If true one-dimensional compression conditions exists, $\Delta\sigma_v(z_{cr}, \Delta p + \Delta s) = \Delta p + \Delta s$.

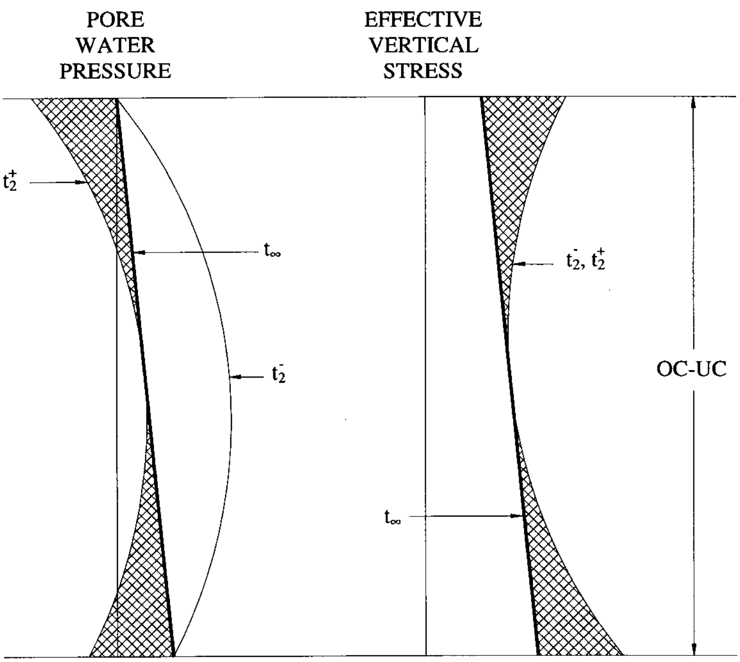


FIGURE 6A.192 Pore water and effective vertical stress profiles within a doubly drained clay layer if additional surcharge is removed at time t_2.

3. Set the consolidation ratio at the critical depth ($U_{z\text{-}cr}$) equal to the value calculated in step 1 divided by the value calculated in step 2. That is,

$$U_{z\text{-}cr} = \frac{\Delta\sigma_v(z_{cr}, \Delta p)}{\Delta\sigma_v(z_{cr}, \Delta p + \Delta s)} \tag{6A.121}$$

4. Assume a value of $T_{v\text{-}sr}$ and solve for $U_{z\text{-}cr}$ using Eq. 6A.120. If the values of $U_{z\text{-}cr}$ calculated from Eqs. 6A.120 and 6A.121 are not the same, assume other values of $T_{v\text{-}sr}$ until the two values are the same. This is the correct value of $T_{v\text{-}sr}$.
5. Calculate t_{sr} from the following equation:

$$t_{sr} = \frac{T_{v\text{-}sr} \cdot H_{dp}^2}{c_v} \tag{6A.122}$$

Because the theory is based on many simplifying assumptions that may deviate significantly from reality, and there are errors associated with determining the input parameters for the soil, the value of t_{sr} determined using this method should be considered an approximation only. In practice, the pore water pressures at various depths and locations beneath the preloaded area should be monitored at regular intervals to ascertain when the additional surcharge should be removed.

If the additional surcharge is not removed until time t_3, the preloaded soil will be precompressed beyond the level needed to ensure that primary consolidation settlement will be controlled. This will result in additional primary heave above that which will occur if $t_{sr} = t_2$. In some instances, this may be desirable to reduce post-preloading settlements resulting in soils such as highly organic clays and peats where secondary compression is the dominant mechanism of settlement. In soils where secondary compression is to be controlled, the equation for $U_{z\text{-}cr}$ that is analogous to Eq. 6A.121 is as follows:

$$U_{z\text{-}cr} = \frac{S_c(\Delta p, t_{pr}) + C_{\alpha\varepsilon} H_p \log(t_d/t_{pr})}{S_c(\Delta p + \Delta s, t_\infty)} \tag{6A.6.123}$$

Once $U_{z\text{-}cr}$ is calculated, t_{sr} can be determined in the same manner described above for controlling primary consolidation settlement.

6A.6.4 Precompression with Vertical Drains

Vertical drains can be used in many cases to accelerate the rate of primary consolidation settlement within the preloaded zone, whether or not additional surcharge is used. However, they are ineffective in soils such as highly organic clays and peats whose settlement behavior is dominated by secondary compression, owing to the small amount of water squeezed from these soils during second compression. Vertical drains allow drainage to take place horizontally in addition to vertically, as illustrated in Fig. 6A.193. In typical applications where vertical drains are used, most of the drainage will occur horizontally rather than vertically for two reasons: (a) the horizontal drainage path is generally shorter than the vertical drainage path for most of the compressible layer, and (b) most clay deposits have greater permeability in the horizontal direction than in the vertical direction. Representative values of horizontal to vertical permeability (k_h/k_v) for soft clays (undrained shear strength less than 1 ksf = 48 kPa) are given in Table 6A.18. Actual values of k_h/k_v for any given soil should be verified by the designer.

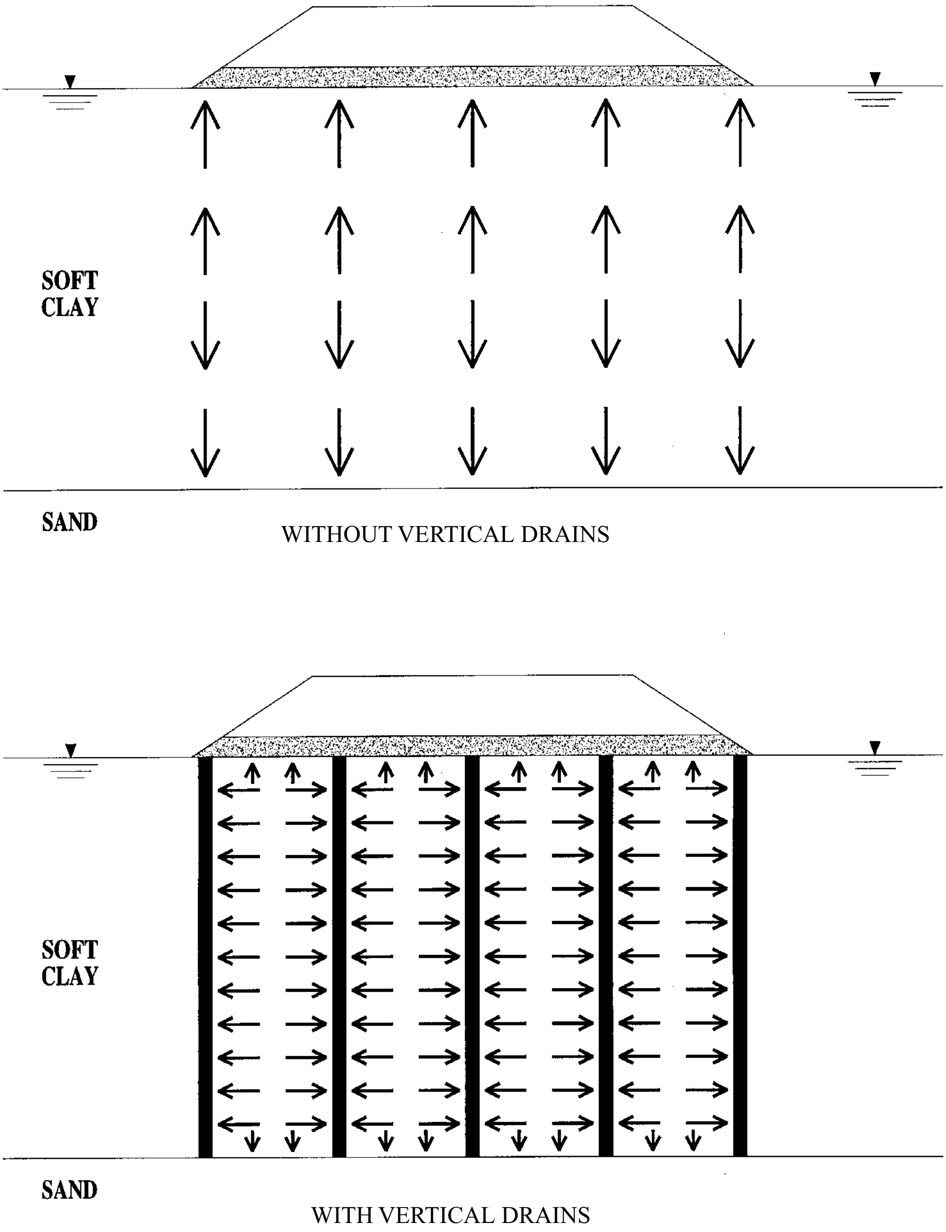


FIGURE 6A.193 Comparison of drainage during precompression without and with vertical drains.

TABLE 6A.18 Representative Values of k_h/k_v for Soft Clays (from Rixner et al. 1986)

Description	k_h/k_v
No evidence of layering (partially dried clay has completely uniform appearance)	1.0 to 1.4
No or only slightly developed macrofabric (e.g., sedimentary clays with discontinuous lenses and layers of more permeable soil)	1.0 to 1.5
Slight layering (e.g., sedimentary clays with occasional silt dustings to random silty lenses	2 to 5
Fairly well to well developed macrofabric (e.g., sedimentary clays with discontinuous lenses and layers of more permeable material	2 to 4
Varved clays in Northeastern United States	5 to 15
Varved clays and other deposits containing embeded and more or less continuous permeable layers	3 to 15

6A.6.4.1 Types of Vertical Drains

Sand drains were used to accelerate primary consolidation in early precompression applications in the United States (Rixner et al. 1986). A U.S. patent for a sand drain system was granted in 1926. Since that time, sand drains have been used successfully worldwide on a large number of precompression projects. Gravel drains have also been used successfully. Details on sand and gravel drains can be found in Section 6A.5.1.4.

Prefabricated sand drains have also been used. These drains consist of sand prepacked in a fabric sock.

In the late 1930s, the first prefabricated vertical (PV) drain was developed in Sweden and patented (Kjellman 1948). This drain, called a *cardboard wick*, was band-shaped (rectangular cross-section) with a width of about 4 in (100 mm) and a thickness of about 3 mm (0.13 in). It came in continuous strips about 1,300 ft (400 m) long and was rolled on a drum.

Most modern PV drains have many of the same general characteristics as the cardboard wick but are made of different materials. They are typically band-shaped with widths of about 4 in (100 mm) and thicknesses varying from about 0.08 to 0.25 in (2 to 6 mm). The main components typically consist of a plastic core surrounded by a synthetic geotextile jacket, as shown in Fig. 6A.194. Configuration of four types of PV drains are illustrated in Fig. 6A.195. The primary functions of the core and jacket are summarized as follows (Ladd 1986).

Jacket

1. Separate the flow channels of the core from the surrounding fine-grained soils
2. Act as a filter to allow easy passage of water into the core while preventing the intrusion of soil
3. Prevent closure of internal drain flow paths under lateral soil pressure

Core

1. Provide internal flow paths along the drain
2. Maintain drain configuration and shape
3. Provide resistance to buckling and longitudinal stretching of the drain

The major advantages of PV drains compared to sand drains are simplicity and speed of installation, less disturbance to the soil mass, and usually lower cost. Because of these advantages, the use of PV drains has largely replaced sand drains for most preloading applications.

PV drains are usually installed using equipment similar to that shown schematically in Fig. 6A.196. The drain is enclosed within a mandrel and attached to an anchor plate at the bottom of the

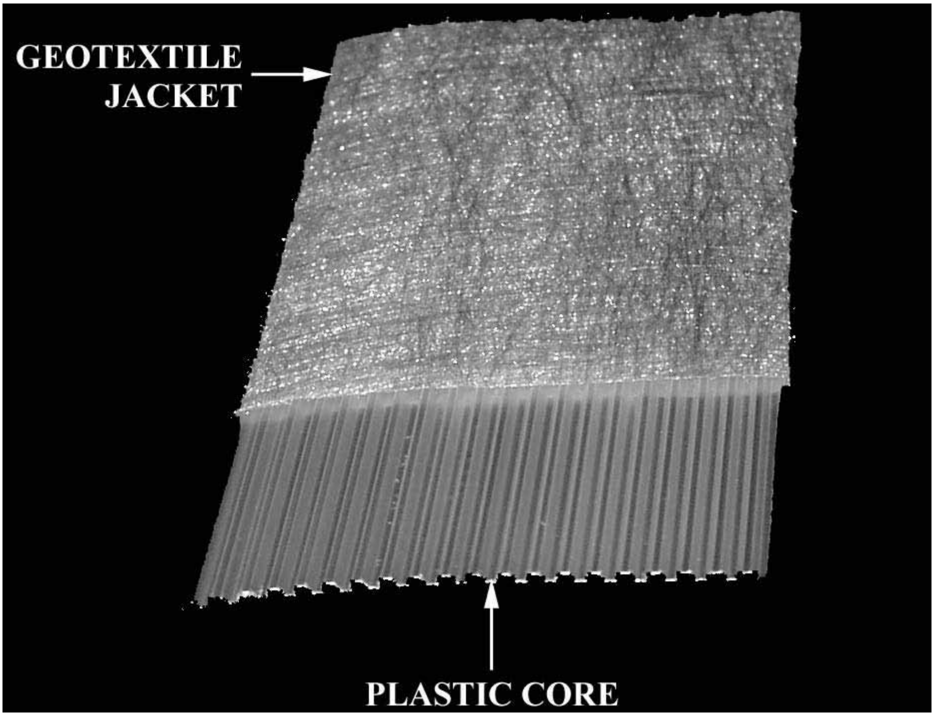


FIGURE 6A.194 Typical band-shaped PV drain in common use today.

mandrel. The mandrel is then pushed, driven, or vibrated into the soil to the desired depth. The anchor plate holds the drain in the soil while the mandrel is extracted. The drain is then cut off, a new anchor plate installed, and the process repeated at another location. PV drains are typically installed in a triangular pattern at a spacing ranging from about 3 to 30 ft (1 to 10 m) to depths up to about 160 ft (50 m).

Band-shaped PV drains are more commonly referred to as *wick drains* in the United States. However, this term is a misnomer because "wick" suggests that water is absorbed by and rises within the drain owing to capillary action. In fact, the excess pore water pressure generated by the preload increases the hydraulic head and causes the water to flow into the PV drain and then either upward or downward to a drainage layer. Thus, use of the term wick drain is not recommended, but because its use has been prevalent for a long period of time within the construction and civil engineering industries, usage of the term may well continue. Koerner (1994) reported that his attempts in the United States to change usage from wick drain to strip drain were unsuccessful.

Detailed information regarding using, designing, and specifying PV drains can be found in a number of sources. Good references on this topic include Bergado et al. (1996), Holtz et al. (1991), and Rixner et al. (1986).

6A.6.4.2 Radial and Combined Drainage

Barron (1948) provided the first comprehensive theory for vertical consolidation resulting from axisymmetric radial drainage into a cylindrical drain well. Barron's theory was based on the same simplifying assumptions as Terzaghi's one-dimensional consolidation theory, and thus suffers from the same limitations discussed in Section 6A.6.2.

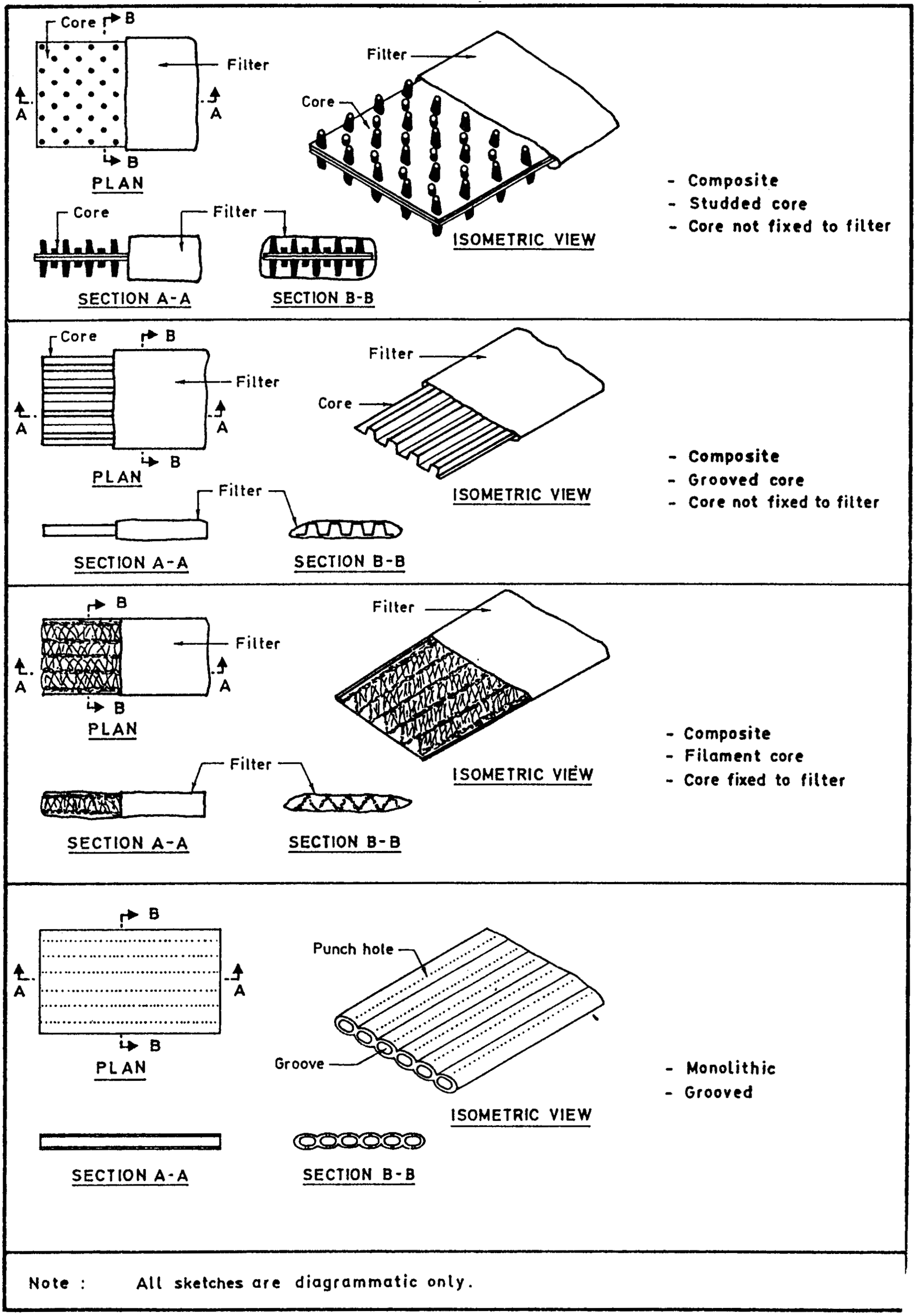


FIGURE 6A.195 Configuration of different types of PV drains (from Bergado et al., 1996).

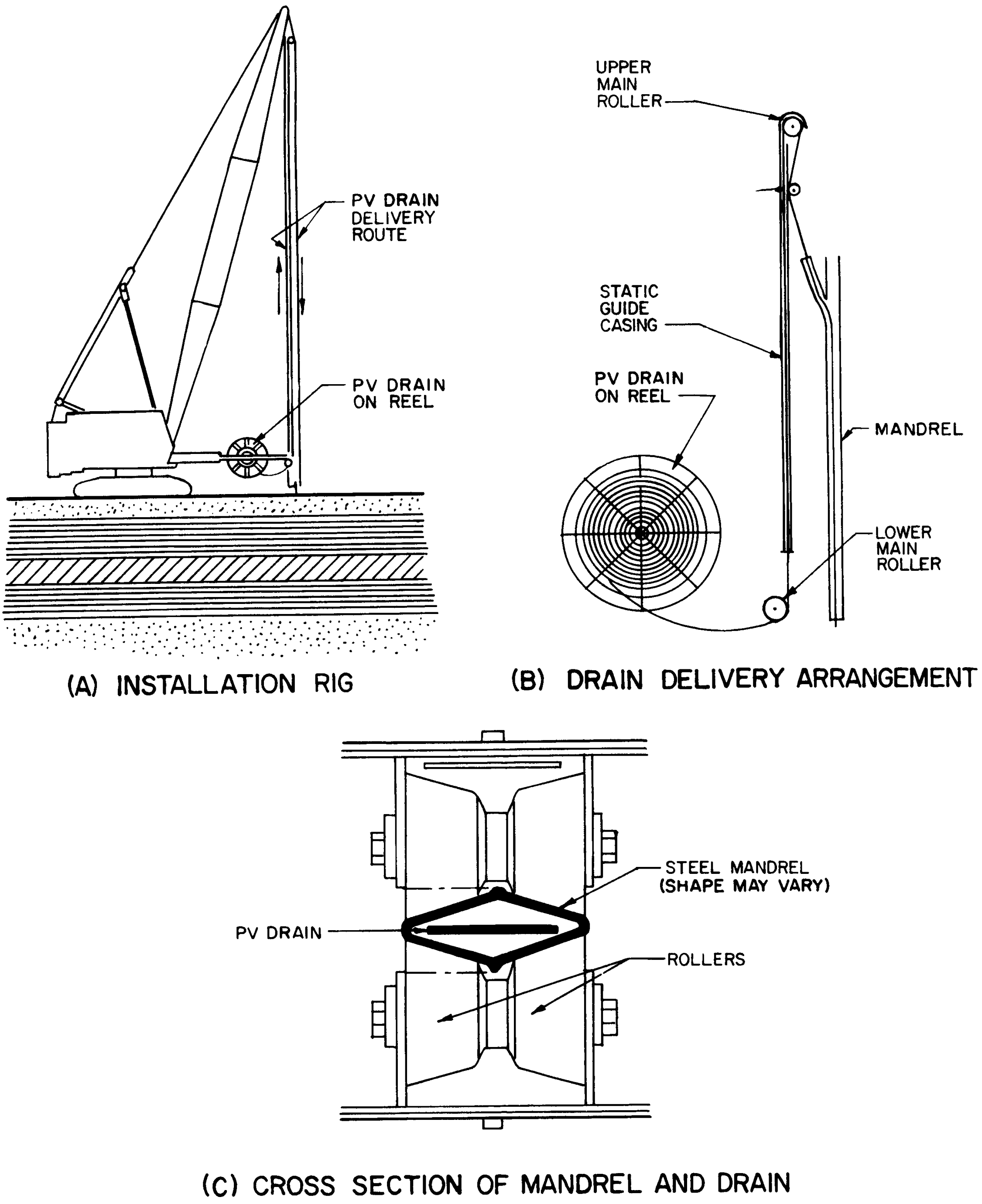


FIGURE 6A.196 Schematic illustration of typical equipment used to install PV drains (from Rixner et al., 1986).

Barron developed theories for two cases of vertical strain: (a) free strain, corresponding to a flexible foundation such as a soil embankment, and (b) equal strain, corresponding to a rigid foundation such as a thick, reinforced concrete footing. Although the free strain theory is more applicable to most preloading projects, Barron found that the two theories generally give similar results. Since the solution to the equal-strain theory is simpler to use, it is preferred in practice. Furthermore, settlement observations in the field strongly support the assumption of equal vertical strains (Hansbo 1979).

For the idealized case of no soil disturbance and no resistance to flow within the drain, the following differential equation governs consolidation for equal vertical strains and radial flow only:

$$\frac{\partial u_e}{\partial t} = c_h\left(\frac{1}{r} \cdot \frac{\partial u_e}{\partial r} + \frac{\partial^2 u_e}{\partial r^2}\right) \tag{6A.124}$$

where u_e = average excess pore water pressure at any point and time
t = time after instantaneous application of the preload
r = radial distance from the center of the drain
c_h = horizontal coefficient of consolidation

The solution to Eq. 6A.124 is as follows:

$$u_e = \frac{u_i}{r_c^2 \cdot \mu}\left[r_c^2 \ln\left(\frac{r}{r_d}\right) - \frac{r^2 - r_d^2}{2}\right]e^{\lambda} \tag{6A.125}$$

where u_i = initial excess pore water pressure caused by the preload
r_c = radius of equivalent soil cylinder from which radial drainage occurs
r_d = radius of a sand or gravel drain, or equivalent radius of a PV drain (see subsequent discussion)

$$\mu = \frac{n^2}{n^2 - 1} \cdot \ln(n) - \frac{3n^2 - 1}{4n^2}$$

$n = r_c/r_d = d_c/d_d$
$d_c = 2r_c = 1.05s$ for triangular pattern of drains $= 1.13s$ for square pattern of drains (see Fig. 6A.163), where s is the center-to-center spacing of the drains
$d_d = 2r_d$
$\lambda = -8T_r/F(n)$
$T_r = c_h t/d_c^2$ = dimensionless time factor for radial drainage
ln = natural logarithm = logarithm to the base e

From Eq. 6A.125, it can be seen that at any time t, u_e increases with radial distance (r) from the drain, as would be expected.

The average radial consolidation ratio (U_r) within the zone of drainage for each vertical drain is given by the following equation:

$$U_r = 1 - \frac{u_{e\text{-}av}}{u_i} = 1 - e^{\lambda} \tag{6A.126}$$

where $u_{e\text{-}av}$ = average excess pore water pressure with the drainage zone at any time.

It is important to note that U_r is the same at all depths within the clay layer (z) for the simplifying assumptions of the theory.

A number of methods have been used to calculate the equivalent diameter of a band-shaped PV drain. Of these methods, two are used predominantly in practice. The first method was proposed by Kjellman (1948) and is based on the assumption that the drainage capacity of the drain depends on the area of contact with the soil. With this assumption, the perimeter of a PV drain with a thickness of a and a width b is set equal to the perimeter of an equivalent circular with a diameter of d_d. Solving for the equivalent diameter d_d of the PV drain gives

$$d_d = \frac{2(a + b)}{\pi} \tag{6A.127a}$$

Finite element analyses and other studies have indicated that the following equation provides a better estimate of d_d (Rixner et al. 1986):

$$d_d = \frac{a + b}{2} \tag{6A.127b}$$

Hansbo (1987) also supported the use of this second equation. Eq. 6A.127b is the more conservative of the two equations and is recommended for use in practice.

Two important factors can slow the rate of water drainage from the soil into the drain: (a) disturbance of the fabric of the soil adjacent to the drain during installation of the drain, also known as *smear*; and (b) resistance to flow of water within the drain itself. A schematic diagram illustrating both effects is depicted in Fig. 6A.197. Hansbo et al. (1981) proposed the following equation for the average degree of radial consolidation that accounts for smear and resistance to flow ($U_{r\text{-}sr}$):

$$U_{r\text{-}sr} = 1 - e^{-8T_r/\mu_{sr}} \tag{6A.128}$$

where $\mu_{sr} = \ln\frac{n}{m} + \frac{k_h}{k_s}\ln m - \frac{3}{4} + \pi z_d(2L_{dp} - z_d)\frac{k_h}{q_d}$

$m = d_s/d_d$
d_s = diameter to the outer perimeter of the smear zone
k_s = coefficient of horizontal permeability for the smeared zone
z_d = vertical distance from the closest end of the drain where drainage occurs
L_{dp} = length of the longest drainage path along the vertical drain = L for drainage at one end only = $L/2$ for drainage at both ends
q_d = discharge capacity of the drain under a hydraulic gradient of one = $k_d A_d$
k_d = axial coefficient of permeability for the drain
A_d = cross-sectional area of the drain

It can be seen in Eq. 6A.128 that when the resistance of flow within the drain is finite, the value of average radial consolidation varies with depth within the clay layer.

The size of the smear zone and the degree of disturbance depend significantly on the type of drain and the method of installation. For sand and gravel drains, displacement methods of installation result in greater disturbance than nondisplacement methods. The size of the smear zone is also a function of the diameter of the drain. For PV drains, the disturbance depends mostly on the size and shape of the mandrel and the shoe, the soil macrofabric, and the installation procedure. Typical shapes of mandrels and shoes are shown in Figs. 6A.198 and 6A199. In general, it appears that the amount of disturbance increases with increasing cross-sectional area of the mandrel (Bergado et al. 1991). Therefore, the mandrel cross-sectional area should be the smallest possible that allows clearance for the PV drain and gives adequate stiffness for installation. The shape of the mandrel tip and anchor should be as tapered as possible to reduce disturbance. The outer diameter of the smear zone (d_s) can be estimated from the following equation (Jamiolkowski and Lancellotta 1981, Hansbo 1987):

$$d_s = (2 \text{ to } 3)d_m \tag{6A.129}$$

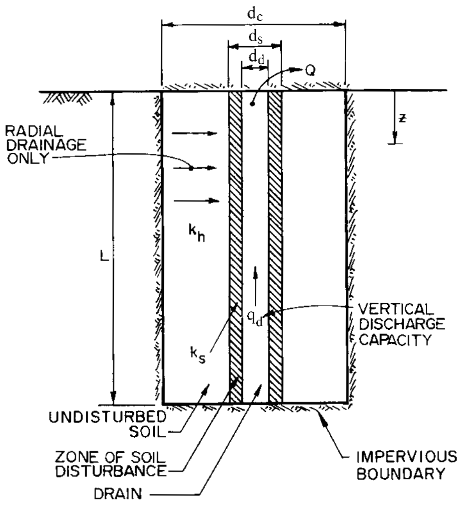


FIGURE 6A.197 Schematic of vertical drain with drain resistance and soil disturbance (from Rixner et al., 1986).

where d_m is the diameter of a circle with an area equal to the cross-sectional area of the mandrel. The effect of disturbance on the coefficient of horizontal permeability of the soil is less well known. It appears that values of k_h/k_s range from about 1 to 10 (Rixner et al. 1986, Holtz et al. 1991). It should be recognized, however, that the disturbance is not uniform throughout the smeared zone; rather, it transitions from greater disturbance near the drain to less disturbance near the outer perimeter of the smear zone.

If the discharge capacity of the drain (q_d) is reached during consolidation, the overall consolidation process will be slowed. The discharge capacity of sand and gravel drains depends on the permeability of the sand or gravel. Clean sand or gravel should be used in these drains. The discharge capacity of PV drains varies considerably depending on the make of the drain and decreases with increasing lateral pressure, as shown in Fig. 6A.200. Therefore, long drains are more susceptible to reduction in consolidation rate owing to drain resistance. Accurate measurement of q_d requires sophisticated laboratory or field testing and is time consuming. It is also almost always less significant than spacing of the drains and disturbance factors. Therefore, q_d is usually estimated rather than measured. Although values of q_d are generally reported by PV drain manufacturers, the test procedures are not standardized. In general, q_d can be conservatively assumed to be 3500 ft^3/yr (100 m^3/yr) (Rixner et al. 1986).

c_h is usually estimated from the following equation:

$$c_h = \frac{k_h}{k_v} c_v \tag{6A.130}$$

Values of k_h/k_v can be estimated (see Table 6A.18) or determined from laboratory or field tests. Alternately, c_h can be determined from the following equation:

$$c_h = \frac{k_h}{m_v \gamma_w} \tag{6A.131}$$

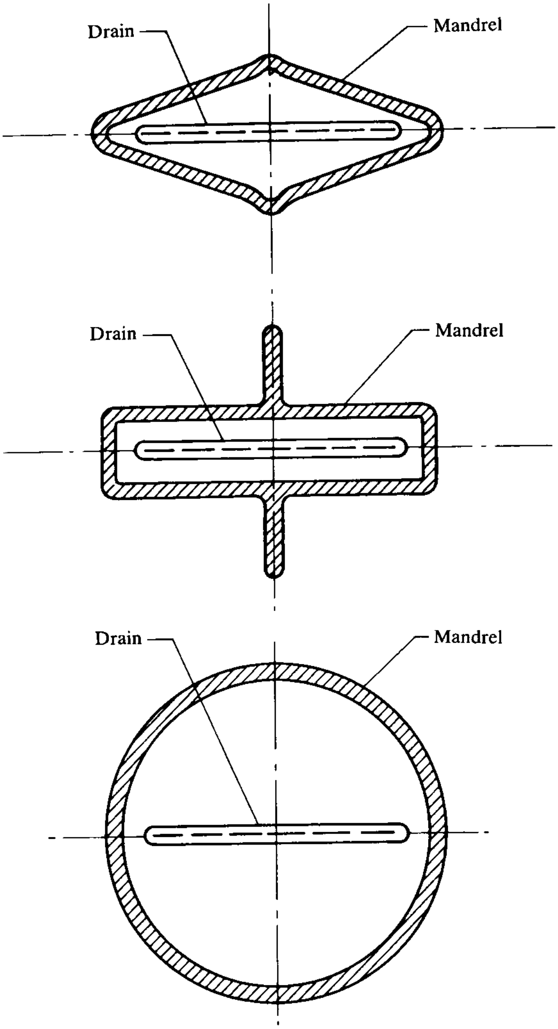


FIGURE 6A.198 Typical shapes of mandrels for band-shaped PV drains (from Holtz et al., 1991).

where m_v is the one-dimensional coefficient of volume change and γ_w is the unit weight of water. m_v is generally obtained from laboratory one-dimensional consolidated tests and k_h is usually determined by small-scale pumping tests in piezometers or from self-boring permeameters. A number of other methods can be used to determine c_h. Some of these methods are summarized in Ladd and Foott (1977) and Rixner et al. (1986).

There is still considerable uncertainty about the influence of smear and drain resistance on the effectiveness of vertical drains. In addition, it is frequently difficult to determine accurate values of q_d, d_s, and k_s to use in Eq. 6A.128. Hence, many engineers prefer to use reduced values of c_h,

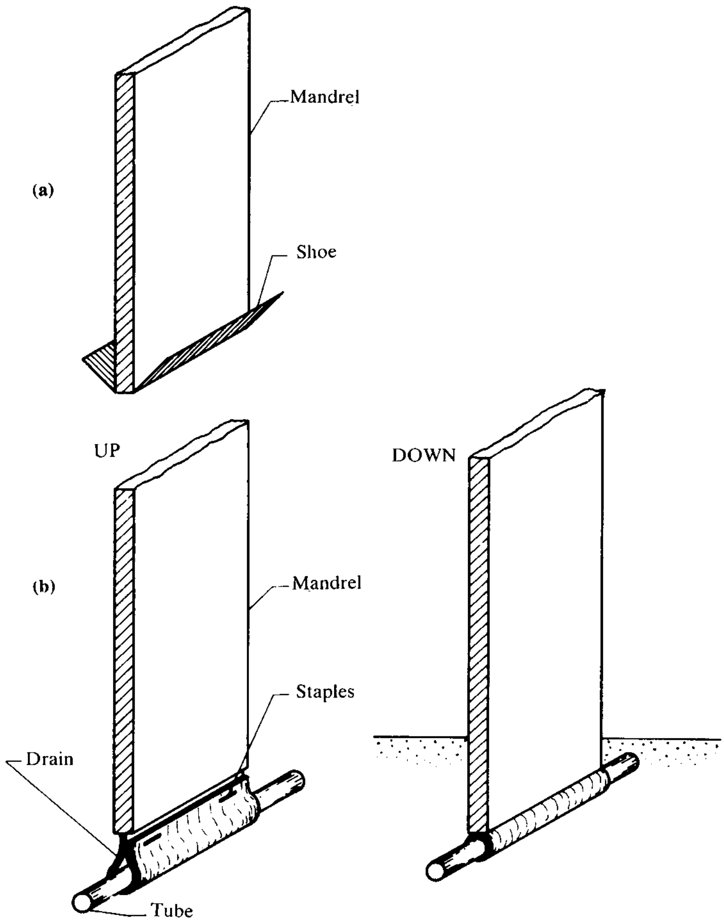


FIGURE 6A.199 Typical detachable shoes used with mandrels for band-shaped PV drains (from Holtz et al., 1991).

d_d, or both in the simpler Eq. 6A.126 to account for the possible negative effects of these phenomena.

Buckling and kinking of PV drains has been observed in the laboratory and the field. It is very difficult to predict when and where these phenomena will occur, and if they do occur, how they will affect the discharge capacity of the drains.

In a typical design situation, the engineer must determine the required spacing of vertical drains to achieve a specified degree of consolidation (usually 90 or 95%) within a certain amount of time usually dictated by various aspects of the construction process. If vertical consolidation is ignored, which is sometimes done to be conservative, either Eq. 6A.126 or 6A.128 can be used in a trial and error process to find the appropriate spacing. In this iterative process, a spacing is assumed for the pattern of the wick drains to be used, and the corresponding value of d_c is calculated. Then, using Eq. 6A.126 or 6A.128, a corresponding value of U_r is calculated. If the calculated value of U_r is not equal to the desired value of U_r, the process is repeated until the calculated and desired values are the same.

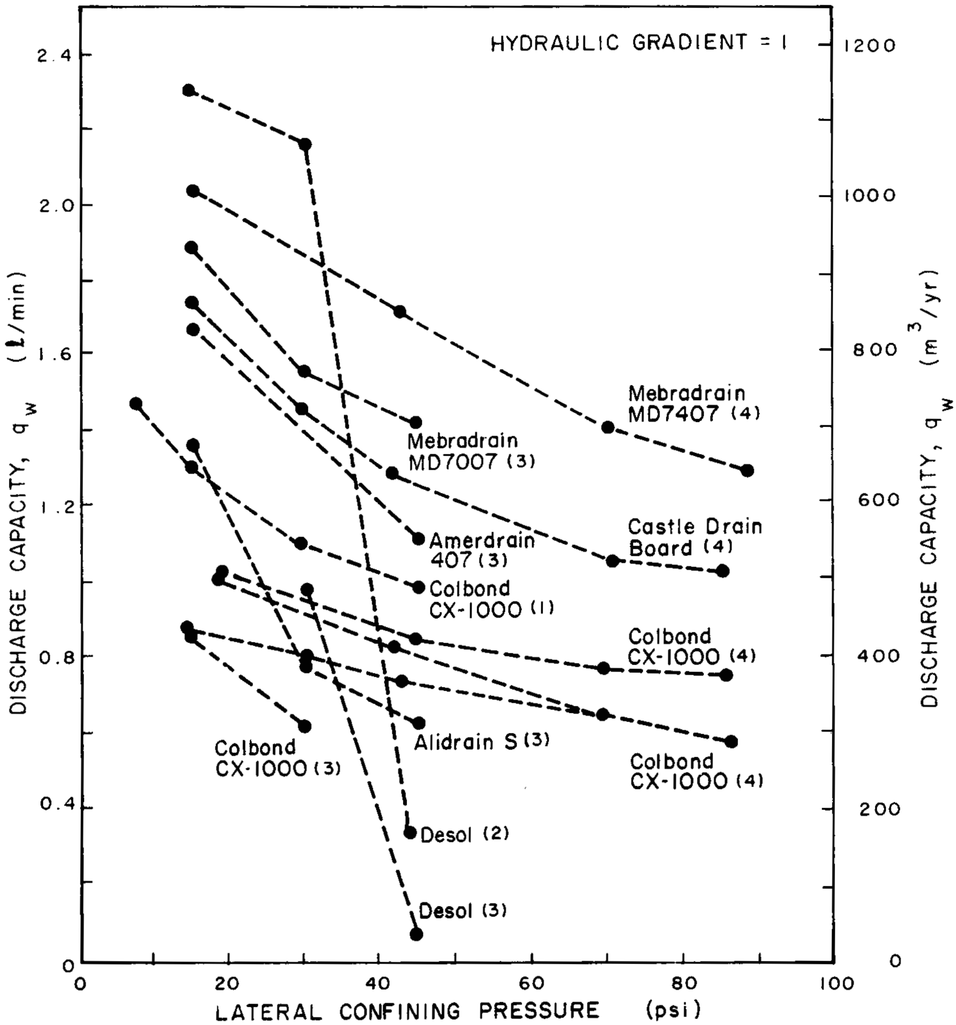


FIGURE 6A.200 Effect of confining pressure on vertical discharge capacity of some typical PV drains (from Rixner et al. 1986). *Note:* Data from sources other than (3) not verified. Test methods vary. Data Sources: (1) Colbond promotional literature; (2) Desol internal report; (3) Reference 8; (4) Jamiolkowski and Lancellotta, unpublished.

If both vertical and radial drainage are considered, the combined degree of consolidation (U_{vr}) can be found from the following equation (Carillo 1942):

$$U_{vr} = 1 - (1 - U_v)(1 - U_r) \tag{6A.6.132a}$$

A trial and error process is also required to determined the required spacing of the vertical drains for combined drainage. First, U_v is calculated for the desired time (t) from Eq. 6A.118. Next, the required value of U_r to provide the desired value of U_{vr} is determined by rearranging Eq. 6A.6.132a to solve for U_r:

$$U_r = 1 - \frac{1 - U_{vr}}{1 - U_v} \tag{6A.6.132b}$$

A trial and error process similar to that described above for radial drainage only is then used to determine the spacing of drains that will give the value of U_r obtained from Eq. 6A.132b in time t.

It is important to note that the values of consolidation ratio calculated for U_v, U_r, and U_{vr} are not the portion of the ultimate primary consolidation settlement that will occur in time t. Instead, these values are the average degree of dissipation of excess pore pressure within the clay layer.

6A.6.5 Strength Considerations

Since precompression is typically conducted on soft, saturated clays, and the preloads can be large, strength is an important consideration in the process. In the short-term, large positive excess pore water pressures can be generated that may lead to bearing capacity failure or large deformations near the perimeter of the preloaded area. As the excess pore water pressures dissipate and primary consolidation occurs, the soil becomes stronger. Therefore, potential strength problems can be alleviated by preloading in stages, where the a portion of the total preload is applied and allowed to consolidate some prior to the application of additional preload. Vertical drains can also be used to accelerate the consolidation process, and hence the strengthening process. Sand drains, gravel drains, or other granular columns can also provide an increase in strength and reduction in vertical and lateral deformations (see Section 6A.5).

6A.6.6 Monitoring Field Performance

The performance of the precompression in the field should always be monitored. The deformations of the soil (settlement and horizontal movement) should be determined, as well as the pore water pressures in the soil if it is saturated and fine-grained. Settlement is usually monitored using settlement plates, gauges, and leveling points. The horizontal movements are monitored using alignment stakes and vertical inclinometers. Pore water pressures are monitored using piezometers. Details on the instrumentation and monitoring of the preloading process can be found in many references (e.g., Hanna 1973 and Mitchell 1981). Variations in performance can be determined and adjustments made, as necessary, to the time required to leave the surcharge in place, the magnitude of the preload, or any other aspects of the preloading process.

*6A.7 CHEMICAL STABILIZATION**

The mechanical properties of a soil can be improved by chemical stabilization, which involves the injection or mixing of a chemical agent or agents into the soil. Chemical stabilization can be used on in situ soils or on borrow or excavated materials prior to compaction. The two most common types of chemical reactions used to stabilize soils are cation or base exchange reactions with clay particles and cementitious or pozzolanic reactions. Commonly used chemical agents for cementitious and pozzolanic stabilization include Portland cement, lime, fly ash, sodium silicate, polyacrylamides, and bituminous emulsions. Chemicals used for base exchange include inorganic cations

*Coauthored by Robert Wade Brown.

such as Ca^{2+} and organic products such as polyquarternaryamines. Engineering properties of a soil that may be intentionally altered during chemical stabilization, or that may occur as a by-product of the stabilization process, include the following:

1. Strength
2. Compressibility
3. Permeability
4. Expansiveness
5. Collapsibility
6. Potential for liquefaction
7. Erodability

Because of environmental concerns and regulations, an engineer must consider the potentially adverse impact that chemical stabilization may have on the soil and local groundwater. Before any chemical stabilization is conducted, the appropriate local, state, and federal environmental regulatory agencies should be consulted to ensure that the chemicals to be used are not prohibited.

6A.7.1 Chemicals and Reactions

This section discusses chemicals that are commonly used to stabilize soils, along with the reactions that may occur and the manner in which the chemicals stabilize the soil.

6A.7.1.1 Cation and Base Exchange with Clay Particles

Clay Mineralogy, Clay-Water Interaction, and Cation Exchange. The near-surface clay minerals are basically composed of various hydrated oxides of silicon, aluminum, iron, and to a lesser extent potassium, sodium, calcium, and magnesium (Brown, 1992). Since clays are produced from the weathering of certain rocks, the particular origin of a clay determines its nature and properties. 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 and their specific physical and chemical behavior.

Owing to their loose crystalline structure, most clays exhibit the properties of ion exchange and moisture absorption and adsorption. Among the more common clays, the activity of the clays—defined in general terms as the capacity for electrochemical interaction with water and chemicals in the soil—decreases in the following order: montmorillonite, illite, attapulgite, chlorite, and kaolinite. A specific clay may adsorb and absorb water to varying degrees—from a single layer to six or more layers—depending on such factors as its structural lattice, presence of exchange ions, temperature, and environment. The moisture taken on by a clay 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., swelling and shrinkage) problems encountered with expansive soils. If this movement is to be controlled, each of these forms of moisture must be controlled and stabilized.

Interstitial water and surface adsorbed water both occur within the soil mass external to individual soil grains and are generally accepted as capillary moisture. The interstitial water is held by interfacial tension, whereas the surface adsorbed water is held by molecular attraction between the clay particles and the dipolar water molecules. Variations in this combined moisture are believed to account for the principal volume change potential of the soil (see also Sec. 1).

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: crystalline spacing, elements present in the crystalline structure, and presence of exchange ions. For example, bentonite (sodium montmorillonite) will swell approximately thirteen times its original volume when saturated in fresh water. If the same clay is added to water containing sodium chloride (NaCl), the expansion is reduced to about threefold. If the bentonite is added to a calcium hydroxide [$Ca(OH)_2$] solution, the expansion is suppressed even further, to less than twofold. This reduction in swelling is produced principally by ion exchange within the crystalline lattice of the clay. The absorbed 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 capacity.

As a rule (and as indicated by the preceding example), the divalent cations such as Ca^{2+} produce a greater collapse of the lattice than the monovalent ions such as Na^+. Exceptions to this rule may be found with potassium (K^+) and hydrogen (H^+) ions. The potassium ion, owing to its atomic size, is believed to fit almost exactly within the cavity on 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 because of its relatively high bonding energy. Therefore, in most cases the presence of H^+ interferes with the cation exchange capacity of most clays. This has been verified by several authorities. Orcutt and coworkers (1955) showed that sorption of Ca^{2+} by halloysite was increased by a factor of 9 as the pH was increased from 2 to 7. Although these data are limited and qualitative, they are sufficient to establish a trend. Grim (1953, 1962) indicated that this trend would be expected to continue to a pH range of 10 or higher. Stabilization of clays with cement or lime represents one condition in which clays are subjected to Ca^{2+} at high pH (see Secs. 6A.7.1.2 and 6A.7.1.3). Under any conditions, as the concentration of exchanged ions within the clay increases, the capacity of a soil for ion exchange decreases, and hence the potential for moisture sorption and desorption (swelling/shrinkage) also decreases.

The preceding discussions have referred to changes in the potential for volumetric expansion caused 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^+ substituted montmorillonite will be more volatile (expansive) than will soils containing montmorillonite with equivalent substitution of Ca^{2+} or Fe^{3+}.

To this point, the discussion has been limited to inorganic ion exchange. However, available data suggest that organic ion adsorption might have even more practical importance to construction problems (McLaughlin et al., 1976). The exchange mechanism for organic ions is basically identical to that discussed above except 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 (1939) reported that montmorillonitic clays lost or reduced their tendency to swell in water when treated with several selected organic cations. The surface 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 (fractures in the macrofabric). The effect of this cracking is to increase the permeability of the soil (Foreman and Daniel, 1986), which can be helpful to reduce runoff and ponding and to facilitate penetration by chemicals for stabilization. The extent of these benefits would depend on the performance of the specific chemical product. From a broad viewpoint, certain chemical qualities—such as high pH, high OH substitution, low ionic radii, high molecular size, high polarity, high valence (cationic), and highly polar vehicles-tend to help stabilize active clays. Examples of organic chemicals that possess a combination of these features include polyacrylamides, polyvinylalcohols, polyglycolethers, polyamines, polyquarternaryamines, pyridine, collidine, and certain salts of each.

Because none of the organic chemicals listed above possess all the desired qualities, they are often blended with other additives to enhance their performance. For example, the desired pH can be attained by the addition of hydrated lime [$Ca(OH)_2$], hydrochloric acid (HCl), or acetic acid (C_2H_3OOH); the polar vehicle is generally satisfied by dilution with water; high molecular size can be accomplished by polymerization; and surfactants can be used to improve penetration of the chemical through the soil. Additional details concerning the chemical reactions of base exchange in montmorillonite are given in the next two sections.

Organic chemicals generally can be formulated to be far superior to lime with respect to stabilization of clay. The organic products can be rendered soluble in water for easy introduction into the soil. Chemical characteristics can be more finitely controlled, and the stabilization process is more nearly permanent. About the only advantages lime has over specific organics, at present time, are lower treatment cost, more widespread usage (general knowledge), and greater availability.

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 with this exchange capacity dictate the affinity for water and bonding to the structure of the clay.

Chemical Bonding in Cation Exchange. The principal conditions that create the affinity of a clay to cation bonding include the following:

1. *Broken bonds around the edges of the silica-alumina units.* These broken bonds give rise to unsatisfied negative charges that can be balanced by adsorbed cations. In montmorillonite, this accounts for about 20% of the cation exchange capacity.
2. *Substitutions of cations within the crystalline lattice*. Examples include Al^{3+} for Si^{4+} in the tetrahedral sheet, and ions of lower valence, particularly Mg^{2+} for Al^{3+}, in the octahedral sheet. The latter substitution results in unbalanced negative charges within the structural units of the clay (see Fig. 6A.201). These exchanges account for about 80% of the total cation exchange capacity of montmorillonite.
3. *Hydrogens of exposed hydroxyl groups that are replaced by exchangeable cations*. There is some doubt that this occurrence is substantial in the cation exchange reaction, since it seems that the hydrogen bond to the hydroxyl group would resist the exchange reaction (Grim 1962).

A simplified relationship between the exchanged cation and the substituted cation, as well as that relationship to adsorbed water, is also shown in Fig. 6A.201. Water held directly on the surface of the unit is in a physical state different from liquid water. Generally, the thickness of the nonliquid layer is restricted to 3 to 10 molecular layers of interlayer water (Brown, 1989).

The manner in which adsorbed ions on the surface of clay affect the adsorption of water can be described in simple terms as follows:

1. Adsorbed ions may serve as a bond to hold the clay mineral units together or limit the distance over which they can be separated. Either phenomenon would resist the intrusion of water.
2. The adsorbed cations may become hydrated (that is, develop an envelope of water molecules), and then might either interfere with or influence the configuration of adjacent adsorbed water molecules.
3. The size and geometry of the adsorbed ion may influence the manner in which it fits into the configuration of the adsorbed water molecules. This would thereby influence the nature of the configuration of the adsorbed water molecules and the extent to which the configuration could develop.

Hence it follows that the type of bonding (or the energy required to break this bond) suggests the ease by which water can be removed or exchanged from the clay. Temperature is generally the source of that energy. Water lost at various levels of temperature can be categorized as (*a*) water in the pores (interstitial), on the surface, and on the surface and around the edges of the clay mineral particles (surface adsorbed); (*b*) interlayer water between the unit-cell layers of the minerals; and (*c*) the water that occurs within the tubular openings between elongated structural units (sepiolite, attapulgite, polygorskite). Type *a* water is easily removed, substantially in its entirety, with little energy such as drying at slightly above room temperature. Types *b* and *c* water require definite energy for their removal. Specifically, in the case of montmorillonite, temperatures in the range of about 100°C (212°F) are required for the removal of interlayer water. Some clay minerals—montmorillonite in particular—rehydrate with great difficulty if the dehydration is absolutely complete but with great ease if only trace amounts of water remain between unit layers.

The amount of energy required to liberate a water molecule depends on the strength of the bond

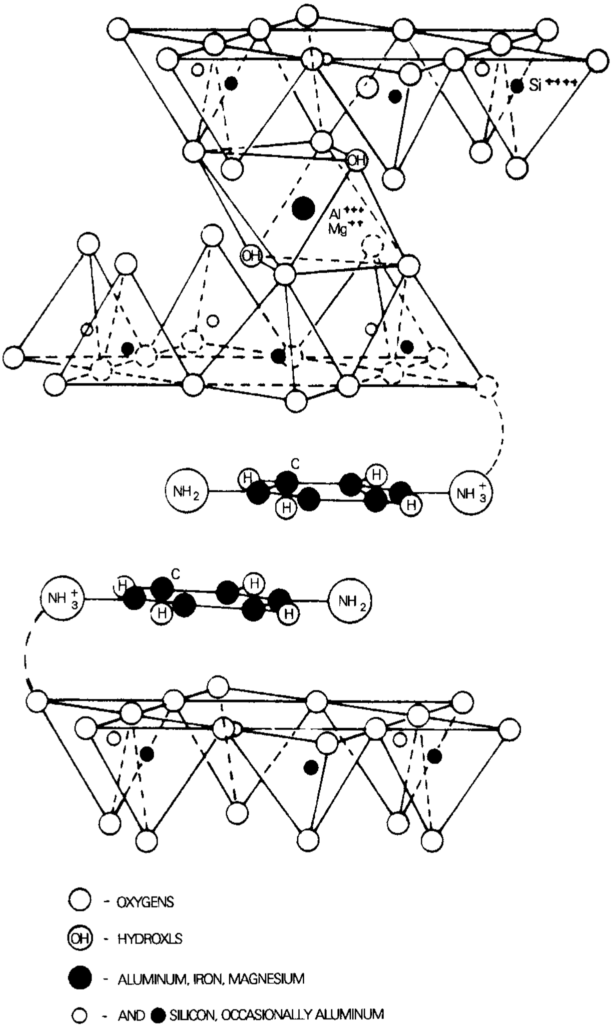


FIGURE 6A.201 Cationic substitution within clay crystalline structure (from Brown, 1992).

between that molecule and the clay particle. These bonds typically involve intermolecular electrostatic forces rather than the more conventional ionic or covalent bonds. [Ionic bonds (typical of salts) result from the complete transfer of electrons. Covalent bonds result from the sharing of a pair of electrons, one supplied by each atom forming the bond. (Shared electrons must be of opposite spin.)] The electrostatic (coulombic) forces generally develop from some variation of hydrogen bonding, ion-dipole attraction, or dipole-dipole attraction. A dipole is said to exist when two equal and opposite charges are separated in space. In a pictorial representation (see Fig. 6A.202), one end of the molecule bears a partial negative charge whereas the other end has a positive charge of equal value. Molecular shapes plays an important role in determining polarity, as bond polarity alone does not necessarily lead to a polar molecule. It is entirely possible for a molecule to contain polar bonds but for the entire molecule to be nonpolar, as is the case for CO_2. The two C–O bond dipoles in the linear CO_2 molecule cancel each other, and therefore their combined effects cancel. Hydrogen bonding is an exceptionally strong dipole-dipole attraction and results from the situation where a hydrogen atom serves as a bridge between two electronegative atoms. The hydrogen is held by a rather strong covalent bond (about 75 to 100 kcal/mol depending on the electronegative atom) and by a hydrogen bond (4 to 10 kcal/mol) of purely electrostatic forces. For hydrogen bonding to become predominate, both electronegative atoms must come from the group F, O, or N (Fig. 6A.202).

In the absence of a true dipole moment, electrons in constant movement can, at any instant, exist in an unequal distribution, thus creating in effect a net dipole. The electrostatic forces so produced are referred to as van der Waals forces. The larger the molecule, the stronger the van der Waals forces. This, at least in part, gives rise to the organic treatment or modification of clay.

Organic Modification of Clay. Many clay units have a natural attraction to specific organic ions through van der Waals and coulombic forces. Larger organic ions are difficult, if not impossible, to replace by smaller ions owing to the intensity of the van der Waals forces. Furthermore, whereas smaller ions are absorbed only up to the exchange capacity, larger ions may be absorbed to a greater extent and are not disassociated.

The dipolar nature of water (plus its capacity for hydrogen bonding) makes it an excellent vehicle for the implementation of cation exchange or organic modification. The most important reactions for modification of clays include absorption, cation exchange, and interlocation (Evans and Pancoski, 1989). Interlocation refers to the process of absorption owing to the pillaring of clay. Pillaring is the process whereby the clay platelets are forced apart, thereby creating greater porosity. Both effects provide greater access to the surface for organic cations to replace exchangeable inorganic cations (see Fig. 6A.203). The organic cations are more restrictive to the inclusion of water than are the inorganic cations. In fact, the organically modified clays become organophilic (hydrophobic) rather than hydrophilic when treated with long-chain organic amines. The structure of smectite modified by an organic amine is depicted in Fig. 6A.203.

In addition to the foregoing, many organic chemicals tend to shrink the double diffuse layer that surrounds the clay particles, causing the clay particles to 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 (Broderick and Daniel, 1990). The exposure of greater surface area may further facilitate the base exchange of certain organic molecules (Jordan, 1949).

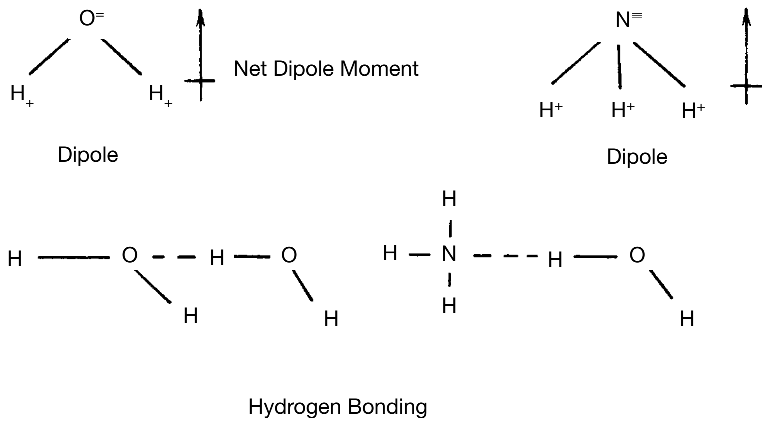


FIGURE 6A.202 Dipole moments and hydrogen bonding (from Brown, 1992).

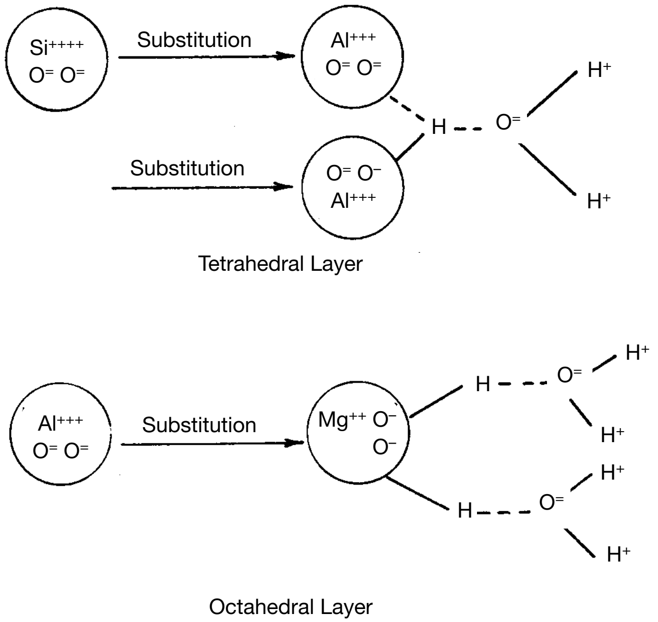


FIGURE 6A.203 Structure of organically modified smectite layers (from Grim, 1968).

6A.7.1.2 Cement

The most common application of cement-stabilized soils is for roadway and airfield pavement systems, but cement stabilization has also been used successfully in many other types of applications, including the following:

1. Strengthening and stiffening bearing soils to support structures by the overexcavation/replacement method (Sec. 6A.2)
2. Reducing the potential for swelling and shrinkage in expansive soils
3. Reducing the potential for liquefaction in liquefiable sand and silty sand deposits
4. Slope protection (erosion control) for embankments, dams, canals, reservoirs, etc.
5. Strengthening of embankment materials to allow steeper side slopes
6. Repairs of failed slopes

Except for grouting techniques, which are discussed in Sec. 6B, cement stabilization is achieved by wetting or drying the soil to an appropriate moisture condition, mixing the cement into the soil, and compacting the stabilized soil into place. The soil to be stabilized may be either an in situ soil that is excavated and replaced in the same location or a borrow material that is used in a fill.

Cement can be used to stabilize all types of soils except organic soils containing more than about 1 to 2% organic matter (Ingles and Metcalf, 1973). Fat (highly plastic) clays are difficult to stabilize with cement for two reasons: (*a*) It is difficult to mix the cement into the clay in the field, and (*b*) high percentages of cement are required to effect a significant change in engineering properties. Pretreatment of fat clays with a small percentage (2 to 3%) of either hydrated lime or cement is a common method used to overcome these difficulties. The pretreatment, also called *modification*, reduces the plasticity of the soil and renders the soil more workable. After a curing period of 1 to 3 days, the modified soil is stabilized with cement in the usual way. Certain salt solutions, especially sulfates, can disrupt the structure of the cement and thus reduce the effectiveness of the cement. This problem can be overcome in most cases by increasing the cement content. Some salts, if present in the soil-cement mixture at the time the cementitious reactions occur, may have a beneficial effect (e.g., Lambe et al., 1960).

Portland cement, because it is readily available and is relatively inexpensive, is the most widely used type of cement for stabilizing soils, but any type of cement may be used. Ordinary Portland cement (type I) and air-entraining cement (type IA) have been used extensively and have given about the same results (Little et al., 1987). Type II cement, because of its greater resistance to sulfates, has been used more frequently in recent years. High early strength cement (type III) has produced higher strengths in some soils. Subsequent discussions about cement will be limited to Portland cement, because it is used predominantly in stabilizing soils.

Portland cements are hydraulic cements—that is, they set and harden by reacting with water (hydration). The four principal compounds in Portland cement are the following (PCA, 1968):

Name of compound	Chemical formula	Abbreviation
Tricalcium silicate	$3CaO \cdot SiO_2$	C_3S
Dicalcium silicate	$2CaO \cdot SiO_2$	C_2S
Tricalcium aluminate	$3CaO \cdot Al_2O_3$	C_3A
Terracalcium aluminoferrite	$4CaO \cdot Al_2O_3 \cdot Fe_2O_3$	C_4AF

The primary reactions that occur when water is added to Portland cement can be summarized as follows:

$$2(3Ca \cdot SiO_2) + 6H_2O \rightarrow 3CaO \cdot 2SiO_2 \cdot 3H_2O + 3Ca(OH)_2$$

$$2(2Ca \cdot SiO_2) + 4H_2O \rightarrow 3CaO \cdot 2SiO_2 \cdot 3H_2O + Ca(OH)_2$$

$$3CaO \cdot A1_2O_3 + 12H_2O + Ca(OH)_2 \rightarrow 3CaO \cdot A1_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$$

$$4CaO \cdot Al_2O_3 \cdot Fe_2O_3 + 10H_2O + 2Ca(OH)_2 \rightarrow 6CaO \cdot Al_2O_3 \cdot Fe_2O_3 \cdot 12H_2O$$

where $CaO_2 \cdot SiO_2 \cdot 3H_2O$ = calcium silicate hydrate (tobermorite gel)
$Ca(OH)_2$ = calcium hydroxide (hydrated lime)
$CaO \cdot Al_2O_3 \cdot Ca(OH)_2 \cdot 12H_2O$ = calcium aluminate hydrate
$CaO \cdot Al_2O_3 \cdot Fe_2O_3 \cdot 12H_2O$ = calcium aluminoferrite hydrate

C_3S hardens quickly and is primarily responsible for initial set and early strength. C_2S hardens slowly and contributes mainly to strength increase at ages beyond 1 week. C_3A liberates a large amount of heat during the first few days of hardening and contributes slightly to early strength development. Sulfate-resistant cements have less than 8% of C_3A. The formation of C_4AF during manufacturing reduces the clinkering temperature and thus assists in the manufacturing process. C_4AF hydrates rather rapidly but contributes very little to strength. A detailed discussion on the chemistry, preparation, and properties of concrete can be found in Sec. 3A.

Unhydrated Portland cements contain particles in the range of 0.5 to 100 μm, with a mean of about 20 μm (Ingles and Metcalf, 1973). Calcium silicate hydrate (CSH), also known as *tobermorite*, is the predominant cementing compound in hydrated Portland cement. Although the specific surface of dry portland cement powder is only about 0.3 m^2/g (1460 ft^2/lb), the tobermorite gel after hydration has a specific surface of about 300 m^2/g (1.46×10^6 ft^2/lb) (Little et al., 1987). In predominately coarse-grained soils, this large specific surface is responsible for the cementing action of cement pastes owing to adhesion forces to adjacent surfaces of particles. In fine-grained soils, the clay phase may also contribute to the stabilization through solution in the high-pH environment and reaction with free lime from the cement to form additional CSH. The crystallization structure formed by the hardened cement is primarily extraneous to the soil particles. This structure can be disrupted by subsequent swelling of soil particle groups (clods) if insufficient cement is used.

The amount of cement needed for treatment depends on the type of soil being treated and the intended use and desired engineering characteristics of the treated soil. The extent of treatment can be classified by two general categories—modification and stabilization. *Modification* occurs when a low percentage of cement is incorporated within a soil that improves its workability owing to a reduction in plasticity and an agglomeration of particles. If a high percentage of cement is mixed with the soil, the surface molecular properties of the soil are changed and the grains are cemented together. Typical values of cement content used to stabilize soils according to their AASHTO and USCS group symbols are given in Table 6A.19, which can be used as a general guideline to estimate the amount of cement that may be required to stabilize a particular soil.

As a general rule, the cement requirement increases as the fine-grained fraction increases. The one exception to this rule is that poorly graded (uniform) sands require more cement than sandy soils containing some silt and clay.

The actual proportion of cement to be used on any particular project should be determined from an appropriate mixture design evaluation. The mixture design procedure typically consists of Proctor tests on the cement-stabilized soil to determine the approximate optimum water content and maximum dry density for either standard or modified energy, followed by a series of laboratory tests on cement-stabilized specimens with varying cement contents to determine the minimum cement content that will achieve the desired engineering properties. For example, if strength is the primary requirement, the general procedures for a typical mixture design would be as follows:

1. Establish a minimum desired strength based on rational engineering procedures. In soil–cement applications, the minimum strength is typically specified as unconfined compressive strength. In applications where the cement-stabilized soil will be subjected to significant confining pressures, it is more appropriate to specify strength in terms of friction angle and cohesion intercept determined by triaxial or other appropriate strength tests.
2. Estimate the required cement content based on personal experience or guidelines provided by others (such as Table 6A.19).
3. Conduct a series of Proctor tests on the cement-stabilized soil at three cement contents—the estimated value from step 2 and one value above and one value below the estimated value. It is com-

TABLE 6A.19 Typical Cement Requirements According to AASHTO and USCS Soil Groups

AASHTO group symbol	Corresponding USCS group symbols*	Usual range in cement requirement†	
		Percent by volume	Percent by weight
A-l-a	GW, GP, GM, SW, SP, SM	5 to 7	3 to 5
A-l-b	GM, GP, SM, SP	7 to 9	5 to 8
A-2	GM, GC, SM, SC	7 to 10	5 to 9
A-3	SP	8 to12	7 to 11
A-4	ML, CL-ML	8 to 12	7 to 12
A-5	ML, MH, CH	8 to 12	8 to 13
A-6	CL, CH	10 to 14	9 to 15
A-7	OH, MH, CH	10 to l4	10 to l6

*Based on correlation presented by Air Force.

†For most *A* horizon soils, the cement content should he increased by 4 percentage points if the soil is dark gray to gray, and by 6 percentage points if the soil is black.

mon practice to use 1% increments for estimated cement contents of 5% or less and 2% increments for estimated cement contents of 6% or more. As an illustration, if the estimated cement content is 8%, the Proctor tests typically would be conducted at cement contents of 6, 8, and 10%. Determine the optimum water content and maximum dry density for each of the three cement contents.

4. Perform a series of unconfined compression tests on specimens compacted at the corresponding optimum water content to the corresponding maximum dry density for each of the three cement contents used in step 3. Cement-stabilized specimens are generally stored (cured) at room temperature in a relative humidity of approximately 100% for periods of 2, 7, and 28 days and then immersed in water for 4 h prior to testing (see ASTM D1632 and D1633). Usually at least three replicate specimens are tested for each cement content and curing period.
5. Three situations are possible depending on how the measured values of strength obtained in step 4 compare with the specified minimum value developed in step 1.
   a. The measured values of unconfined compressive strength at the three cement contents are both above and below the specified minimum value. The actual (design) cement content to be used can be determined as the lowest cement content that yields strengths greater than or equal to the minimum desired value. In cases where a small difference in cement content would significantly affect the economics of the project, steps 2 and 3 could be repeated at intermediate cement contents to better establish the most economical cement content for the desired strength characteristics.
   b. The measured values of strength at all three cement contents are less than the specified minimum value. Repeat steps 2 to 4 at three cement contents higher than the previous maximum value.
   c. The measured values of strength at all three cement contents are greater than the specified minimum value. Repeat steps 2 to 4 at three cement contents lower than the previous minImum value.
6. Additional testing can be conducted to determine if other important engineering characteristics are met for the preliminary design cement content established in steps 2 through 5. If the other criteria are met, the preliminary design cement content is the final design cement content. If not, the final design cement content is determined based on the results of the additional tests, ensuring that the strength requirement is also met. For example, in soil-cement applications in pavement systems, wet-dry and freeze-thaw tests (ASTM D559 and D560) are also conducted. In applications for building foundations, compressibility and liquefaction resistance are also important engineering properties.

The presence of organic material or sulfates in a soil may inhibit the proper hydration of the cement. Organics with low molecular weights, such as nucleic acid and dextrose, retard hydration and reduce strength (Clare and Sherwood, 1954). Certain types of organics, such as undecomposed vegetation, may not adversely affect cement stabilization. For situations where organics are present in the soil, Little and coworkers (1987) recommend conducting a pH test on a 10:1 mixture (by weight) of soil and cement. If the pH of the soil-cement 15 min after mixing is at least 12.1, the organic content probably will not interfere with normal hardening. One method by which organics repress the hydration of cement is absorption of calcium ions liberated during the hydrolysis of the calcium silicates and aluminates in the cement grains (Clare and Sherwood, 1954, 1956). These ions are subsequently not available to form the compounds constituting the set cement matrix. The addition of a material with readily available calcium, such as calcium chloride ($CaCl_2$) or hydrated lime, may satisfy the adsorptive capacity of the organic matter before hydration of the bulk of the cement has begun, thus enabling the soil to be successfully stabilized with cement. For the calcium additive to be effective, it must be present in a sufficient quantity to completely satisfy the adsorptive capacity of the organic matter, as illustrated by the following data from Clare and Sherwood (1956) for an otherwise clean silica sand stabilized with 10% ordinary Portland cement:

Additive (by weight of dry soil)	Unconfined compressive strength	
	psi	kPa
None (clean silica sand)	355	2450
0.01% tartaric acid	285	1970
0.02% tartaric acid	190	1310
0.03% tartaric acid	150	1030
0.04% tartaric acid	25	172
0.05% tartaric acid	10	69
0.10% tartaric acid + 0.25% calcium chloride	10	69
0.10% tartaric acid + 0.50% calcium chloride	10	69
0.10% tartaric acid + 1.00% calcium chloride	260	1793

Research by Sherwood (1962) has shown that the presence of sulfates can significantly affect the durability, integrity, and strength of cement-stabilized cohesive soils but has relatively little influence on cement-stabilized granular soils. These trends are illustrated in Fig. 6A.204, which shows how immersion in magnesium sulfate solutions of different concentrations affected the unconfined

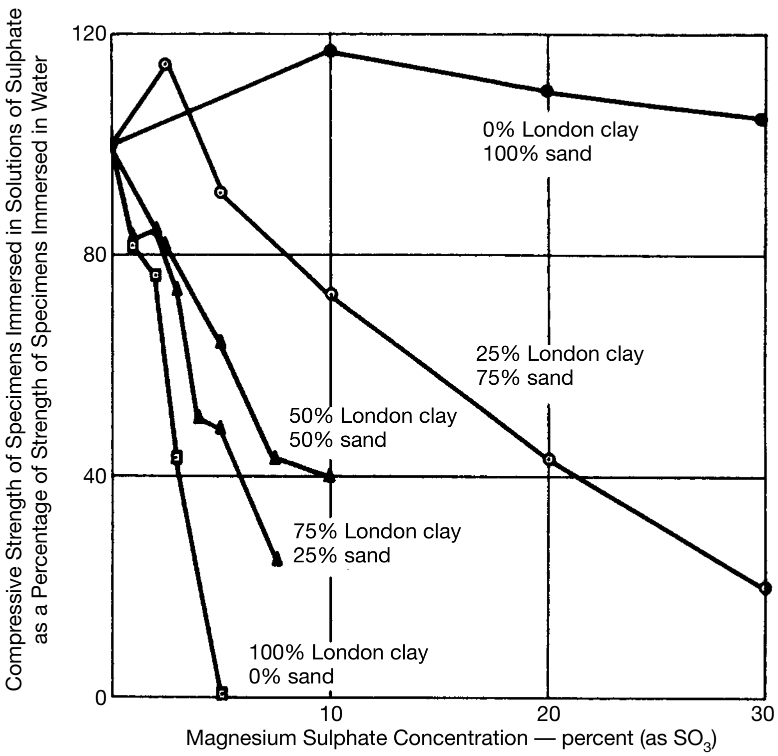


FIGURE 6A.204 Effect of immersion in magnesium sulfate solution on the unconfined compressive strength of London clay-silica sand mixtures stabilized with 10% ordinary Portland cement (from Sherwood, 1962).

compressive strength of cement-stabilized specimens of London clay, silica sand, and various mixtures of the two soils. The presence of magnesium sulfate in the immersion water reduced the strength of all specimens containing London clay except one. In contrast, the strengths of the 100% silica sand specimens were somewhat greater when immersed in magnesium sulfate solutions than when immersed in water without magnesium sulfate. Note that 0.5% magnesium sulfate was sufficient to cause complete disintegration of the stabilized London clay. Similar trends were found for the same soils immersed in calcium sulfate solutions. Owing to the problems that sulfates can cause in cement-stabilized cohesive soils, Little and colleagues (1987) suggest that the use of cement be avoided for fine-grained soils containing more than about 1% sulfates.

6A.7.1.3 Lime

Two types of lime are commonly used to stabilize cohesive soils—CaO (calcium oxide, also known as *quicklime*) and $Ca(OH)_2$ (calcium hydroxide, also referred to as *hydrated lime* or *construction lime*). Hydrated lime is the most frequently used lime product for soil stabilization in the United States, and quicklime is used more often in Europe (Bell, 1988). Both quicklime and hydrated lime are produced by burning limestone ($CaCO_3$, calcium carbonate), as illustrated by the following reactions (Fox, 1968):

$$CaCO_3 + \text{Heat} \rightarrow CaO + CO_2$$

$$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat}$$

Less pure forms of lime are also sometimes used to stabilize soils. These alternate forms include monohydrated dolomitic lime [$Ca(OH)_2 \cdot MgO$], dolomitic quicklime ($CaO \cdot MgO$), and by-product limes (Little et al., 1987). Ground limestone ($CaCO_3$), also known as *lime dust* or *agricultural lime*, is ineffective in chemically stabilizing soils but is sometimes used as a filler to increase the fines content of a soil, which modifies the gradation of the soil for better compaction and sometimes enhances stabilization by other chemicals.

Treatment with lime is generally effective in soils with plasticity indices in the range of 10 to 50 (Bell, 1988). Lime stabilization is typically ineffective in soils with little or no clay content because the primary improvements in engineering behavior are produced by reactions between the lime and the clay minerals. All clay minerals react with lime, with the strength of the reaction generally increasing in proportion to the amount of available silica. Thus, the three-layer clay minerals (e.g., montmorillonite, illite, chlorite) are more reactive than the two-layer clay minerals (e.g., kaolinite). The availability of the silica in the clay minerals is also important. Hence, illite and chlorite are much less reactive than montmorillonite because their silicas are bound to other silicas by ions that are not readily exchangeable.

Four basic types of reactions occur in cohesive soils treated with lime: (*a*) carbonation, (*b*) cation exchange, (*c*) flocculation-agglomeration, and (*d*) pozzolanic cementation. Carbonation occurs if carbon dioxide from the air or stagnant water gains entry into the lime-soil matrix and converts the lime back to calcium carbonate. Calcium carbonate is a weak cement because it is soluble in acidic water. Carbonation is undesirable because it reduces that amount of lime available to produce the primary cementitious reactions, which are pozzolanic. Carbonation can be prevented by using appropriate construction techniques.

Lime mixed with water results in free calcium cations that may replace other cations within the exchange complex of the soil. In some instances the exchange complex may be saturated with Ca^{2+} before the addition of lime, yet cation exchange may still occur, since the capacity for cation exchange increases as the pH increases (TRB, 1987). Cation exchange is at least partially responsible for the flocculation and agglomeration of clay particles that occur in lime-treated soils (Herzog and Mitchell, 1963). The rapid formation of calcium aluminate hydrate cementing materials probably significantly assists in the flocculation-agglomeration process (Diamond & Kinter 1965). The net result of flocculation and agglomeration is a change in texture of the soil as the clay particles clump together into larger clods.

The pozzolanic reactions that occur in lime-treated soils, although not completely understood, are similar to those that occur in cement-treated soil (see Sec. 6A.7.1.2). We know that the lime and water react with silica and alumina in the soil to form various cementitious compounds (TRB, 1987). Typical sources of alumina and silica in soils include clay minerals, quartz, feldspars, micas, and similar silicate or alumino-silicate materials (either crystalline or amorphous). The addition of lime also raises the pH of the soil, which is beneficial because it increases the solubility of the silica and alumina present in the soil. If a significant amount of lime is added to the soil, the pH may reach 12.4, which is the pH of saturated lime water. The following is an oversimplification of the reactions that occur in a lime-treated soil (TRB, 1987):

$$Ca(OH)_2 \rightarrow Ca^{2+} + 2(OH)^-$$

$$Ca^{2+} + 2(OH)^- + SiO_2 \text{ (clay silica)} \rightarrow CSH$$

$$Ca^{2+} + 2(OH)^- + Al_2O_3 \text{ (clay alumina)} \rightarrow CAH$$

where $C = CaO$
$S = SiO_2$
$A = Al_2O_3$
$H = H_2O$

These reactions occur only if water is present and able to bring Ca^{2+} and $(OH)^-$ to the surfaces of the clay particles (that is, while the pH is still high) (Bell, 1988). Hence, the reactions will not occur in dry soils and will cease in a previously wet soil if it dries out. The primary characteristics of a soil that determine the effectiveness of lime treatment include the following (TRB, 1987):

1. Type and amount of clay minerals
2. Silica-alumina ratio
3. Silica-sesquioxide ratio
4. pH
5. Organic carbon content
6. Natural drainage
7. Degree of weathering
8. Presence of carbonates, sulfates, or both
9. Extractable iron

Caution is needed when one is considering lime stabilization of soils that contain sulfates and carbonates. Sherwood (1962) provided experimental results showing that lime-stabilized soils can be deleteriously affected by sulfates in a similar manner to cement-stabilized soils. A first set of experiments was conducted in which specimens of London clay were stabilized with 10% lime and cured at constant water content for 7 days. The specimens were then immersed in solutions of either sodium sulfate or magnesium sulfate at concentrations ranging from 0 to 1.5%. In all cases, the specimens immersed in the sulfate solutions cracked and swelled to the extent that they were too weak to be tested in unconfined compression. A second series of experiments was then performed in which varying amounts of calcium, sodium, and magnesium sulfates were mixed with the soil before adding 10% lime. Two sets of specimens were made for each sulfate concentration and were cured at constant moisture content. After 7 days of curing, one set of specimens was immersed in water while the other set was allowed to continue curing at constant moisture content. At the end of a second 7-day period, both sets of specimens were tested in unconfined compression. The following trends can be observed from the results of these tests, which are given in Fig. 6A.205.

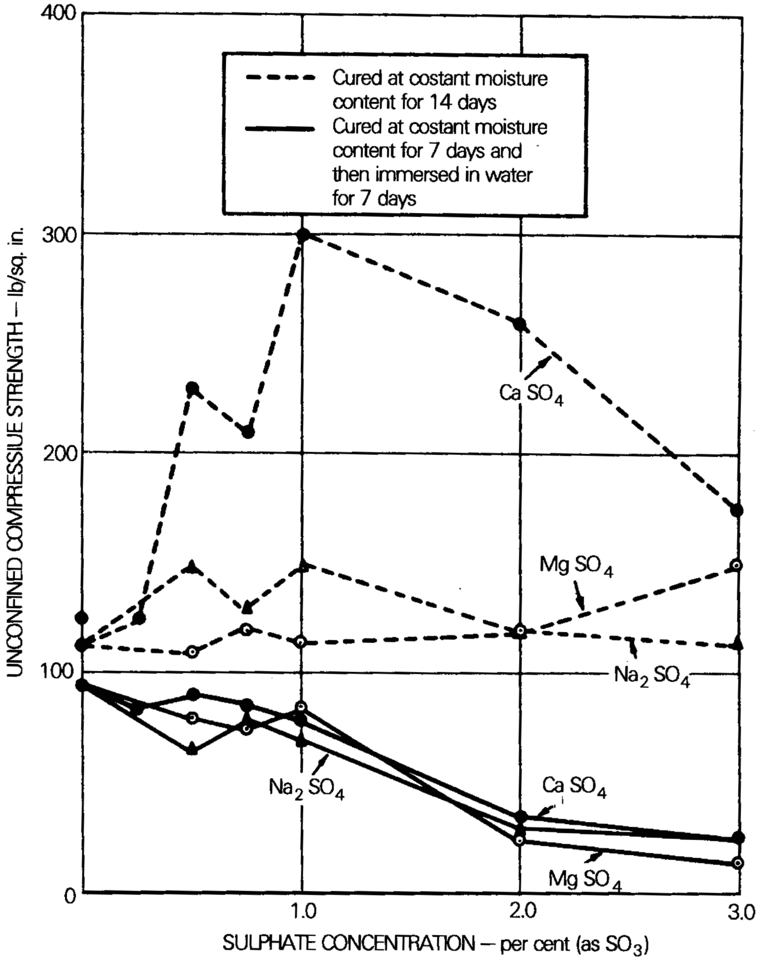


FIGURE 6A.205 Influence of sulfates on the unconfined compressive strength of London clay stabilized with 10% hydrated lime (from Sherwood, 1962).

1. For the specimens cured at constant moisture condition for 14 days, the calcium sulfate had a substantial beneficial effect, and the magnesium sulfate and sodium sulfate had no significant influence.
2. All the immersed specimens containing sulfates swelled and cracked, resulting in a loss in strength, with a general trend of decreased strength with increased sulfate content. At the higher sulfate concentrations (2%), the losses in strength were considerable, ranging from about 65 to 85%.

Poststrength testing analyses of the 2% calcium sulfate specimens indicated that the calcium sulfate had reacted with the clay and that ettringite $\{Ca_6 \cdot [Al(OH)_6]_2 \cdot (SO_4)_3 \cdot 26H_2O\}$ had been formed as a result of the reaction. Ettringite is a very expansive material that is one of the materials responsible for the deterioration of concrete by sulfate attack.

Mitchell (1986) presented a case history in which the presence of sodium sulfate in the soil contributed to the failure of lime-stabilized subbases in Las Vegas, Nevada. The results of postmortem investigations that showed the effect of the sulfates can be summarized as follows:

1. The untreated soil contained up to 1.5% soluble sodium sulfate.
2. The lime-treated soil in the failed zones had swelled more than untreated soils also exposed to water.
3. Calcium silicate hydrates (CSH), the cementitious material produced during successful lime stabilization, were not found in specimens from the failed zones. This lack of cementation was attributed to the low pH of the treated soils (8 to 10.5), well below the minimum pH of 12.4 necessary for the pozzolanic reactions to occur.
4. Significant amounts of ettringite and thaumasite $\{Ca_6 \cdot [Si(OH)_6]_2 \cdot (SO_4)_2 \cdot (CO_3)_2 \cdot 24H_2O\}$ were found in the failed zones. Thaumasite also is very expansive and is known to contribute to the deterioration of concrete by sulfate attack.
5. Negligible amounts of ettringite and thaumasite were found in treated samples from locations where failure had not occurred.

Owing to problems such as these, Mitchell (1986) recommended that lime stabilization not be used for soils containing more than 5000 ppm (0.5%) of soluble sulfates. Petry and Little (1992) indicated that the minimum level of sulfates that may represent potential problematic situations depends on the method used to extract the sulfates from the soil and may be as low as 500 ppm (0.05%) for 1:1 soil-to-water ratios to 2000 ppm (0.2%) for 1:10 soil-to-water ratios. Additional case histories in which sulfates in lime-treated soils resulted in swelling and structural damage to pavements related to the formation of ettringite and thaumasite can be found in Hunter (1988), Grubbs (1990), and Penn (1992). Hunter (1988) also provided a geochemical model for the formation of ettringite and thaumasite.

Note that lime stabilization in expansive soils with high sulfate contents is possible. This is confirmed by the fact that lime has been used successfully to stabilize sulfate-bearing soils in Texas for over 30 years (Petry and Little, 1992). Unfortunately, the construction and application techniques responsible for successful stabilization have yet to be identified. The following methods have been studied or are currently being evaluated as potential techniques to overcome the detrimental aspects of sulfates in lime-treated soils:

- Pretreatment of the soils with compounds of barium (Little and Deuel, 1989; Ferris et al., 1991).
- Double applications of lime
- Use of compounds to sustain high pH values and thus promote pozzolanic reactions
- Addition of materials to enhance pozzolanic reactions

The amount of lime needed for treatment depends on the characteristics of the soil being treated and the intended use and desired engineering characteristics of the treated soil. As with cement stabilization, the treatment of soils with lime can be divided into two general classes (Ingles and Metcalf, 1973): (*a*) modification, which reduces plasticity, improves workability, increases resistance to deflocculation and erosion, and produces a rapid increase in strength as a construction expedient and (*b*) stabilization, which provides permanent increases in strength and stiffness to increase bearing capacity, reduce settlement, and so on (Table 6A.20).

Several lime-soil mixture design methods have been developed. TRB (1987) summarizes eight procedures. These procedures are of two general types—those relating to soil modification and those pertaining to soil stabilization. The Illinois procedure will be summarized below to illustrate the basic concepts for both modification and stabilization (from TRB, 1987).

Modification. The mixture design procedure for modification is based on the effect that the lime has on the plasticity index (PI) of the soil. CBR testing can also be performed but is optional.

TABLE 6A.20 Typical range of hydrated lime content for modification and stabilization of various types of soils (from Ingles and Metcalf, 1973)

Type of soil	Typical range of hydrated lime content, % by dry weight of soil	
	Modification	Stabilization
Fine crushed rock*	2 to 4	Not recommended
Well-graded clayey gravels	1 to 3	Approximately 3
Sands†	Not recommended	Not recommended
Sandy clay	Not recommended	Approximately 5
Silty clay	1 to 3	2 to 4
Heavy clay	1 to 3	3 to 8
Very heavy clay	1 to 3	3 to 8
Organic soils	Not recommended	Not recommended

*Lime only effective if fines are plastic.
†Lime used in bitumen stabilization for promoting adhesion; quicklime in loessial materials.

1. The liquid limit (LL), plastic limit (PL), and PI of the soil treated with various percentages of lime are determined. The soil-lime-water mixtures are loose-cured (conditioned) for 1 h before the LL and PL tests are conducted.
2. A plot is prepared of PI versus lime content. The design lime content may be based on one of the following criteria: (*a*) The lime content above which no additional appreciable reduction in PT occurs; or (*b*) the minimum lime content that produces an acceptable reduction in the PI.
3. If desired, CBR tests may also be performed to assess the stability or swelling potential of the soil, or both. Depending upon the objectives of the lime treatment, curing and soaking the CBR specimens before testing is optional. If the results of the CBR tests indicate that a higher lime content is required for successful modification, the design lime content is changed accordingly.
4. The design lime content for field construction is increased by 0.5 to 1% to offset the effects of variability in soil properties and construction procedures.

Stabilization. The mixture design procedure for stabilization is based on the results of unconfined compressive strength tests:

1. Standard Proctor tests (AASHTO T-99, ASTM D698) are conducted on the natural soil and soil-lime mixtures at various lime contents to determine the optimum water content and maximum dry density for the natural soil and each of the soil-lime mixtures.
2. Specimens 2 in (51 mm) in diameter and 4 in (102 mm) in height of the natural soil and soil-lime mixtures are prepared at the corresponding optimum water content and corresponding maximum dry density. The soil-lime mixtures are cured at 120°F (48.9°C) for 48 h before testing.
3. The unconfined compressive strength of the soil-lime mixture containing 3% lime must be at least 50 psi (345 kPa) greater than that of the natural soil. If not, the soil is considered unsuitable for lime stabilization. If the criterion for 3% lime is met, the design lime content is the value above which additional lime does not produce a significant increase in strength. Minimum strength requirements are 100 psi (690 kPa) for use as a subbase and 150 psi (1030 kPa) for use as a base course.
4. The design lime content for field construction is increased by 0.5 to 1% to offset the effects of variability in soil properties and construction procedures.

TABLE 6A.21 Typical ranges in values for the chemical composition of fly ash (from NCHRP, 1976)

Principal constituent	Content by weight, %
SiO_2	28 to 52
Al_2O_3	15 to 34
Fe_2O_3	3 to 26
CaO	1 to 40
MgO	0 to 10
SO_3	0 to 4

6A.7.1.4 Fly Ash

Two waste products are produced by the burning of powdered coal—fly ash and bottom ash (boiler slag). Bottom ash collects at the base of the furnace, and fly ash is collected from the flue gases by mechanical and/or electrostatic precipitation. The particles in bottom ash range in size from fine sand to gravel, and bottom ash serves well as structural fill in road construction (Hausmann, 1990). Fly ash is powdery and of predominantly silt size and is used for a variety of purposes, including (*a*) as a stand-alone cement, (*b*) as a partial replacement for cement in concrete, (*c*) in combination with lime, cement, or both in soil stabilization, (*d*) as a fill material in the form of lime–fly ash-aggregate (LFA), cement-fly ash-aggregate (CFA), or lime-cement-fly ash-aggregate (LCFA) mixtures, and (*e*) as a drying agent to facilitate compaction of overly wet soils.

The chemical and physical properties of fly ash vary widely and depend primarily on the mineralogy and purity of the coal and the process and equipment used to burn the coal and recover the fly ash. Owing to variations in the properties of the coal burned at any power plant, fly ash within the same stockpiles can be quite heterogeneous. The primary chemical components of fly ash are silica (SiO_2), alumina (Al_2O_3), ferric oxide (Fe_2O_3), and calcium oxide (CaO), and the secondary constituents include magnesium oxide (MgO), titanium oxide (TiO_2), alkalies (Na_2O and K_2O), sulfur trioxide (SO_3), phosphorous oxide (P_2O_5), and carbon (Hausmann, 1990). Typical chemical compositions and physical properties of fly ashes are given in Tables 6A.21 and 6A.22. Fly ash is classified as either class F (pozzolanic properties only) or class C (pozzolanic and some cementitious properties) depending primarily on its CaO content (ASTM C618). Type F fly ashes are generally produced by burning anthracite or bituminous coal, and type C fly ashes are normally derived from the combustion of subbituminous or lignite coal. In the United States, characteristics of the fly ashes from the major coal-producing regions can be broadly summarized as follows (Little et al., 1987; refer to Fig. 6A.206):

- Fly ashes from the bituminous coals of the Appalachian region generally behave as true pozzolans, with little or no inherent cementitious properties.
- Fly ashes from the midcontinent coals, which are mostly bituminous, have some inherent cementitious properties owing to the CaO present in the ash.
- Fly ashes from the subbituminous and lignite coals of the northern and western plains states have a high natural CaO content and are highly cementitious even without the addition of lime.

Several kinds of chemical reactions occur in naturally cementitious fly ash and lime–fly ash mixtures. These reactions are complex, owing in part to the heterogeneous nature of fly ash and have not been completely defined. The principal cementitious reactions are similar to those that

TABLE 6A.22 Typical physical properties for fly ash (from NCHRP, 1976)

Property	Description or typical range in values
Color	Usually light gray, but can vary from light tan through black
Shape	Spherical, solid, or hollow
Glass content (amorphous material)	71 to 88%
Size	1 to 80 μm (3.9×10^{-5} to 3.1×10^{-3} in)
Specific surface	0.2 to 0.8 m^2/g (8800 to 35,000 in^2/oz)

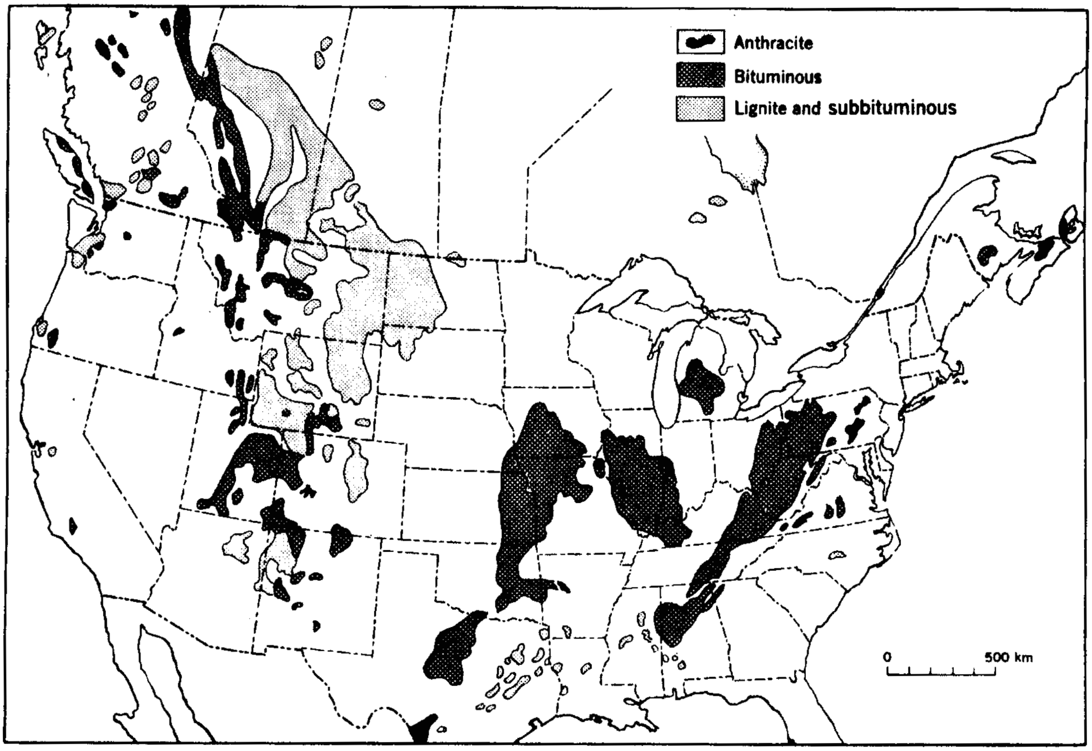


FIGURE 6A.206 Coal fields of the United States and southern Canada (from Skinner and Porter, 1987).

take place in cement and lime-treated soils (see Secs. 6A.7.1.2 and 6A.7.1.3): Alumina and silica in the fly ash react with lime (either naturally occurring in the fly ash or added quicklime or hydrated lime), in the presence of water, to produce calcium silicate hydrates and calcium aluminosilicate hydrates. An illustrative list of some reactions that may occur is given as follows (Minnick, 1967):

$$CaO \xrightarrow{H_2O} Ca(OH)_2$$

$$CaO \xrightarrow{H_2;CO_2} CaCO_3 + H_2O$$

$$Ca(OH)_2 \xrightarrow{CO_2} CaCO_3 + H_2O$$

$$Ca(OH)_2 + SiO_2 \xrightarrow{H_2O} xCaO \cdot ySiO_2 \cdot zH_2O$$

$$Ca(OH)_2 + Al_2O_3 \xrightarrow{H_2O} xCaO \cdot yAl_2O_3 \cdot zH_2O$$

$$Ca(OH)_2 + SiO_2 + Al_2O_3 \xrightarrow{H_2O} xCaO \cdot ySiO_2 \cdot zAl_2O_3 \cdot wH_2O$$

$$Ca(OH)_2 + (SO_3)^{2-} + Al_2O_3 \xrightarrow{H_2O} xCaO \cdot yAl_2O_3 \cdot zCaSO_4 \cdot wH_2O$$

Note that Mg^{2+} may be substituted for Ca^{2+} in all equations.

Not all fly ashes are reactive even when lime is added, so the procedures provided in ASTM C593 can be used to evaluate the suitability of a fly ash for use in lime–fly ash mixtures. Low temperatures, high organic content, and high sulfate content can have deleterious effects on the cementitious reactions. These reactions are retarded at low temperatures and are almost nonexistent at temperatures below about 40°F (14°C) (Leonard and Davidson, 1959; Little et al., 1987). The organics probably affect the cementitious reactions in fly ash in a similar manner as in Portland cement, primarily by absorbing calcium ions. If high organic content is suspected, loss-on-ignition tests (ASTM C114) can be conducted to determine carbon content. Fly ashes having sulfate contents of 5 to 10% appear to have lower rates of initial hydration (Ferguson 1993). For high sulfate contents, high initial strengths have been observed, but durability appears to be substantially reduced. The formation of ettringite, which is highly expansive, is also a potential problem when using fly ashes with high sulfate content.

The amount of fly ash used in any application depends on a variety of factors such as the desired engineering characteristics of the stabilized material, and the properties of the fly ash, the additives (if any), and the soil to be stabilized. For applications where self-cementing fly ash is used without additives, more fly ash is required to achieve the same stabilizing effect as that produced by Portland cement, with typical fly ash requirements on the order of 15 to 25%. In LFA, CFA, and LCFA applications, the stabilizer is defined by designating the total amount of stabilizer (fly ash plus additives) and the lime-to-fly-ash ratio or the cement-to-fly-ash ratio (or both). For example, an LFA mixture of 5% lime, 15% fly ash, and 80% aggregate would be specified as 20% lime plus fly ash with a 1:3 lime-to-fly-ash ratio. Lime-plus-fly-ash contents generally range from 12 to 30%, with fine-grained soils requiring higher percentages and well-graded granular soils requiring lower percentages (NCHRP 1976). The lime-plus-fly-ash content generally increases with increasing angularity and roughness of the particles. Lime-to-fly-ash ratios normally range from 1:10 to 1:2, with ratios of 1:3 to 1:4 being common. Factors that tend to increase the lime requirement are greater fines content, increased PI, and increased pozzolanic reactivity of the fly ash.

Because of the unique characteristics of each stockpile of fly ash and the complex chemical reactions that occur in fly ash-stabilized soils, mixture proportions cannot be established from separate analyses of the components (fly ash, soil, additives). Proportions are properly determined using laboratory or field-based design procedures with representative samples of the component materials to be incorporated into the field mixture. Mixture design procedures for fly ash stabilization are similar to those used for cement or lime-treated soils (Secs. 6A.7.1.2 and 6A.7.1.3) but are somewhat more complicated when additives are used, because the optimal content for the fly ash and the additive(s) must be determined. Figure 6A.207 illustrates a typical mixture design procedure for LFA when strength and durability are the most important engineering properties.

6A.7.1.5 Emulsions, Sodium Silicates, and Other Chemicals

A variety of other chemicals—including emulsions, sodium silicates, polyacrylamides, phosphoric acid, and iron and aluminum oxides—have been successfully used to stabilize soils. The use of these other chemicals in foundation applications has been limited and thus will be only briefly discussed here. A discussion of some of the less frequently used chemicals can be found in ASCE (1987).

An emulsion is a fluid formed by the suspension of a very finely divided oily or resinous liquid in another liquid. Various emulsions have been used to stabilize soils, of which the most common types are bituminous. The principal component of bituminous emulsions is bitumen, which refers to a class of cementitious substances composed mainly of hydrocarbons with high molecular weights. Included in the class of bitumens are asphalts, tars, pitches, and asphaltites. Bituminous emulsions

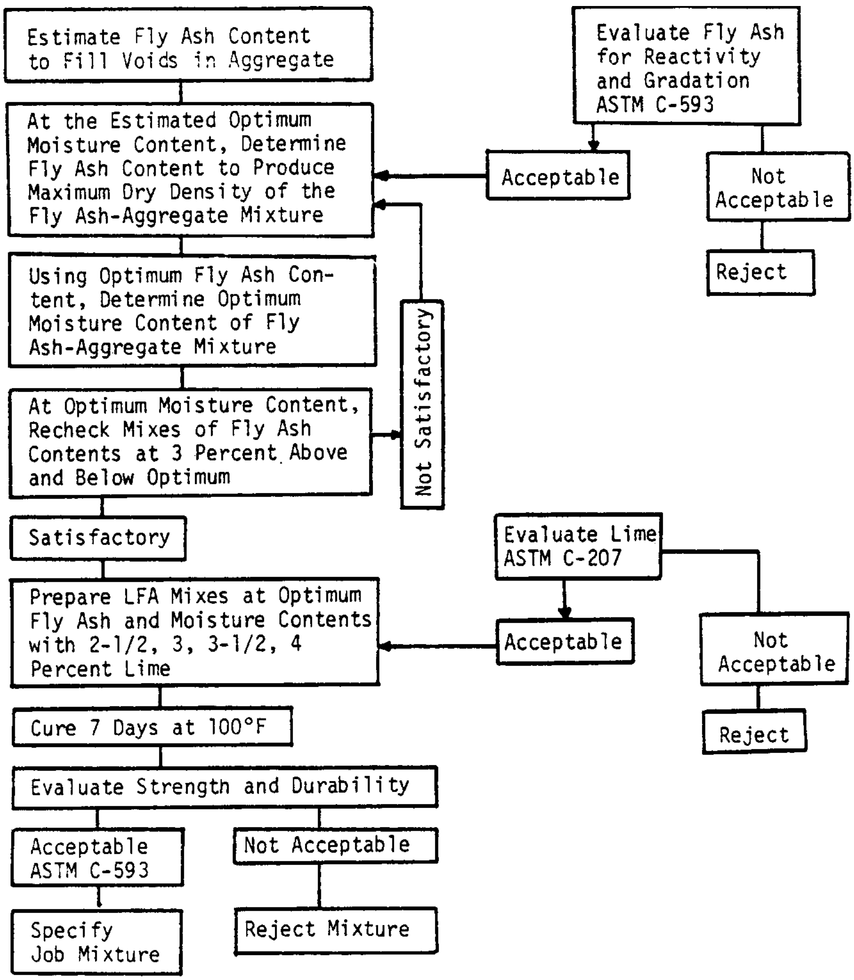


FIGURE 6A.207 Mixture design flow chart for lime-fly-ash-aggregate (LFA) mixtures (from Little et al., 1987).

are typically either anionic or cationic, depending on the predominance of the type of charges on the discontinuous phase, and are incorporated into the soil using either hot or cold mixing methods. A number of nonbituminous emulsions have been developed to stabilize soils, many of which are proprietary. The primary use for emulsions in treating soils has been in pavement applications (stabilization of soils for use in subgrade, subbase, and base courses). Detailed discussions of bituminous stabilization of soils are given in Ingles and Metcalf (1973) and Little and coworkers (1987), among others.

Details on the use of sodium silicates and acrylamides in grouting applications are given in Section 6B. The difference between the usage of these materials for grouting and chemical stabilization is the nature of the chemical reactions. In solution grouting, the basic chemical is combined with a hardener that, upon setting, produces a solid material that fills the pore spaces of the soil. Thus, the chemical reaction in grouting is external to the soil particles; that is, the reaction is between the primary component and the hardener. In contrast, when used in chemical stabilization, sodium silicates and polyacrylamides react chemically with the soil particles to produce the characteristics of stabilization (reduced swelling potential, increased strength and stiffness, and so on). Sodium silicates and polyacrylamides are injected as solutions into the soil.

The cementitious reactions that occur between sodium silicate and the clay fraction of a soil can be summarized as follows (Sokolovich and Semkin, 1984). Adsorbed calcium cations are exchanged with sodium cations from the introduced reagent:

$$Ca^{2+} + 2NaOH \rightarrow 2Na^{+} + Ca(OH)_2$$

Hydrated alumina interact with liberated calcium hydroxide:

$$Al(OH)_3 + NaOH \rightarrow NaAlO_2 + 2H_2O$$

$$2NaAlO_2 + Ca(OH)_2 \rightarrow Na_2O \cdot CaO \cdot Al_2O_3 \cdot 2H_2O$$

as well as

$$2A1(OH)_3 + 3Ca(OH)_2 \rightarrow Ca_3[Al(OH)_6]_2$$

Interaction of hydrated silica with calcium hydroxide and sodium aluminate is as follows:

$$Si(OH)_4 + Ca(OH)_2 \rightarrow CaO \cdot SiO_2 \cdot 3H_2O$$

$$25i(OH)_4 + 2NaAlO_2 \rightarrow Na_2O \cdot Al_2O_3 \cdot 2SiO_2 + 4H_2O$$

Because the permanence of sodium silicate grouts has been questioned, caution should be exercised when considering the use of sodium silicate for soil stabilization.

6A.7.2 Engineering Properties and Behavior of Chemically Stabilized Soils

In many instances, chemical stabilization of a soil produces significant and often dramatic changes in the engineering properties and behavior of the soil compared to its natural condition. Although the magnitude of the changes produced in any soil depend on the type and amount of chemicals used, the qualitative features of these changes in successfully stabilized soils are often similar for all the types of chemicals commonly used. In this section, results illustrating the nature and magnitude of these changes will be presented and discussed according to the different engineering properties that may be affected by the stabilization process. Most of the data and results presented will be for soils treated with cement, lime, and fly ash because of the vast amount of information publicly available for these stabilizers and the dearth of data available for most other chemicals.

6A.7.2.1 Plasticity and Workability

As discussed in Sec. 6A.7.1, chemical modification using admixtures involves mixing a relatively small amount of a chemical stabilizer into a cohesive soil to improve its workability from a construction standpoint—primarily resulting in a reduction in plasticity and an agglomeration of particles into larger clods. The net result of successful modification is a friable soil with a silty texture.

These changes are visually illustrated in Fig. 6A.208 in which a highly plastic clay is shown in its natural state and after modification with hydrated lime.

Quantitative changes in the liquid limit (LL), plastic limit (PL), plasticity index (PI), and shrinkage limit (SL) are shown in Figs. 6A.209 and 6A.210 for lime and cement-treated soils, respectively, as a function of stabilizer content. In general, chemical modification of a cohesive soil reduces the liquid limit and increases the plastic limit, with both changes producing a reduction in the plasticity of the soil as indicated by a decrease in plasticity index. The reduction in plasticity can be extensive; for example, the PI of Porterville Clay (Fig. 6A.209) was reduced from 47 to 3 with the addition of 8% lime. The reduction in plasticity increases with increasing stabilizer content up to a certain content, above which little or no additional effect occurs. Many, but not all, soils can be made nonplastic through chemical stabilization (at contents typically used in practice), and generally the amount of a particular stabilizer required to effect a nonplastic state increases with increasing PI of the natural soil. The changes in plasticity are also time-dependent, as depicted in Fig. 6A.211. In most cases, a considerable reduction in PI occurs within the first 24 h, and most of the reduction occurs within the first week. However, the process may continue for years.

6A.7.2.2 Swelling

Areas of the United States that have a general and local abundance of high-clay, expansive soils are shown in Fig. 6A.212. The darker areas indicate those regions suffering most seriously from problems related to expansive soils. These data indicate that eight states have extensive, highly active soils and nine others have sufficient distribution and content to be considered serious. An additional 10 to 12 states have problems that are generally viewed as relatively limited. As a rule, the 17 "problem" states have soil containing montmorillonite, which is, of course, the most expansive clay mineral. The 10 to 12 states with so-called limited problems (represented by the lighter shading) gener-

FIGURE 6A.208 Modification of highly plastic clay with lime in Vietnam in 1967, illustrating the reduction in plasticity and agglomeration of particles into clods (courtesy of Dr. Nathaniel S. Fox, Geopier Foundation Company, Scottsdale, Arizona).

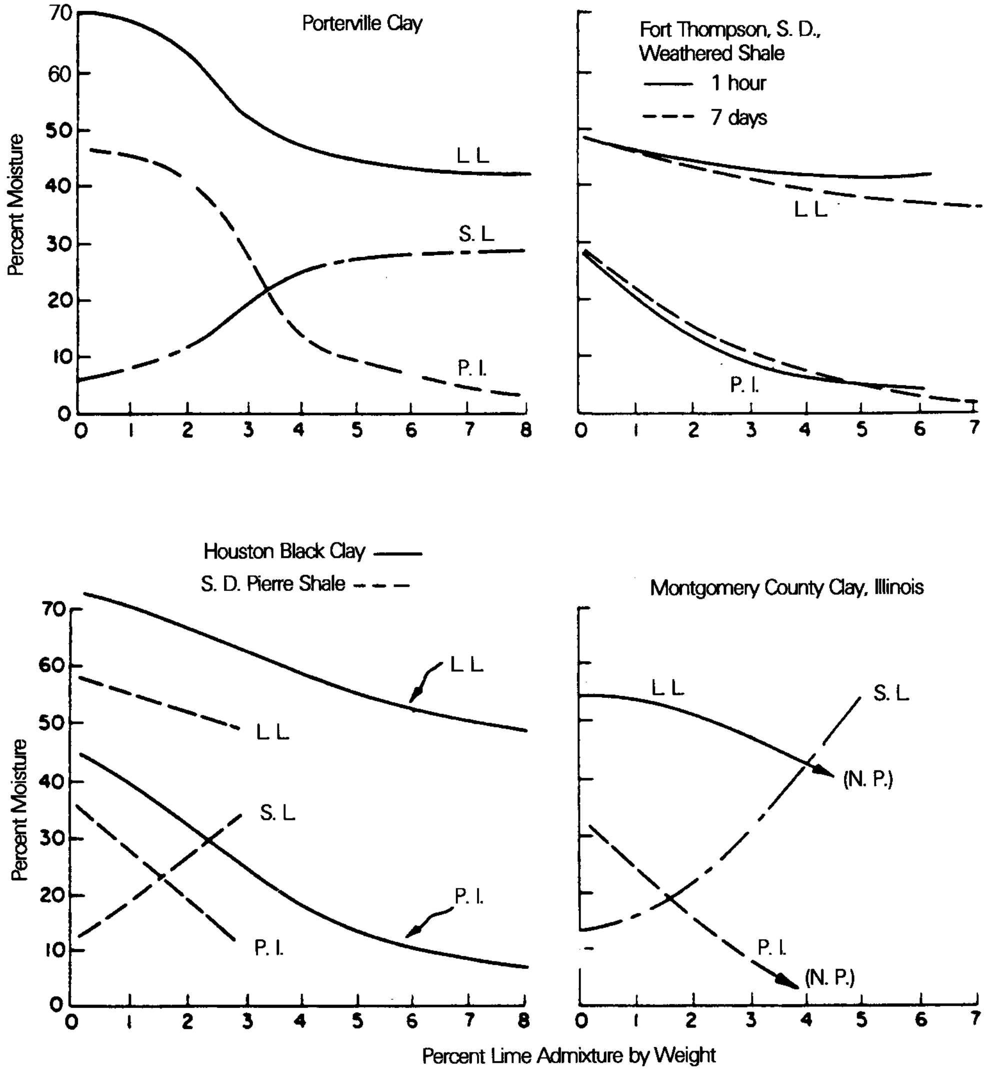
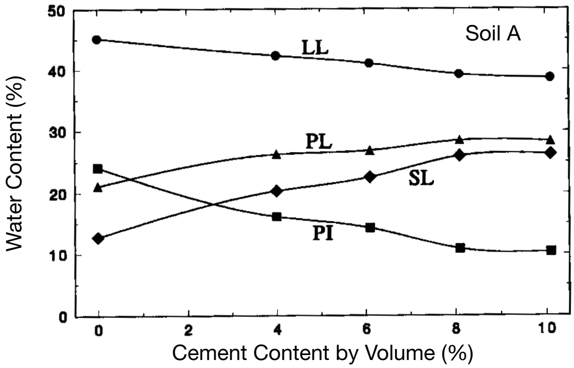


FIGURE 6A.209 Effect of lime on the plasticity characteristics of montmorillonitic clays (from TRB, 1987; after Holtz, 1969).

ally have soils which contain clays of lesser volatility such as illite and/or attapulgite, or montmorillonite in lesser abundance.

Chemical stabilization is the most effective and permanent method available for controlling the swelling potential of expansive soils. In new construction where controlled, intimate mixing with the expansive soil is feasible, hydrated lime [$Ca(OH)_2$] is the most commonly used stabilizer for controlling swelling. Data illustrating the effectiveness of lime in reducing the swelling potential of a number of soils are given in Table 6A.23. Other stabilizers such as cement, fly ash, lime–fly ash,



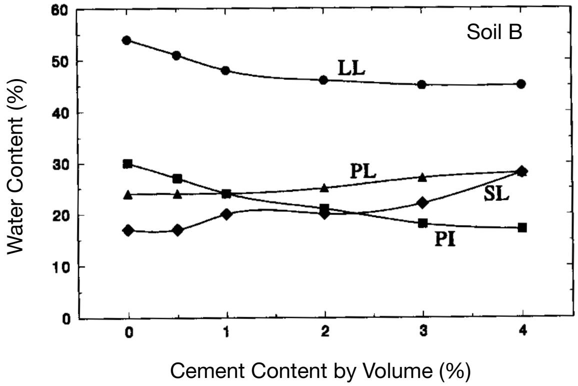


FIGURE 6A.210 Effect of cement content on the consistency limits of two cohesive soils (data from PCA, 1971).

and organic chemicals can also be used but are generally more expensive than lime. In remedial applications, such as stabilizing the soil beneath an existing structure, lime has inherent shortcomings. Lime is difficult to introduce into the soil matrix with any degree of uniformity, penetration, or saturation largely because (*a*) lime is sparsely soluble in water and (*b*) the expansive soil needing stabilization is both impermeable and heterogeneous. In addition, the presence of sulfates or organics in the soil may produce detrimental side effects when lime, cement, fly ash, or lime–fly ash are used (see Sees. 6A.7.1.2, 6A.7.1.3, and 6A.7.1.4).

For the reasons cited in the previous paragraph, organic chemicals are the preferred stabilizer for remedial applications. As early as 1965, certain surface-active organic chemicals were evaluated and utilized with some degree of success. One chemical used in the late 1960s and early 1970s was unquestionably successful in stabilizing the swell potential of montmorillonitic clay. The chemical was relatively inexpensive and easily introduced into the soil. However, the product maintained a "nearly permanent" offensive aroma which 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, and so on, generally blended and containing surface-active agents to enhance soil penetration (Williams

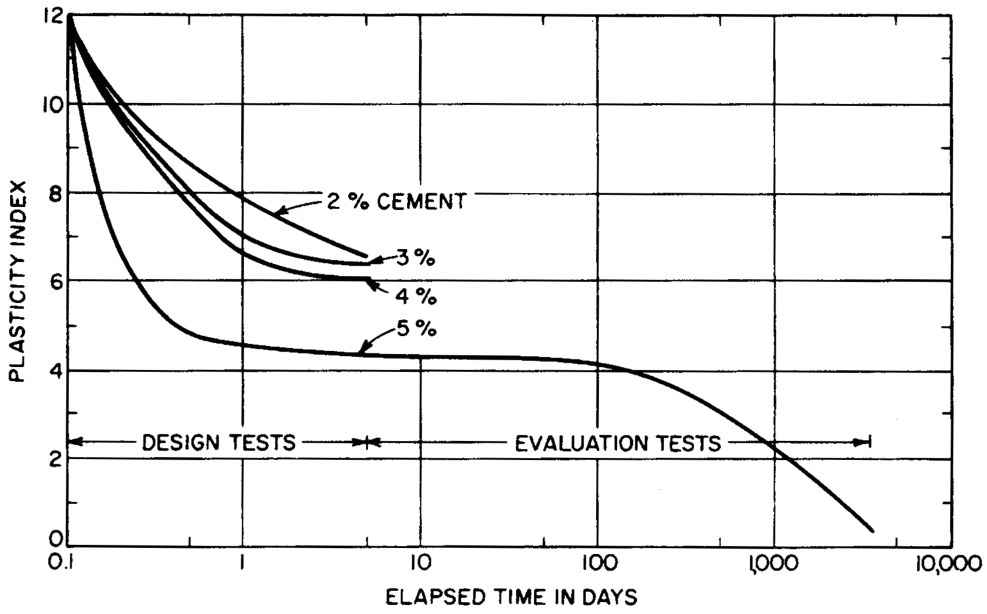


FIGURE 6A.211 Influence of aging on the plasticity index of cement-treated soil mixes from Hot Springs, Arkansas (from Redus, 1958).

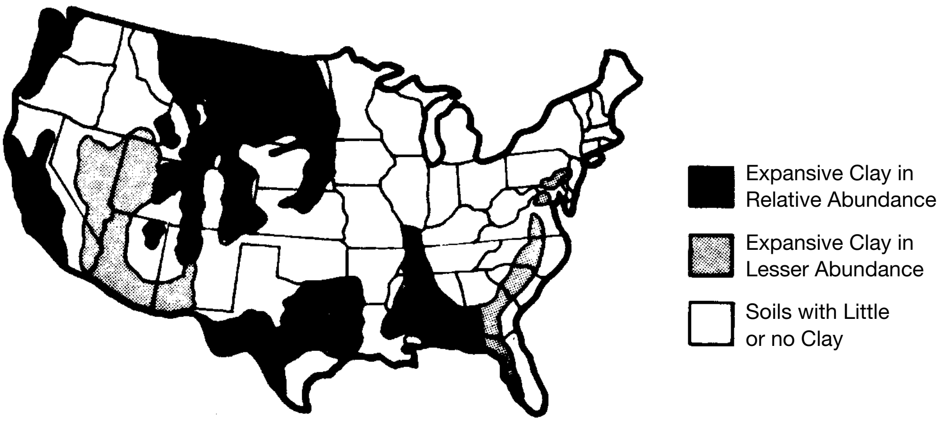


FIGURE 6A.212 Areas in the United States with expansive soils (from Godfrey, 1978).

TABLE 6A.23 Values of Swell for Selected Soils and Soil–Lime Mixtures (Data from Thompson, 1969)

Name of soil	Swell, %			Lime content, %
	Natural soil	Soil-lime mixtures: No curing*	Soil-lime mixtures: 48-h curing at 120°F (48.9°C)	
Accretion Gley 2	2.1	0.1	0.0	5
Accretion Gley 3	1.4	0.0	0.1	5
Bryce B	5.6	0.2	0.0	3
Champaign Co. till	0.2	0.5	0.1	3
Cisne B	0.1	0.1	0.1	5
Cowden B	1.4	—	0.0	3
Cowden B	2.9	0.1	0.1	5
Cowden C	0.8	0.0	0.0	3
Darwin B	8.8	1.9	0.1	5
East St. Louis clay	7.4	2.0	0.1	5
Fayette C	0.0	0.0	0.1	5
Illinoian B	1.8	0.0	0.0	3
Illinoian till	0.3	0.1	0.0	3
Illinoian till	0.3	0.9	0.1	3
Sable B	4.2	0.2	0.0	3
Fayette B	1.1	0.0	0.0	3
Miami B	0.8	0.0	0.0	3
Tama B	2.0	0.2	0.1	3

*Specimens were placed in 96-h soak immediately after compaction.

and Undercover, 1980). Certain combinations of chemicals seemed to be synergistic in behavior (the combined product achieved superior results to those noted for any of its constituents). Such a product is Soil Sta (proprietary to Robert Brown); similar laboratory and field data are not publicly available for any other organic stabilizer. 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 others wishing to evaluate an organic chemical stabilizer. All descriptive information was supplied through the courtesy of Brown Foundation Repair & 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.

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 had also indicated that soils exhibiting LL less than 35 or PI less than 23 (montmorillonite content less than about 10% by weight) would not swell appreciably (Sherif et al., 1982). Hence, the soils used in the laboratory tests and field applications contained at least 10% montmorillonite.

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

- Reduced the free swell potential of montmorillonitic clay (see Fig. 6A.213)
- Appeared stable to repeated cycles of weathering (a simulated period of 50 years)
- Increased shear strengths in some soils by 2-fold

- Increased soil permeabilities up to 40-fold
- Reduced soil shrinkage by amounts varying from 11 to 50% (Petry and Brown, 1987; Brown, 1989)

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, and, in a goodly proportion of those, there was a serious question as to the cause.

Some interesting points are illustrated in Fig. 6A.213:

1. At moisture contents above about 10.5%, the chemical treatment of these particular montmorillonitic soils reduced the free swell potential to levels often considered as "tolerable." Free swell potential was essentially nil above in situ moisture contents of about 18%.
2. The untreated soil is somewhat moisture-stabilized (against swell) at natural moisture contents above about 21% and would be totally stabilized against swell at natural moisture contents above about 27%.
3. The PL of this soil was 30%; hence, the moisture stabilization (against swell) occurs several percentage points below the PL.
4. Even at natural moisture contents below 10.5%, the effective abatement of swell is still profound, nearly fourfold. However, in the "real world," the natural soil moisture in foundation bearing soils (expansive in nature) is seldom found to be materially less than 10.5%.

Again, the principal intended function for this particular product (Soil Sta) is to abate swell. Under certain conditions, recurrent settlement could be a persistent problem. For example, Soil Sta is not designed to prevent soil shrinkage resulting from either soil moisture losses (evaporation or transpiration), compaction, or consolidation.

6A.7.2.3 Shrinkage

Successful chemical stabilization of expansive soils results in reduced shrinkage potential. This reduction in shrinkage potential is illustrated in Figs. 6A.209 and 6A.210 by the increase in the SL for the stabilized soils compared to the natural soils and by the data for linear shrinkage of three cement-stabilized cohesive soils provided in Fig. 6A.214. Natural granular (cohesionless) soils do not shrink because of a lack of clay content, but cement-stabilized granular soils will shrink because the cement shrinks owing to increased cohesion associated with the cementing action.

Shrinkage of properly stabilized soils is usually of little or no significance in most foundation applications. The magnitude of the shrinkage is generally so small that little or no settlement will occur in foundations bearing on chemically stabilized soils. For example, data from Nakayama and Handy (1965) showed that the total shrinkage was less than 1% for four soils (two sands, a silt, and a clay) treated with various levels of cement and tested under a variety of curing and drying conditions. The cement-stabilized sands shrank very little (<0.1%). Shrinkage may produce minor cracking of the stabilized soil, which may result in some small but probably insignificant reduction in strength. Perhaps the most important potential problem related to shrinkage-induced cracking is that it may subject the stabilized soil to greater attack from sulfates that are leached from adjacent sulfate-bearing soils by providing easier access into the interior of the stabilized mass.

6A.7.2.4 Collapse

The preconstruction collapse potential of both naturally deposited and compacted soils can be substantially reduced or eliminated by chemical stabilization. Naturally deposited collapsible soils can be chemically treated either by adding the chemical in an aqueous solution to the soil via either injection or ponding or by excavating the soil, mixing lime or cement with the soil at an appropriate water content, and recompacting the soil in place (Clemence and Finbarr, 1981). The collapse potential of compacted fills can be controlled by mixing the stabilizer with the borrow soil prior to compaction (Lawton et al., 1993). To the authors' knowledge, there are no methods of chemical stabilization currently available for remedial applications in collapsible soils.

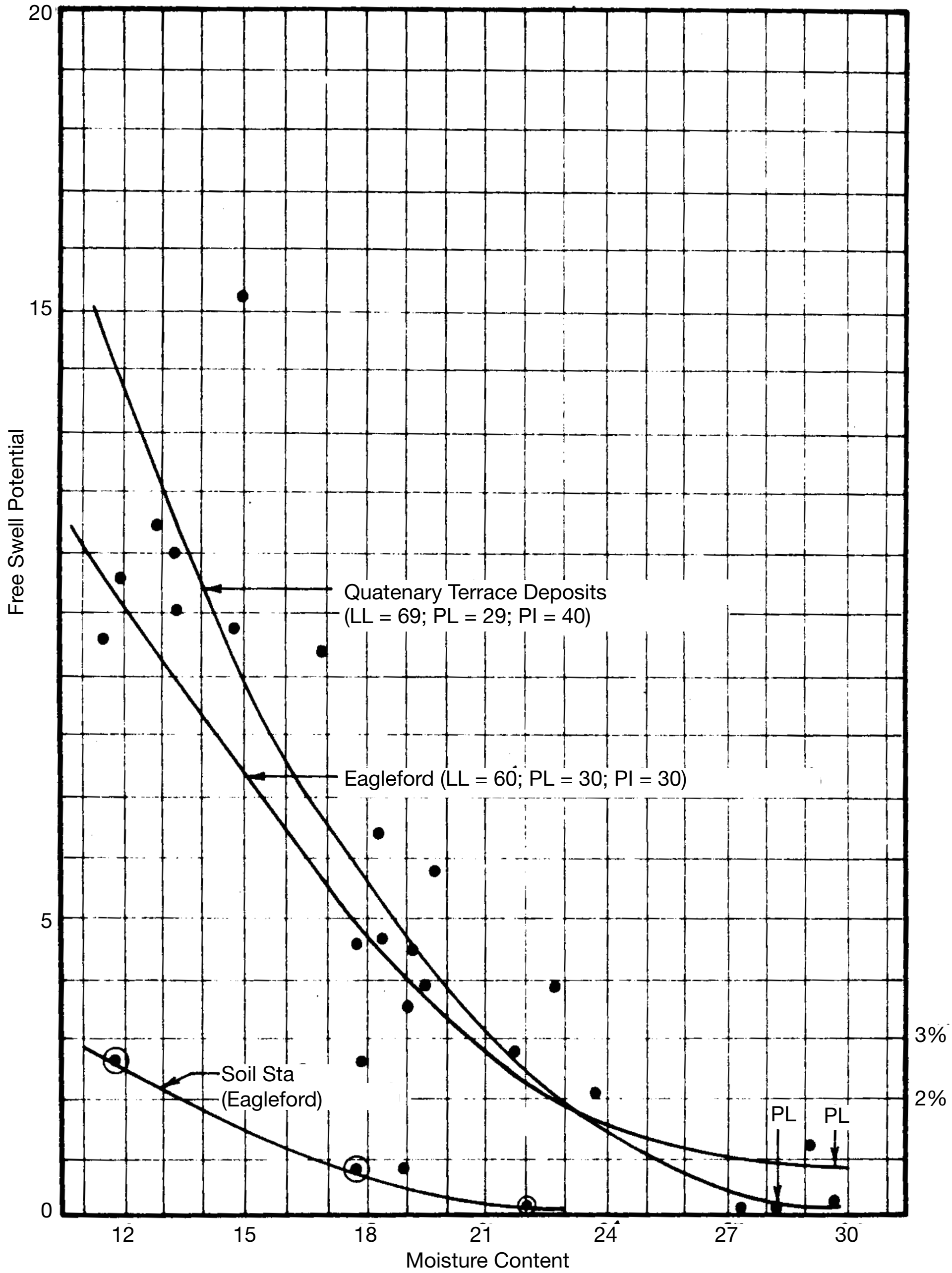


FIGURE 6A.213 Effectiveness of Soil Sta in reducing the free swell potential of montmorillonitic clays (from Brown, 1992).

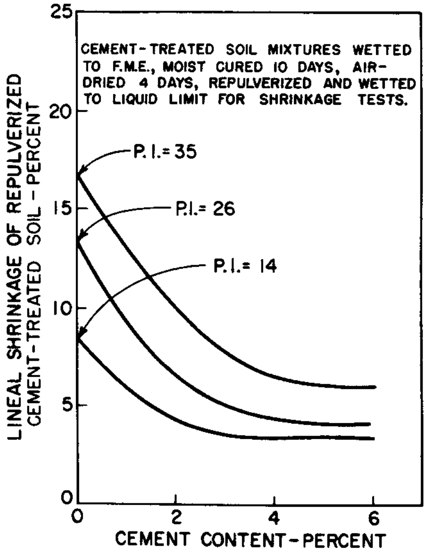


FIGURE 6A.214 Influence of cement content on the linear shrinkage of repulverized cement-treated soil (from HRB, 1961).

The stabilization of naturally deposited collapsible soils using sodium silicate has been demonstrated by laboratory and field tests conducted in the former Soviet Union and in the United States. Sokolovich and Semkin (1984) reported the results of laboratory collapse tests conducted on specimens of loess that had been prewetted in the field by injecting either water or a 2% solution of sodium silicate in water. The prewetted soil was allowed to dry for several months before samples were taken from the field at depths of 4 m and 6 m (13 ft and 20 ft). Upon wetting in the laboratory (applied stress not given), the specimens prewetted with water collapsed an average of 4.1%, whereas those prewetted with the sodium silicate solution collapsed an average of 0.26%. Rollins and Rogers (1994) conducted a series of full-scale field load tests to evaluate the effectiveness of different treatment methods in reducing the collapse potential of a deposit of clayey sandy silt located in an alluvial fan. One of the test sections was prewetted with a 2% sodium silicate solution that was ponded on the ground surface and allowed to percolate into the ground. After periods of 1.5 months for curing and drying and two weeks for sampling, a 1.5 m (4.9 ft) square reinforced concrete footing was built and loaded to an average bearing stress of 85 kPa (1.8 ksf). The site was then wetted with water. The resulting collapse settlement was less than 27 mm (1.1 in), compared to an average settlement of 280 mm (11 in) for the untreated soil. Furthermore, the treatment with sodium silicate reduced the creep settlement after 6 months from 12 mm (0.47 in) to 9 mm (0.35 in).

Laboratory results provided by Lawton and coworkers (1993) have demonstrated the effectiveness of lime and cement in substantially reducing the collapse potential of compacted cohesive soils. One-dimensional collapse tests were conducted on comparable untreated and treated specimens of compacted clayey sand wherein the dry density and water content of the soil exclusive of the stabilizer were the same. By comparing the results of the collapse tests on these comparable untreated and treated specimens, which are provided in Table 6A.24, the fundamental influence of the stabilizer on the collapse potential can be determined. In all instances where there was sufficient free water in the soil for complete hydration of the stabilizer (a compaction water content of at least 8.8% in these tests), the reduction in collapse was significant, and the magnitude of collapse was

TABLE 6A.24 Results from Single Oedometer Collapse Tests on Comparable Untreated and Chemically Stabilized Specimens Consisting of 70% Silica Sand and 30% Kaolin (from Lawton et al., 1993)*

	γ_d*‡				σ_v		
Stabilizer†	Mg/m³	pcf	w‡, %	S_r‡, %	kPa	ksf	ε_{vw}, %
None	1.50	94	8.8	30	400	8.4	–17.04
5% cement	1.50	94	8.8	30	400	8.4	–0.02
None	1.60	100	12.5	50	400	8.4	–0.60
2% cement	1.60	100	12.5	50	400	8.4	–0.08
None	1.60	100	7.5	30	400	8.4	–13.47
5% lime	1.60	100	7.5	30	400	8.4	–4.04
None	1.67	104	10.0	45	400	8.4	–0.31
0.5% cement	1.67	104	10.0	45	400	8.4	–0.01
None	1.67	104	10.0	45	800	16.7	–0.06
0.5% cement	1.67	104	10.0	45	800	16.7	–0.01
None	1.67	104	7.0	31	400	8.4	–13.06
2% cement	1.67§	104§	7.0§	31§	400	8.4	–13.51
None	1.67	104	7.0	31	800	16.7	–11.28
2% cement	1.67	104	7.0	31	800	16.7	–9.56

*γ_d = dry density of soil (exclusive of stabilizer); w = compaction water content of soil (exclusive of stabilizer); S_r = degree of saturation of soil (exclusive of stabilizer); σ_v = applied (total) vertical stress; ε_{vw} = wetting-induced strain.

†Cement = Portland cement; lime hydrated lime [$Ca(OH)_2$].

‡Values given are nominal. Actual values for all specimens, except as noted below, were close to their nominal values.

§Actual γ_d = 1.60 Mg/m³ = 100 pcf; actual w = 6.4%; actual S_r = 25%

small (≤0.08%). The most dramatic example of reduction in collapse was from 17.04 to 0.02% by the addition of 5% cement (by weight) at a dry density of 1.50 Mg/m³ (93.6 pcf), a water content of 8.8%, and an applied stress of 400 kPa (8.4 ksf). For a compaction water content ≤7.5%, the hydration of the stabilizer was incomplete, and, although collapse was generally reduced, the magnitude of collapse was still quite high (≥4.04%) and unacceptable from a design standpoint.

The addition of chemical stabilizers also substantially reduced the time to the end of collapse (t_c) as illustrated in Fig. 6A.215. For γ_d = 1.60 Mg/m³ (100 pcf), w = 7.5%, and σ_v = 400 kPa (8.4 ksf), t_c for the 5% lime specimen was 4 min compared to 60 min for the untreated specimen. Similarly, for γ_d = 1.67 Mg/m³ (104 pcf), w = 7.0%, and σ_v = 800 kPa (16.7 ksf), the addition of 2% cement reduced t_c from 240 min to 60 min. Similar reductions in t_c occurred in the other treated specimens.

The chemically treated soils also exhibited significantly less postcollapse creep strain than the comparable untreated soils. For example, for the same specimens discussed in the preceding paragraph, the modified secondary compression index [$C_{\alpha\varepsilon} = -\Delta\varepsilon_v/\Delta(\log t)$] decreased from 0.094 to 0.025% for the lime-treated specimen and decreased from 0.36 to 0.014% for the cement-treated specimen. This trend was consistent in all treated specimens.

6A.7.2.5 Stress-Strain-Strength Behavior

The general effect of chemical stabilization on soils is to increase both the strength and stiffness of the soil while making the soil more brittle. Where the stabilization occurs primarily by cation and base exchange with clay particles, the changes in these characteristics are generally modest. For example, as noted in Sec. 6A.7.2.2, the increases in shear strength of montinorillonitic soils produced

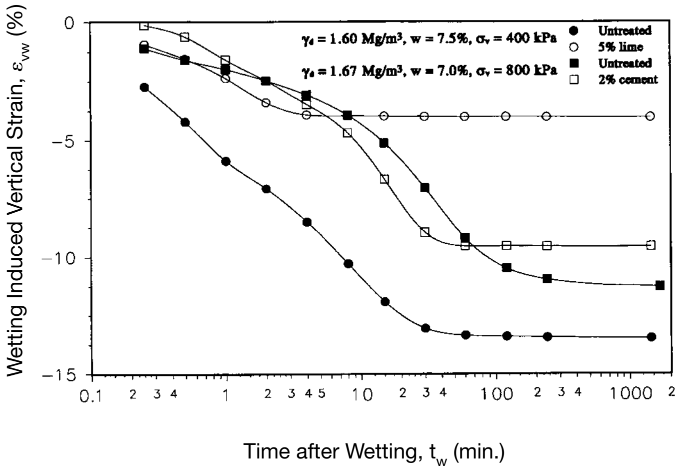


FIGURE 6A.215 Effect of lime and cement on the time rate of collapse of compacted specimens consisting of 70% silica sand and 30% kaolin (from Lawton et al., 1993).

by Soil Sta, an organic stabilizer designed to control swelling potential, are typically a maximum of about 100% (that is, it doubles the shear strength of the natural soil). In contrast, cementitious or pozzolanic stabilizers may produce 50-fold or greater increases in stiffness and strength. To illustrate the substantial changes in stiffness and strength that may occur from chemical stabilization, the results of triaxial compression tests on untreated and cement-treated specimens of a micaceous silty sand are presented in Fig. 6A.216. At a confining pressure (σ_3) of 30 psi (207 kPa), the maximum deviator stress increased from 78.7 psi (543 kPa) for the untreated soil to 971 psi (6.70 MPa) for the same soil treated with 10% cement (by dry weight), an increase of more than 12-fold. The stiffness

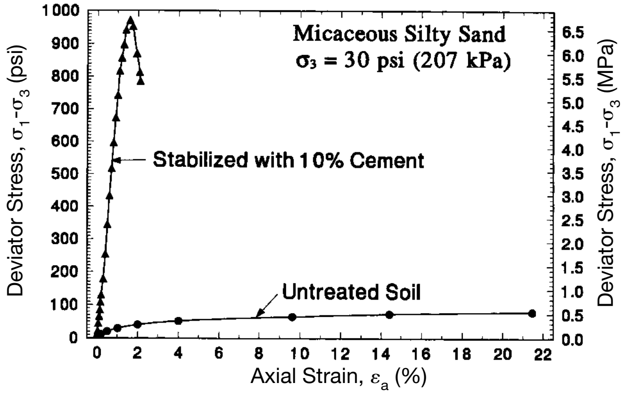


FIGURE 6A.216 Influence of cement stabilization on the triaxial stress-strain characteristics of a micaceous silty sand.

of the cemented-treated specimen, as measured by the secant modulus at 50% of the maximum deviator stress (E_{50}), was about 37 times that of the natural soil: 73.9 ksi (510 MPa) for the cement-stabilized soil compared to 2.0 ksi (13.8 MPa) for the untreated soil. Note also that the stress-strain behavior changed from ductile for the untreated soil to brittle for the cement-treated soil. The cement stabilization also produced substantial increases in the strength parameters. For σ_3 ranging from 5 to 30 psi (34 to 207 kPa), the friction angle (ϕ) increased from 33° to 64°, and the cohesion intercept increased from 1.4 psi (9.7 kPa) to 60 psi (410 kPa).

In general, an increase in stabilizer content will produce an additional increase in stiffness and strength so long as the additional stabilizer produces further cementitious or pozzolanic reactions in the soil. Thus, cement-stabilized soils will continue to increase in strength and stiffness with increased cement content so long as there is sufficient water for hydration of the cement (up to the point where the cement constitutes a vast majority of the material and the strength of the mixtures is essentially equal to the strength of the cement). In contrast, the maximum strength and stiffness of lime-treated soils is achieved at a specific lime content that depends primarily on the amount of clay available in the soil for pozzolanic reactions. The addition of lime beyond this point produces no further pozzolanic reactions and may result in a decrease in strength and stiffness. The minimum lime content that produces the maximum benefit in terms of strength and stiffness is about 8% for most clayey soils (Ingles and Metcalf, 1973). Typical characteristics of compressive strength versus stabilizer content for both cement and lime are given in Fig. 6A.217.

Because the cementitious and pozzolanic reactions are time-dependent, the strength and stiffness of stabilized soils generally increases with time for at least several years. Typical results showing the effect of aging on the strength and stiffness of cement-stabilized soils are provided tn Figs. 6A.218 and 6A.219. Aging can have a significant effect on both strength and stiffness. For example, for Soil 2 treated with 14% cement in Figs. 6A.218 and 6A.219, the unconfined compressive strength increased from 1400 psi (9650 kPa) after 7 days of curing to 2300 psi (15,900 kPa) after 90 days of curing, while the compressive secant modulus at one-third of the ultimate load increased from 2000 ksi (13,800 MPa) to 2,800 ksi (19,300 MPa) during the same period of time. Similar effects of aging on the stress-strain-strength characteristics of a lime-treated silty clay are shown in Fig. 6A.220. That the increase in strength and stiffness may occur over long periods of aging is demonstrated in Fig. 6A.221 for two cement-treated soils.

When chemical stabilizers are mixed with the soil, a delay between the time of mixing and the time of compaction may result in a substantial decrease in strength and stiffness of the treated soil. Thus, many specifications require that compaction be completed within a short time after mixing, usually less than 1 or 2 h. A delay in compaction reduces the strength and stiffness in two ways:

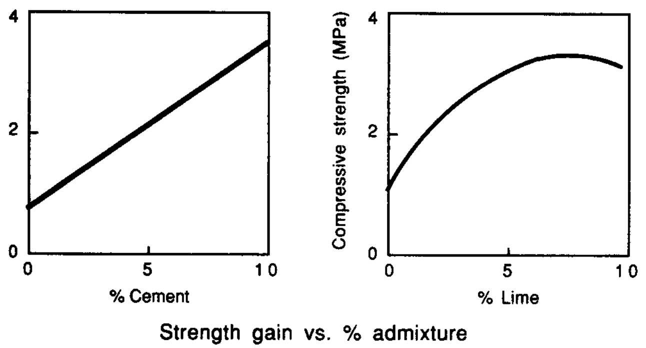


FIGURE 6A.217 Typical characteristics of gain in strength versus admixture content for cement and lime (from Hausmann, 1990).

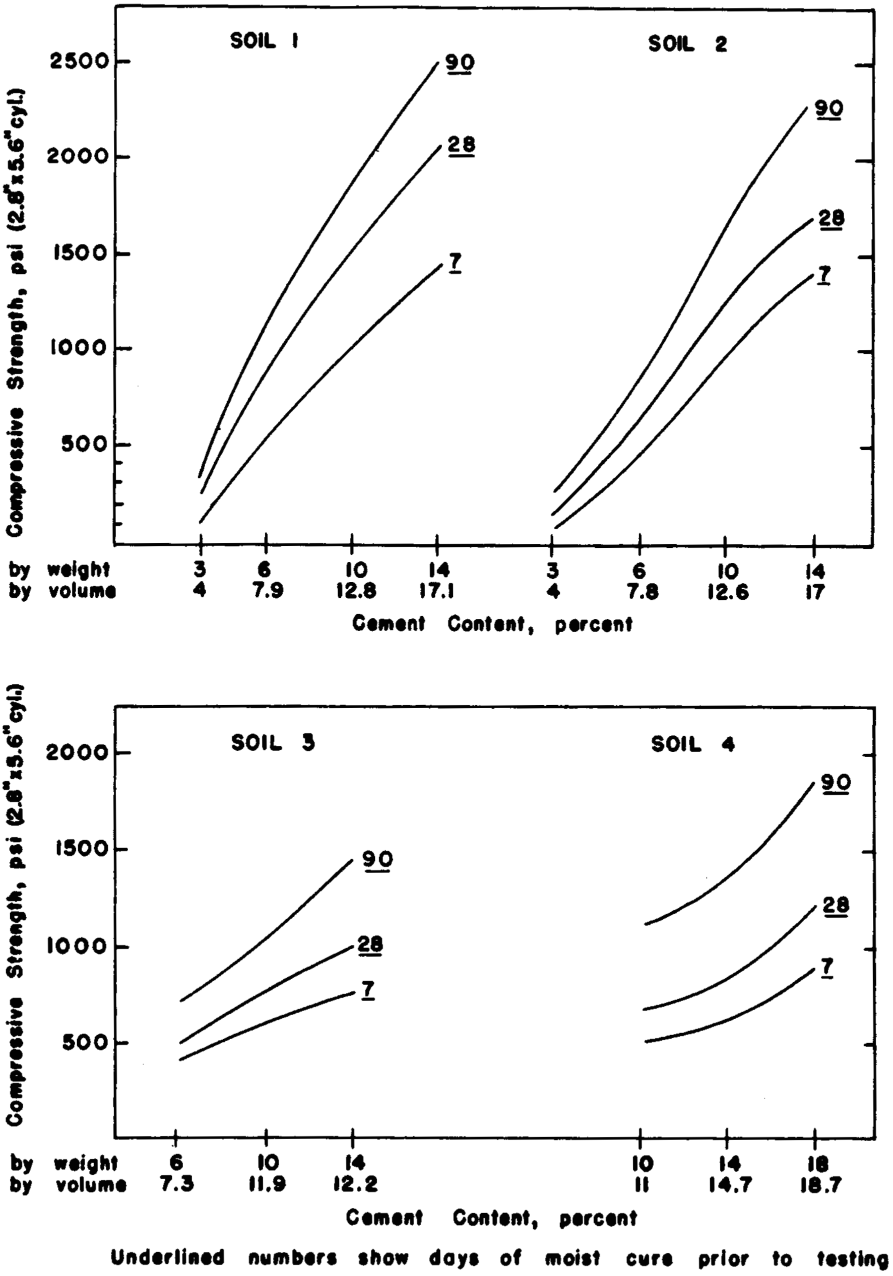


FIGURE 6A.218 Influence of aging and cement content on unconfined compressive strength (from HRB, 1961; after Felt and Abrams, 1957).

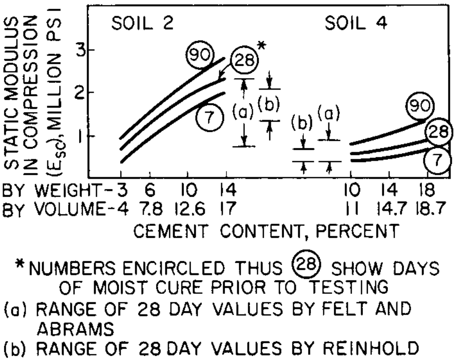


FIGURE 6A.219 Effect of aging and cement content on static unconfined compressive modulus (from HRB, 1961; data from Felt and Abrams, 1957).

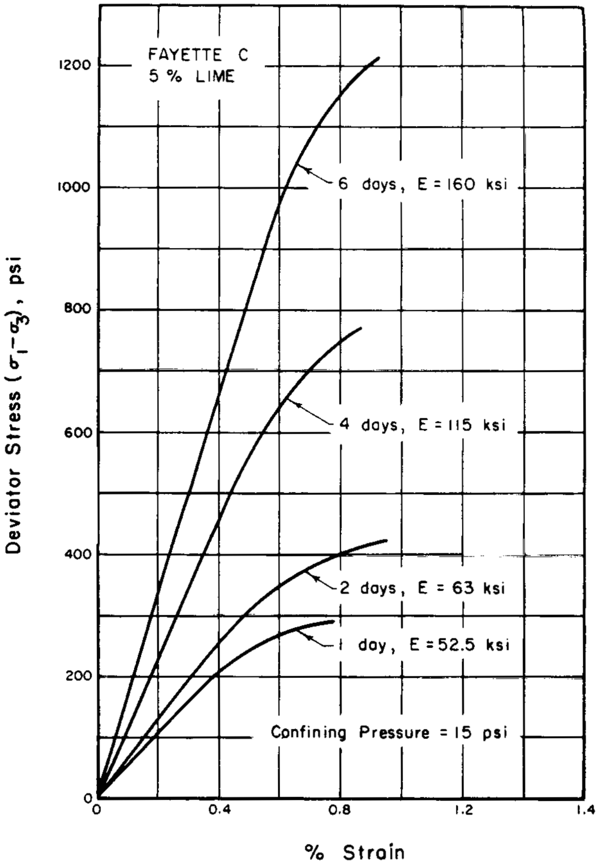


FIGURE 6A.220 Influence of aging on the stress-strain properties of a lime-treated cohesive soil (from Thompson, 1966).

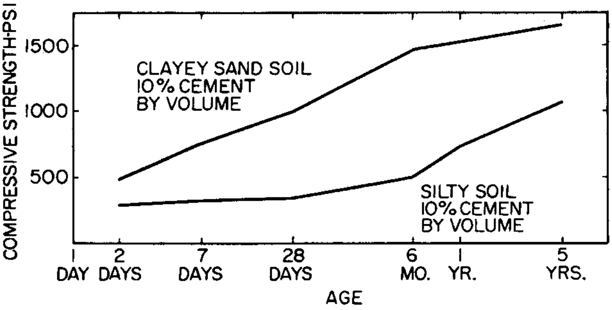


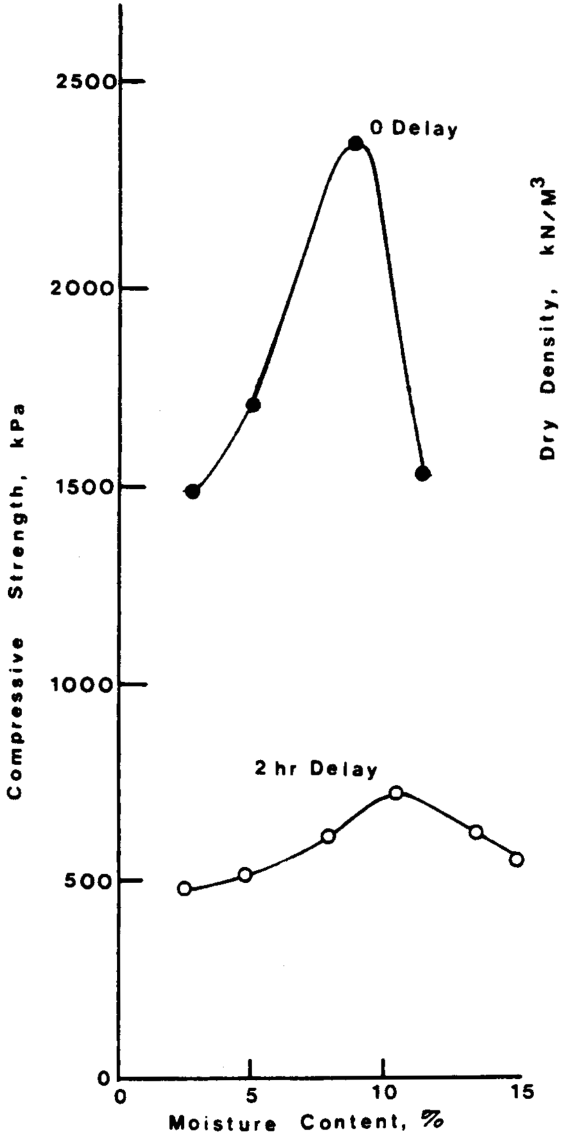
FIGURE 6A.221 Influence of long-term aging on the unconfined compressive strength of two specimens of soil-cement (from HRB, 1961; after Leadabrand, 1956).

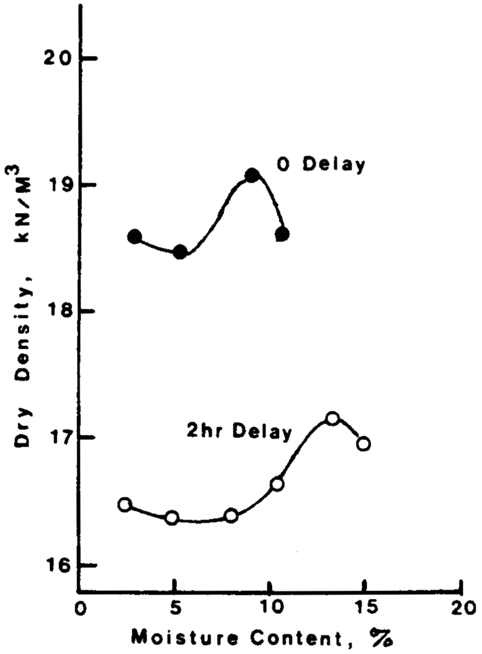
1. Cementitious products (clods) are formed that harden with time. These clods must be broken down or remolded if the material is to be densified. For the same compactive effort, the density will decrease with increasing delay in compaction, as shown in Fig. 6A.222 for a soil stabilized with self-cementing fly ash.
2. As clods form and harden with increasing delay, the effective area of contact between clods in the compacted soil decreases. The result is that a smaller volume of stabilized soil is cemented together because cementation between clods occurs only along the interclod contact areas. In contrast, a well-mixed, chemically treated soil that is compacted immediately after compaction will be relatively homogeneous with cementation occurring throughout the treated mass. Thus, a delay in compaction means that a portion of the stabilizer is lost to the clodforming process and is unavailable for cementation of the entire mass.

The loss in strength resulting from a delay in compaction can be significant. For example, a 2-h delay in compaction of the fly-ash-treated material shown in Fig. 6A.222 reduced the maximum strength for a given compactive effort from about 2300 kPa (330 psi) to about 700 kPa (100 psi)—a decrease of 70%. This magnitude of reduction in strength is typical for class C (cementitious) fly ashes, which "flash-set" when water is added. To counteract this characteristic, set retarders often must be added to the mixture to delay the cementitious reactions for about 2 h to permit hauling, placing, and compaction of the treated material (ASCE, 1987). A delay in compaction may also be a significant problem in cement-treated soils, as illustrated in Fig. 6A.223, but is generally not a significant problem in soils treated with pozzolanic stabilizers (typically lime and lime–fly ash).

Temperature affects the rate at which the cementitious or pozzolanic reactions take place and thus the strength and stiffness of the soil. In general, higher temperatures produce faster gains in strength and stiffness, as illustrated in Fig. 6A.224 for various soils treated with lime and cement, and in Fig. 6A.225 for an LFA mixture. At temperatures below about 40°F (4°C), the cementitious or pozzolanic reactions are severely retarded. Thus, during the winter, the cementitious reactions will be minimal when temperatures drop below 40°F (4°C) but will resume when the weather warms, as illustrated in Fig. 6A.226. Low temperatures, including long periods of freezing, seem to have no permanent effect on the strength that will eventually be achieved.

Some typical guidelines for estimating the stress-strain-strength relationships of soils stabilized with lime, cement, and fly ash have been developed. The typical shape of a stress-strain plot for these types of chemically treated soils is as shown in Fig. 6A.227. In general, the stress-strain relationship is approximately linear up to about 50 to 70% of the peak strength, beyond which the relationship is nonlinear (strain-softening) up to failure.





Pulverized Chip Seal Pavement Treated with 18% LaCygne Fly Ash	
Maximum Density	Opt. Moisture
19.0 kN/M^3 - 0 Delay	9.0%
17.1 kN/M^3 - 2 Hr. Delay	13.5%
Maximum Compressive Strength	Opt. Moisture
2410 kPa - 0 Delay	8.5%
720 kPa - 2 Hr. Delay	10.5%

FIGURE 6A.222 Moisture-density and moisture-strength relationships for a fly-ash-treated material (from Ferguson, 1993).

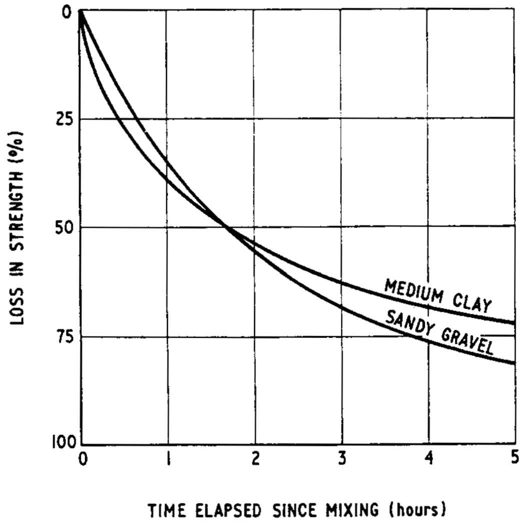


FIGURE 6A.223 Loss in strength caused by delay in compaction for two soils stabilized with 10% cement (standard Proctor compaction) (from Ingles and Metcalf, 1973; after West, 1959)

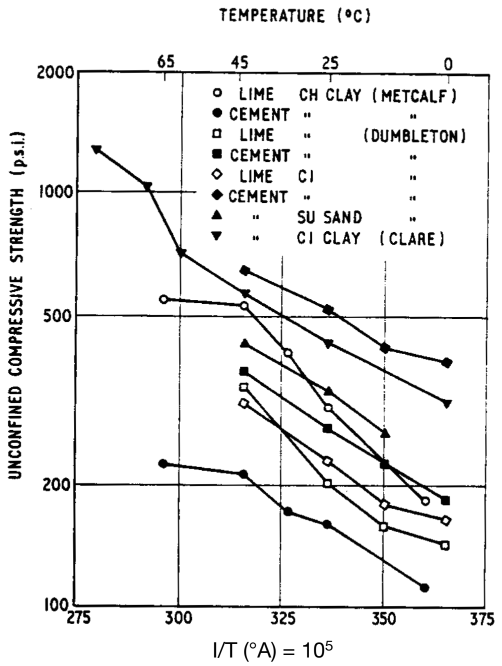


FIGURE 6A.224 Relationship between the rate of gain in strength and curing temperature for lime and cement-stabilized soils (from Ingles and Metcalf, 1973).

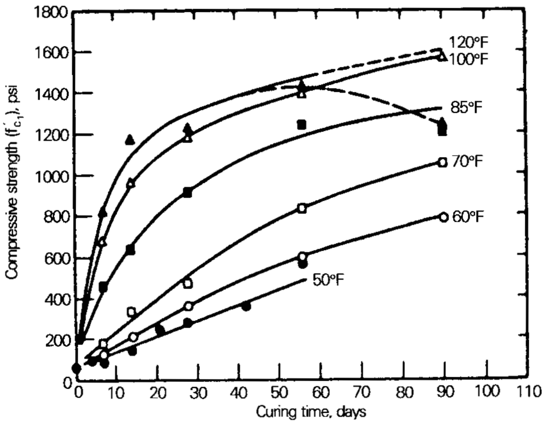


FIGURE 6A.225 Effect of curing time and temperature on the strength development of a lime–fly ash-aggregate mixture (from NCHRP, 1976; after MacMurdo and Barenberg, 1973).

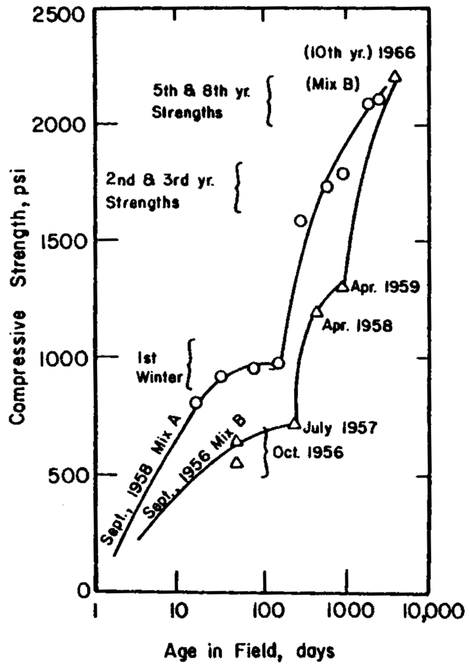


FIGURE 6A.226 Age–strength relationships of lime–fly ash-aggregate mixtures (from Little et al., 1987; after FHWA, 1979).

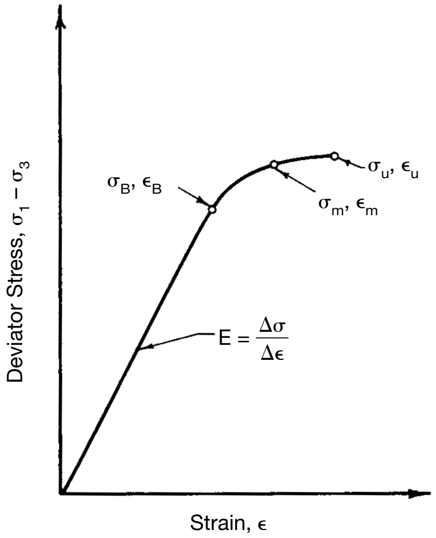


FIGURE 6A.227 Idealized stress-strain curve for lime-stabilized soils (from Thompson, 1966).

Typical relationships between unconfined compressive strength (q_u) and stabilizer content have been established for cement-stabilized soils. Data illustrating these relationships are provided in Fig. 6A.228, from which the following approximate equations have been developed for estimating q_u based on cement content (C in percent by dry weight) for coarse-grained and fine-grained soils. For coarse-grained soils, the equations are

$$q_u(\text{psi}) = (80 \text{ to } 150) \cdot C \tag{6A.133a}$$

$$q_u(\text{kPa})\ (550 \text{ to } 1030) \cdot C \tag{6A.133b}$$

For fine-grained soils the equations are

$$q_u(\text{psi}) = (40 \text{ to } 80) \cdot C \tag{6A.134a}$$

$$q_u(\text{kPa}) = (275 \text{ to } 550) \cdot C \tag{6A.134b}$$

Note that Eqs. (6A.133) and (6A.134)-as well as the rest of the equations given in this section—should be used only to estimate values, not to obtain values for design or analysis.

Typical values of q_u for lime-fly-ash-stabilized materials are given in Table 6A.25 according to general soil type. An equation for estimating q_u for cement-stabilized soils at a later age (d = number of days) based on a known value of q_u at a certain age (d_0) is given as follows (Little et al., 1987; Mitchell, 1981):

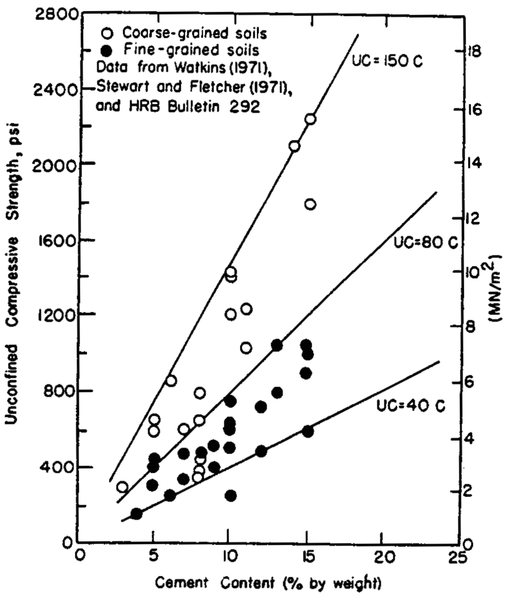


FIGURE 6A.228 Relationship between cement content and unconfined compressive strength for soil-cement mixtures (from Little et al., 1987; after FHWA, 1979).

$$(q_u)_d = (q_u)_{d_0} + F \cdot \log\left(\frac{d}{d_0}\right) \tag{6A.135}$$

where $F = 70C$ for coarse-grained soils and q_u in psi
$F = 480C$ for coarse-grained soils and q_u in kPa
$F = 10C$ for fine-grained soils and q_u in psi
$F = 70C$ for fine-grained soils and q_u in kPa

The stiffness and strength of a chemically stabilized soil are obviously interrelated. The following equation gives an approximate relationship between compressive modulus (E_c) at a confining

TABLE 6A.25 Ranges of unconfined compressive strength for lime-fly-ash-stabilized materials (from NCHRP, 1976).

Type of soil	28-day immersed unconfined compressive strength	
	psi	kPa
Gravels	400 to 1300	2800 to 9000
Sands	300 to 700	2100 to 4800
Silts	300 to 700	2100 to 4800
Clays	200 to 500	1400 to 3400
Crushed stones and slag	1400 to 2000	10,000 to 14,000

pressure of 15 psi (100 kPa) and unconfined compressive strength for lime-treated soils (Thompson, 1966):

$$E_c(\text{ksi}) = 10 + 0.124 \cdot q_u(\text{psi}) \tag{6A.136a}$$

$$E_c(\text{MPa}) = 70 + 0.124 \cdot q_u(\text{kPa}) \tag{6A.136b}$$

Values of E_c for cement-stabilized soils are typically within the range of 1000 to 5000 ksi (7000 to 35,000 MPa) for coarse-grained soils and 100 to 1000 ksi (700 to 7000 MPa) for fine-grained soils. E_c depends on the confining pressure and can be expressed in the following form based on an assumed hyperbolic shape of the stress-strain relationship (Duncan et al., 1980):

$$E_{tc} = K \cdot P_a \cdot \left(\frac{\sigma_3}{P_a}\right)^n \cdot \left[1 - \frac{0.75(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2(c \cdot \cos\phi + \sigma_3 \cdot \sin\phi)}\right]^2 \tag{6A.137}$$

where E_{tc} = tangent compressive modulus at a given stress level $(\sigma_1 - \sigma_3)$
ϕ = angle of internal friction
c = cohesion intercept
P_a = atmospheric pressure
n = modulus exponent (typically 0.1 to 0.5 for cement-stabilized soils)
K = modulus number (typically 1000 to 10,000 for cement-stabilized soils)

E_c for lime-fly-ash-stabilized soils generally varies from about 500 to 2500 ksi (3500 to 17,500 MPa) (Little et al., 1987).

Values of the Mohr-Coulomb strength parameters, angle of internal friction (ϕ) and cohesion intercept (c), vary considerably for chemically stabilized soils depending primarily upon the type of soil, the amount of stabilizer, and the amount and type of curing. Typical values of ϕ for soils treated with cement, lime, and lime–fly ash are given in Table 6A.26. For cement-stabilized soil, c may be as high as a few hundred psi (a few thousand kPa) (FHWA, 1979; Mitchell, 1981). The following equation can be used to estimate c based on q_u for lime-stabilized soils (Thompson, 1966):

$$c\ (\text{psi}) = 9.3 + 0.292 \cdot q_u\ (\text{psi}) \tag{6A.138a}$$

$$c\ (\text{kPa}) = 64 + 0.292 \cdot q_u\ (\text{kPa}) \tag{6A.138b}$$

For the lime–fly ash-stabilized gravels evaluated by Hollon and Marks (1962), c varied from 55 to 128 psi (380 to 880 kPa).

TABLE 6A.26 Typical Ranges of Values for Angle of Internal Friction for Soil Stabilized with Cement, Lime, and Lime–Fly Ash

	Typical values of ϕ (degrees)	
Stabilizer	Coarse-grained	Fine-grained
Cement	40 to 60	30 to 40
Lime	N.A.*	25 to 35
Lime–fly ash	49 to 53†	N.A.*

*N.A. indicates not available or not applicable.
†Based on limited data for gravel (Hollon and Marks, 1962).

Although few data in the literature address values of Poisson's ratio (v) for chemically stabilized soils, some general guidelines can be given for soils stabilized with cement, lime, and fly ash. Values of static v for chemically stabilized soils are typically smaller than for natural soils. In general, v is relatively constant within the linear stress-strain range for a given stabilized soil and confining pressure, as shown in Fig. 6A.229 for a lime–fly ash-stabilized gravel. Values of v within this range of stresses typically vary from about 0.08 to 0.15 for granular soils and 0.15 to 0.20 for cohesive soils. Thus, these values are typical for the normal range of stresses that a stabilized soil would be subjected to during its service life. At higher stresses levels, v increases with increasing stress up to values at failure of about 0.20 to 0.30 for granular soils and 0.25 to 0.35 for cohesive soils. Values of dynamic Poisson's ratio appear to be somewhat higher. In a study of four soils stabilized with cement (Felt and Abrams, 1957), values of dynamic v were found to range from 0.22 to 0.27 for two granular soils, 0.24 to 0.31 for a silty soil, and 0.30 to 0.36 for a clayey soil.

6A.7.2.6 Compaction Moisture-Density Relationships

The addition of a chemical stabilizer usually alters the compaction moisture-density relationships of a soil. The moisture-density curves for a loess treated with varying amounts of quicklime compacted 24 h after mixing are shown in Fig. 6A.230. For nearly all cohesive soils, the addition of lime produces the same types of changes to the moisture-density relationships illustrated in Fig. 6A.230—a decrease in maximum dry density ($\gamma_{d\max}$) and an increase in optimum water content (w_{opt}) for the same method of compaction and compactive effort. In addition, $\gamma_{d\max}$ typically decreases and w_{opt} normally increases with increasing stabilizer content. These changes can be explained in simplified terms as follows:

1. Some cementitious or pozzolanic reactions occur immediately upon hydration. This cementing effect produces a resistance to compaction that results in a decrease in $\gamma_{d\max}$.
2. Extra water (in addition to that needed to facilitate rearrangement of the soil particles during compaction) is required to hydrate the lime and produce the pozzolanic reactions. Hence, w_{opt} is increased.

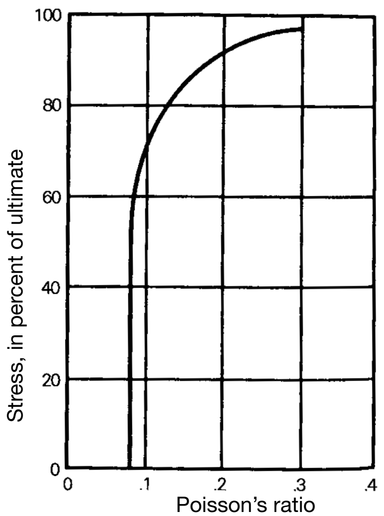


FIGURE 6A.229 Effect of stress level on Poisson's ratio for a lime–fly ash-gravel mixture (from NCHRP, 1976; after Ahlberg and Barenberg, 1965).

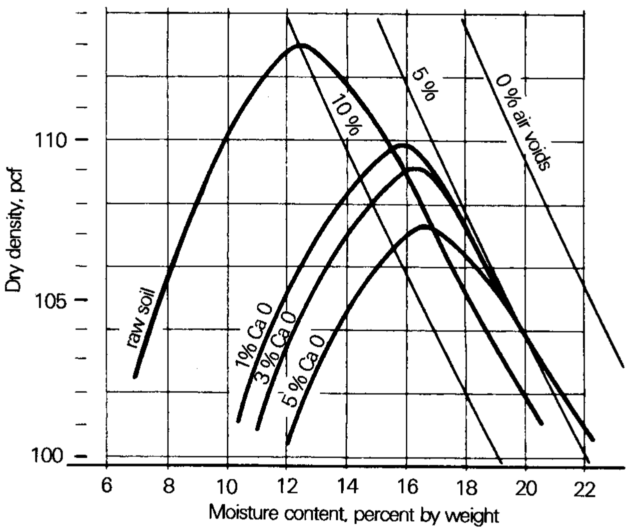


FIGURE 6A.230 Influence of quicklime content on the standard Proctor moisture-density relationship of a loess (from Brand and Schoenberg, 1959).

In soils treated with cement, $\gamma_{d\max}$ and w_{opt} may either increase or decrease (HRB, 1961). Increases in $\gamma_{d\max}$ usually occur for sands and sandy soils and sometimes to a small degree for heavy clays. Little or no change in $\gamma_{d\max}$ generally occurs for light to medium clays. Decreases in $\gamma_{d\max}$ may occur in silts; w_{opt} normally decreases for clays, increases for silts, and changes little for sands and sandy soils. Typical values for changes in w_{opt} and $\gamma_{d\max}$ compared to the values for the comparably compacted untreated soil, are summarized in Table 6A.27 according to type of soil for the normal range of cement contents. The reasons for the wide variation in the changes in w_{opt} and $\gamma_{d\max}$ that occur in soils because of cement stabilization are not completely understood. The diversity of changes in $\gamma_{d\max}$ may relate to the opposing trends caused by the high specific gravity of Portland cement ($G_s \cong 3.15$) compared to that for the soil particles (typical average $G_s \cong 2.65$ to 2.75),

TABLE 6A.27 Maximum dry densities and optimum water contents for cement-treated soils compared to the corresponding values for untreated soils (from HRB, 1961)

	Change in $\gamma_{d\max}$		
Type of soil	pcf	kN/m^3	Change in w_{opt}, %
Sandy loams	0 to +3	0 to +0.5	–1 to +1
Sands	0 to +6	0 to +0.9	0 to –1
Silts and loams	0 to –6	0 to –0.9	0 to +3
Silts	–3 to +1	–0.5 to +0.2	0 to –3
Medium clays	0 to + 1	0 to +0.2	0 to –2
Heavy clays	–1 to +2	–0.2 to +0.3	0 to –4

which tends to increase the density, and the resistance to compaction produced by the cementitious reactions, which tends to decrease the density.

As the length of the delay between mixing and compaction increases, the characteristics of the treated soil, including the moisture-density relationships, continuously changes as the cementitious or pozzolanic reactions occur with time. The effect of a 2-h delay on the moisture-density relationships of a pulverized chip seal pavement stabilized with a cementitious fly ash is illustrated in Fig. 6A.222. In general, $\gamma_{d\max}$ decreases and w_{opt} increases with increasing length of the delay. The effect of delay in compaction is usually much less important in pozzolanic stabilizers (lime and lime–fly ash) than in self-cementing stabilizers (cement and class C fly ash), as illustrated in Fig. 6A.231. Because the length of the delay may significantly affect the engineering characteristics of the stabilized soil (for example, see the reduction in strength shown in Fig. 6A.222), it is essential that when preparing laboratory specimens for testing, the delay between mixing and compaction match the delay expected during the actual field construction. Sometimes retarders are used with cement or class C fly ash to inhibit the cementitious reactions during the anticipated delay between mixing and compaction.

6A.7.2.7 Permeability

The changes in permeability that occur because of chemical stabilization depend on the type of soil, the amount of stabilizer, the process used to add the stabilizer (injection versus mixing), and the density achieved during compaction (admixture-stabilized soil). Unfortunately, the data available in the literature regarding the permeability of chemically stabilized soils is rather limited, with most of the information pertaining to soils treated with cement and lime. Data reflecting changes in permeability produced by the chemical stabilization of particular soils can be found in HRB (1961), Townsend and Klym (1966), Ingles and Metcalf (1973), Poran and Ahtchi-Ali (1989), Bowders and colleagues (1990), and Taki and Yang (1991), among others.

In general, chemical stabilization increases the permeability of soils with very low natural permeabilities and reduces the permeability of soils with moderate to high natural permeabilities. Ex-

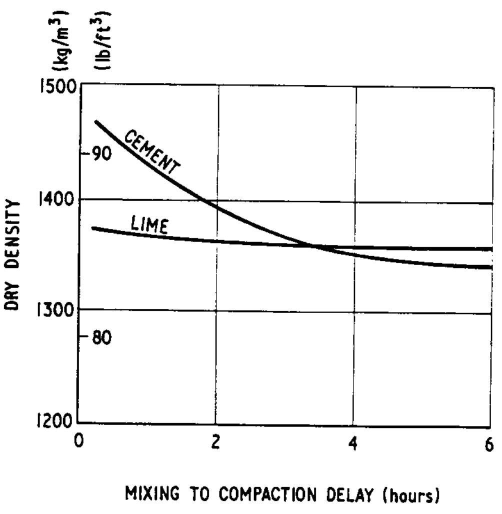


FIGURE 6A.231 Effect of delay in compaction on the as-compacted density of a heavy clay treated with 10% stabilizer (from Ingles and Metcalf, 1973; after Dumbleton, 1962).

amples illustrating these two general trends are given as follows: (*a*) Soil Sta, an organic chemical stabilizer that is injected into expansive soils to control swelling potential (see Sec. 6A.7.2.2), increases the permeability of montmorillonitic soils by as much as 40-fold; and as shown in Fig. 6A.232, the treatment of two fines sands with cement reduced their permeabilities by more than two orders of magnitude. Ingles and Metcalf (1973) have indicated the following general trends for cement stabilization regarding how the permeability changes for different types of soils:

Permeability is generally decreased	Permeability is generally increased
Well-graded gravel	Silt
Well-graded sand	Silty clay
Well-graded gravel-sand-clay	Very poorly graded soil
Sand and gravel	Heavy clay
Poorly graded sand	Organic and sulfate-rich soil
Silty sand	
Sandy clay	
Silty-sandy clay	

Deviations from these general trends may occur, particularly for the borderline soil types (sandy clay, silty-sandy clay, silt, and silty clay), but deviations for the extreme soil types (clean coarse-grained soils and highly plastic clays) are unlikely. These trends are also applicable to other chemical admixtures (lime, lime–fly ash, and fly ash). The effect of the stabilizer on permeability usually increases with increasing stabilizer content, as illustrated in Fig. 6A.232, up to a limiting value of stabilizer content above which little or no additional change occurs.

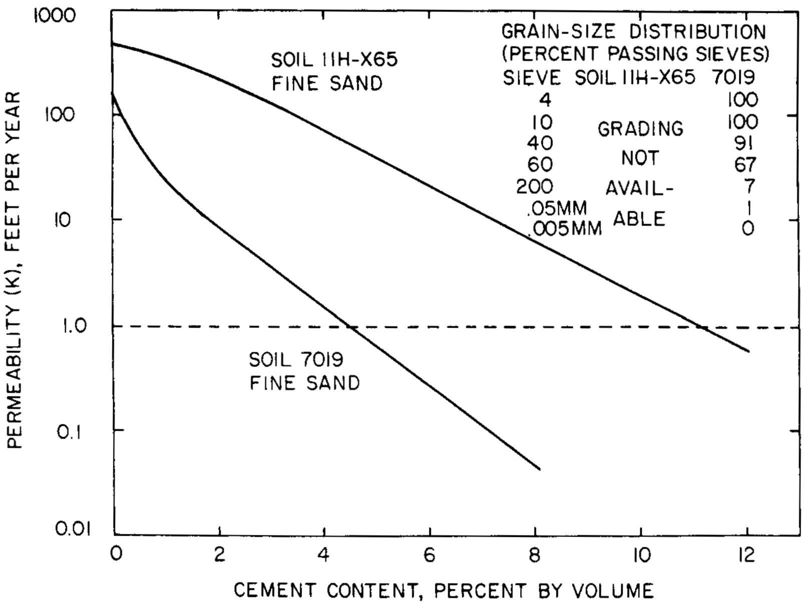


FIGURE 6A.232 Permeability versus cement content for two cement-treated sands (from HRB, 1961; after Leadabrand, 1956).

6A.7.2.8 Liquefaction Potential

For many years it has been recognized that cementation in saturated granular soils, whether occurring naturally or induced artificially, tends to inhibit liquefaction when the soil is subjected to seismic loading. In a laboratory study conducted by Clough and coworkers (1989), the following results and conclusions were determined regarding the liquefaction resistance of sands treated with cement:

1. For the sands studied, when the unconfined compressive strength (q_u) exceeded 100 kPa (14.5 psi), the soil was, for practical purposes, not liquefiable.
2. When q_u for the cemented sand was greater than 60 kPa (8.7 psi), differences in unit weight had little influence on the resistance to liquefaction.
3. The addition of small amounts of cement (1 and 2%) to the sands substantially reduced the potential for liquefaction (see Fig. 6A.233)

Although the use of chemical stabilization to control liquefaction potential is technically viable, it has been used only infrequently in practice. No case histories have been found in the literature in which an entire liquefiable mass beneath a structure has been chemically stabilized. In two recent projects, in situ soil mixing techniques (see Sec. 6A.7.3) were used to reduce the potential for damage from liquefaction by containing the liquefiable soil within cells formed by cement-stabilized walls. In the Jackson Lake Dam modification project, cement-stabilized walls arranged in a hexagonal shape (Fig. 6A.234) were built within the liquefiable soils beneath the earth dam (Taki and Yang, 1991). Research has shown that soil-cement walls constructed in a cellular arrangement can reduce seismically generated pore pressures within loose granular soils by as much as 90%. In a similar application, soil-cement walls were constructed in a circular pattern beneath the walls of two spill tanks for a paper mill (Broomhead and Jasperse, 1992). The soil–cement walls were intersected by cross-drains at regular intervals to allow water generated by the dissipation of excess pore water pressures to drain to the exterior of the tanks (Fig. 6A.235). The cross-drains were connected by a continuous perimeter drain inside the wall footing, which in turn was connected to a gravel drainage blanket beneath the entire floor slab.

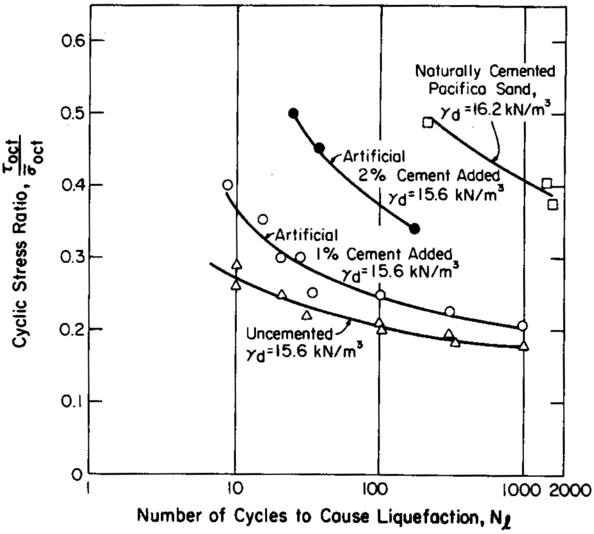


FIGURE 6A.233 Influence of cementation on the number of cycles required to cause liquefaction of cemented sands (from Clough et al., 1989).

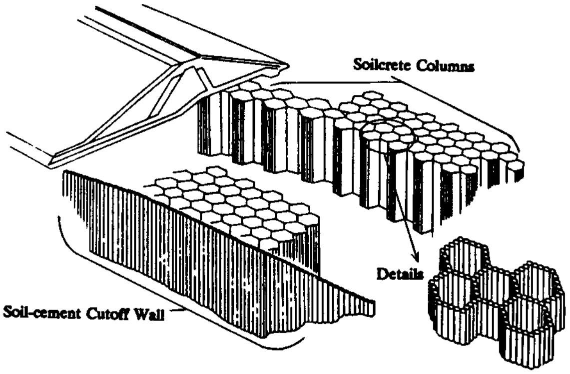


FIGURE 6A.234 Schematic illustration of the soil-cement walls used to control liquefaction potential in the Jackson Lake Dam modification project (from Taki and Yang, 1991).

6A.7.3 Construction Methods

Numerous methods can be used to incorporate chemical stabilizers within soils. These methods can be divided into two general categories—injection and mixing. The information provided in this section is meant not as a complete treatment of the subject but as a summary of the principles, techniques, and equipment used to effect chemical stabilization of soils. Additional details on this topic can be found in NCHRP (1976), PCA (1979), Boynton and Blacklock (1985), Little and coworkers (1987), TRB (1987), and Broms (1993).

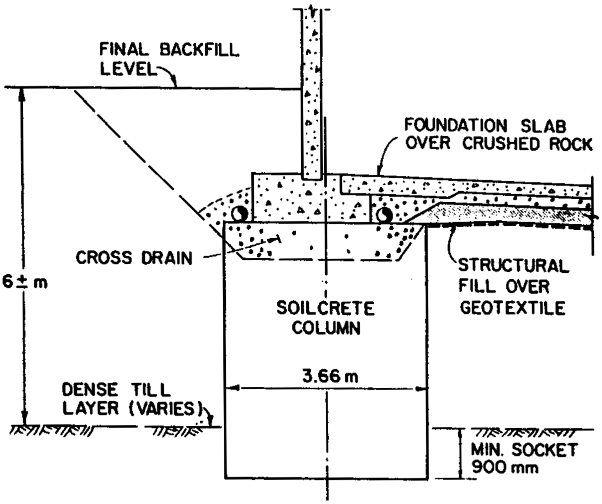


FIGURE 6A.235 Foundation details showing soil-cement walls used to control liquefaction potential beneath two spill tanks (Broomhead and Jasperse, 1992).

6A.7.3.1 Injection

Chemical stabilizers contained within solutions or suspensions (slurries) can be injected into the soil through the openings (void spaces and cracks). Many techniques have been used to inject chemical stabilizers into soils, some of which are similar or the same as those used for grouting (see Sec. 6B). In the following discussions, a technique is described to inject Soil Sta, a proprietary organic chemical stabilizer (refer to Sec. 6A.7.2.2). Bear in mind that Soil Sta is used almost exclusively in remedial applications to stabilize expansive soils beneath existing foundations, and the following discussions are necessarily aimed toward that application.

In special cases in the United States and for most applications within the United Kingdom, chemical injection is performed through a specially designed stem. In the system utilized, 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. Once positioned, the hand valves are switched to close the core and divert flow into the annular space and out the injection ports (Fig. 6A.236).

In some instances, a particular soil might tend to resist penetration by Soil Sta. 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 isolate selective zones (Fig. 6A.237). This equipment permits higher injection pressures and also allows some 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 stabilizer 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.

A similar analysis would be true for depth interval injection. Within a short depth (that is, 6 ft ≅ 2 m or less), chemical penetration can be fairly uniform given time, although placement of the stinger might vary a foot (0.3 m) or so one way or the other. This condition could change if two factors coexisted: (*a*) The depth of penetration is increased substantially, that is, 15 to 20 ft (4.6 to 6 m), and (*b*) packers or other positive seal methods are used to isolate each zone to be injected. Even in the latter example, true penetration of the zone may not occur if the placement pressure or specific soil characteristics favor communication between zones. No matter what precautions might be taken, the normal heterogeneous and fractured nature of the soil would tend to preclude exact placement of a specified volume.

At a pressure differential of about 3.5 psi (24 kPa), the system in Fig. 6A.236 would theoretically place about 12 gal/min (45 L/min) *neglecting line friction*. Hence, 10 s would theoretically place 2 gal (7.6 L) of chemical through the stinger perforations. [This volume equates to ⅛ gal/ft^2 (5 L/m^2) on a 4-ft (1.2-m) spacing pattern.] By timing the period of injection 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 excessive. [In field practice, a pressure differential of about 60 psi (414 kPa) delivered 2 gal (7.6 L) of chemical in 30 s through the stinger and approximately 60 ft (18.3 m) of ½-in (12.7-mm) ID hose.] The following equations are used to calculate velocities and pressure differentials:

Annular velocity is

$$V_a = \frac{Q}{A} \tag{6A.139}$$

which for an annular area, A, = 0.175 in^2 (113 mm^2) reduces, to

$$V_a(\text{ft/s}) = 1.83\,Q \text{ for } Q \text{ in gal/min} \tag{6A.140a}$$

$$V_a(\text{m/s}) = 0.148\,Q \text{ for } Q \text{ in L/min} \tag{6A.140b}$$

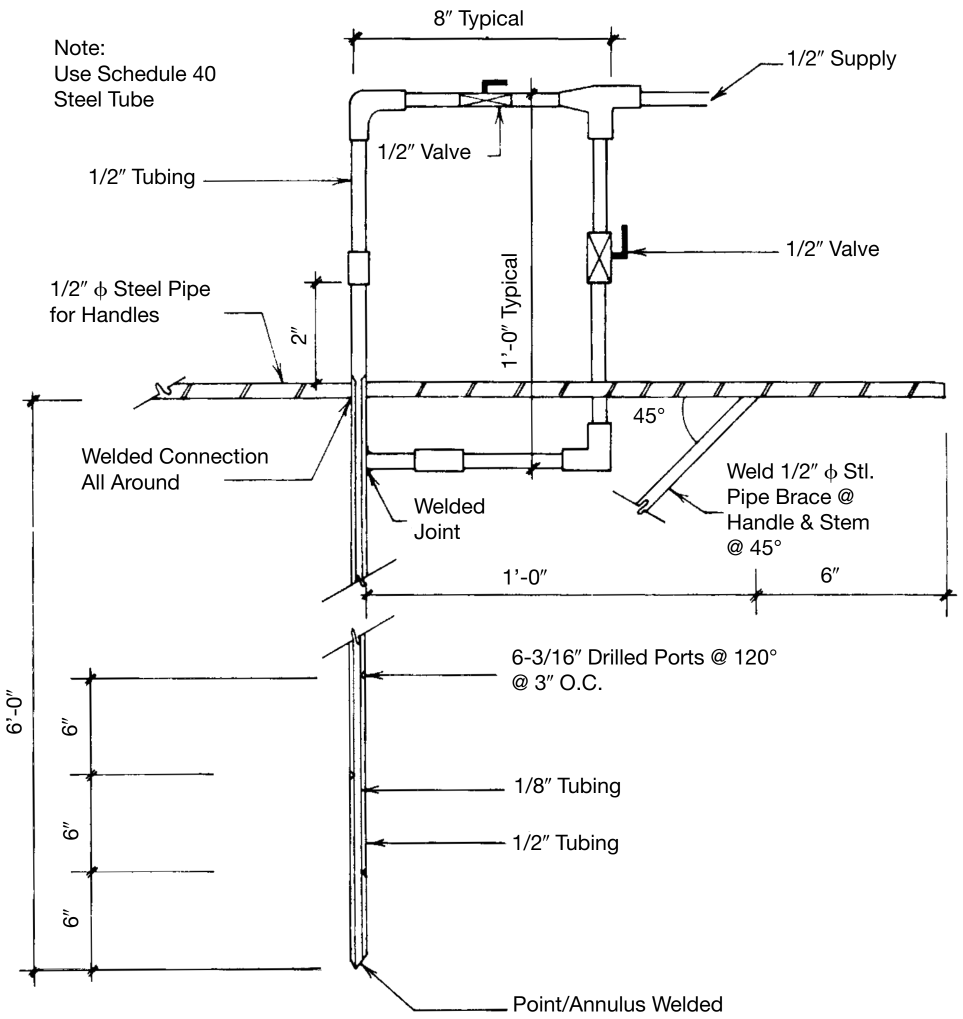


FIGURE 6A.236 Pressure injection stem (from Brown, 1992).

Port velocity

$$V_p = \frac{Q}{A_p C_D} = \frac{4Q}{D_p^2 \pi n C_D} \tag{6A.141}$$

which for $D_p = 3/16$ in (4.8 mm), the number of ports, n, = 6, and the discharge coefficient $C_D = 0.8$ (Brown and Gilbert, 1957), reduces to

$$V_p(\text{ft/s}) = 2.42\ Q \text{ for } Q \text{ in gal/mm} \tag{6A.142a}$$

$$V_p(\text{m/s}) = 0.195\ Q \text{ for } Q \text{ in L/min} \tag{6A.142b}$$

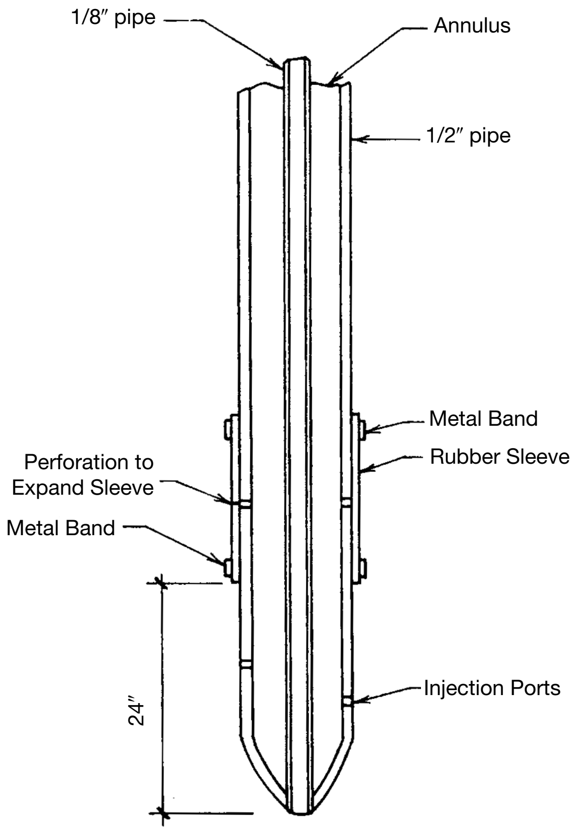


FIGURE 6A.237 Injection stinger with pack-off (from Brown, 1992).

The pressure differential is represented as

$$\Delta P = \frac{\gamma_f (V_p^2 - V_a^2)}{2g} = \frac{\rho_f (V_p^2 - V_a^2)}{2} \tag{6A.143}$$

where γ_f = unit weight (weight density) of the fluid (= 62.43 pcf = 9.807 kN/m^3 for water)
ρ_f = mass density of the fluid (= 1.0 Mg/m^3 = 1000 kg/m^3 for water)
g = acceleration owing to gravity = 32.17 ft/s^2 = 9.807 m/s^2

For V_p and V_a in ft/s [from Eqs. (6A.140*a*) and (6A.142*a*), Eq. (6A.143) can be represented as

$$\Delta P \text{ (psi)} = \frac{G_f (V_p^2 - V_a^2)}{148} \tag{6A.144a}$$

where G_f is the specific gravity of the fluid (= 1.0 for water).

For V_p and V_a in m/s [from Eqs. (6A.140*b*) and (6A.142*b*), Eq. (6A.143) can be represented as

$$\Delta P \text{ (kPa)} = \frac{G_f(V_p^2 - V_a^2)}{2} \tag{6A.144b}$$

Force developed from the hydraulic pressure is

$$F = PA \tag{6A.145}$$

where F = force
P = pressure
A = area

Equation (6A.145) illustrates the factors that create a lifting force on a foundation. Generally, pressure injection of chemical soil stabilizers does not involve lifting, as mudjacking or pressure grouting would. In fact, as a rule, chemical injection pressures are limited to prevent lifting. The safe operating pressure could then be estimated from the rearrangement of Eq. (6A.145) as follows:

$$P = \frac{F}{A} \tag{6A.146}$$

6A.7.3.2 Mixing

The primary objective of all mixing techniques is to obtain a thorough mixture of soil and stabilizer at an appropriate moisture content. There are three main types of mixing methods, broadly categorized as off-site, on-site, and deep in situ mixing. A general overview of these mixing methods will be given in the following subsections.

Off-Site Mixing. In off-site mixing techniques, excavated or borrow material is brought to the mixing site, stabilizers and water (if required) are thoroughly mixed with the soil, and the mixture is then hauled to the site for spreading and compaction. Depending upon preference or performance, the soil can be brought to the compaction moisture content either during the off-site mixing process or on-site with the soil remixed to distribute thoroughly the additional water. If the haul distances are very long or if there is a long delay between the off-site mixing and on-site compaction of the material, the soil should be brought to the compaction water content on-site to avoid potentially substantial losses in strength and stiffness of the stabilized material (see Sec. 6A.7.2.5).

Off-site mixing can be performed either in pads or in a central plant. When prepared in a mixing pad, the soil is spread on the ground surface and the stabilizer is added in either a dry or wet condition. If applied wet, the stabilizer is contained within either a thick suspension (slurry) or a solution depending on the chemical nature of the stabilizer and the primary fluid (usually water). Mixing is accomplished using a disc harrow, a grader, or a traveling mixing machine (see below for discussion of different types of mixing equipment).

The most common type of off-site mixing is conducted in a central plant. A typical layout of a plant for lime–fly ash mixtures is shown schematically in Fig. 6A.238, which is also typical of the plants used for lime and cement stabilization except only one stabilizer is added instead of two. The main components of a mixing plant are as follows (NCHRP, 1976):

1. Conveyor system
2. Aggregate hopper with belt feeder
3. Hoppers for stabilizers with feed control devices
4. Water storage tank with calibrated pump
5. Surge hopper for temporary storage of soil-stabilizer mixture

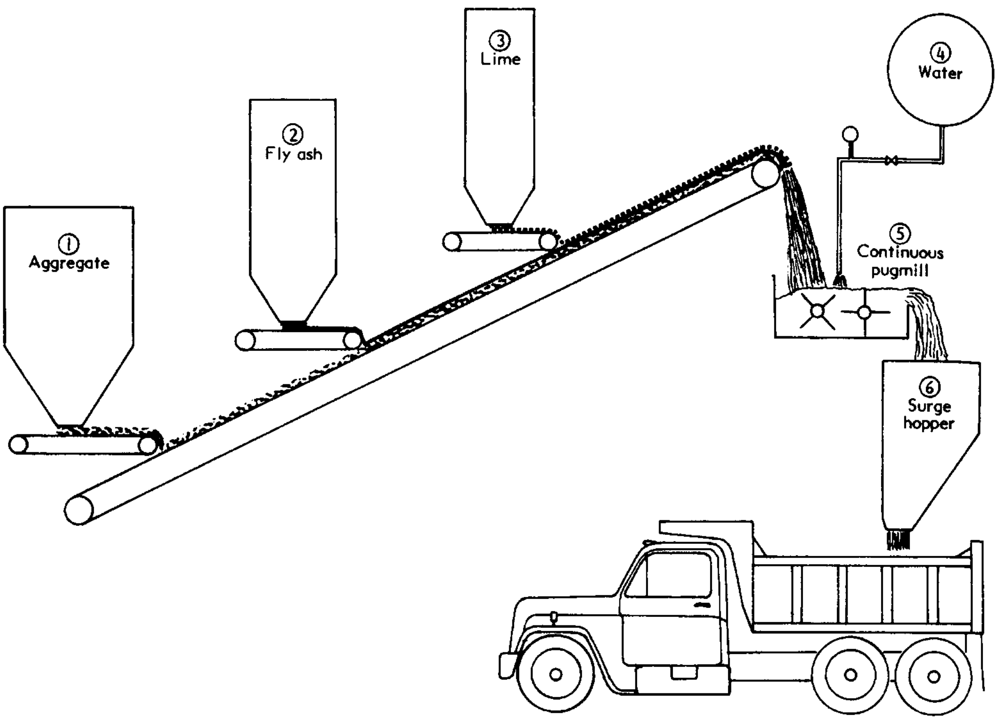


FIGURE 6A.238 Schematic diagram of typical plant layout for lime–fly ash-aggregate (LFA) mixtures (from NCHRP, 1976).

As soon as possible after mixing, the soil-stabilizer mixture is hauled to the site in open-bed or bottom dump trucks. If the haul distances are long, or if drying or scattering of the material en route is a problem, the trucks should be covered with tarpaulins or other suitable covers.

Some soils, particularly highly plastic clays, cannot be adequately pulverized and mixed in central mixing plants unless the soils are modified beforehand with a small amount of stabilizer to reduce plasticity and increase agglomeration. If feasible, however, central plant mixing accomplishes the most thorough and uniform mixing and reduces safety problems associated with the chemicals (for example, health problems for workers caused by the causticity of lime).

On-Site Mixing. On-site mixing can be accomplished in test pads, in a plant, or in portable mechanical mixers or through use of mixed-in-place techniques. Details on mixing in test pads or a central plant are given in the previous section. Mixing in pads is usually conducted off-site rather than on-site owing to a lack of sufficient on-site open space in most cases. On-site mixing plants are similar in concept and equipment to off-site plants but are usually smaller and hence cannot produce mixing at the same volumetric rate. The portable mechanical mixers used to mix soil and stabilizer are the same as or similar to the portable mixers used for concrete.

Mixed-in-place methods can be used on excavated materials, borrow materials, or in situ surficial soils. When borrow or excavated materials are mixed in place, the soil is first spread in either uniform layers or windrows. Wet or dry stabilizer is then added to the soil. Dry stabilizer can be added either in bags (lime or cement) or in bulk from pneumatic tanker trucks or mechanical spreaders. Bags are typically spotted by hand by dropping them at previously marked locations from a slowly moving truck. The bags are then slit open and the contents dumped into piles or transverse windrows across the area to be stabilized. The cement or lime is leveled by hand raking, or using a spike-tooth harrow, a nail drag, or a length of chain-link fence pulled by a tractor or truck. When both lime and fly ash are used, they can be spread separately, or if dry they can be preblended before being added to the soil. After lime (but not cement or lime–fly ash) is placed, the surface is usually sprinkled with water to reduce dusting. Either hydrated lime or quicklime can be added to the soil as a slurry. Lime and water are mixed in a central mixing tank, jet mixer, or tank truck and spread by either gravity or pressure spraying from a tank truck through spray bars.

Many different types of equipment can be used to mix soil and stabilizer in place. Pugmill travel plants can be used on excavated or borrow materials only. When hopper-type pugmill travel plants are used, soil is dumped into a hopper from a dump truck, and stabilizer and water (if desired) are added prior to mixing. Water can also be added after the soil-stabilizer mixture has been placed. For windrow-type pugmill travel plants, soil is placed in windrows and stabilizer added using one of the methods described in the previous paragraph. Rotary mixers, graders, and disc harrows can be used on excavated, borrow, or in situ surficial soils.

The difficulty of in-place mixing increases with increasing fineness and plasticity of the soils being treated. A double application of lime (modification of the soil with a small amount of lime followed by a suitable period for curing and then a second treatment for stabilization) or modification with lime prior to stabilization with cement or lime–fly ash, is often required when highly plastic clays (PI $\geq$ 50) are to be stabilized. In general, compaction should begin as soon as possible after mixing is completed.

After mixing, accomplished by any of the off-site or on-site methods, and placement, the stabilized soil must be compacted. The same equipment and methods used for near-surface compaction (see Sec. 6A.3.1 for details) can also be used for chemically stabilized soils. For cohesive soils, tamping foot or sheepsfoot rollers are typically used. Static or vibratory rubber-tired or smooth-drum rollers are generally used to compact granular soils or chemically modified cohesive soils that behave essentially as granular materials. When a relatively smooth surface finish is desired, smooth drum rollers can be used to proof-roll the material.

Compaction should commence as soon as possible after uniform mixing of soil, stabilizer, and water has taken place. It is common to specify that the chemically treated materials be compacted within 2 h after mixing. Because the reactions associated with lime stabilization are slow compared to those that occur in cement and lime–fly ash stabilization, additional time may be available between mixing and compaction for lime-treated soils.

A typical density specification for a chemically treated soil is a minimum relative compaction of 95% based on standard Proctor maximum dry density. However, the actual specification given on any project should be based on criteria that will yield the desired engineering characteristics. The reference Proctor tests should be conducted on samples of the soil treated with the same percentage of stabilizer used in the field and allowing the same delay between mixing and compaction as the average (or maximum) delay that occurs in the field. The specifications for compaction moisture condition are critical to successful stabilization. If the soil is too dry when compacted, incomplete hydration of the stabilizer may occur, resulting in engineering properties that may not meet the requirements for the project. In general, the compaction water content should not be less than standard Proctor optimum water content, which may be several percentage points higher than modified Proctor optimum water content. The specifications for compaction water content ideally should be based on the results of laboratory or field tests conducted to establish the range in water contents that will produce the desired engineering characteristics.

Deep in Situ Mixing. Although deep in situ soil mixing techniques originated in the United States in the 1950s, the major development of these methods has occurred over the last two decades in Japan and Sweden (Broomhead and Jasperse, 1992; Hausmann, 1990). Although the techniques were developed almost concurrently in Japan and Sweden and are similar in concept, they are generally referred to by different names—*deep soil mixing* for the Japanese method and *lime columns* for the Swedish method. In general, unslaked lime (quicklime, CaO) is used as the primary stabilizer in the Swedish method, and cement is used most frequently in the Japanese method. Other stabilizers and additives that have been used include fly ash, furnace slag, gypsum, hydroxyaluininum, potassium chloride, and bentonite (Broomhead and Jasperse, 1992; Broms, 1993).

Lime columns are used only in soft clays and silts. The mixing tool, which is shaped like a giant dough mixer (see Fig. 6A.239), is rotated down to the depth that corresponds to the desired length of the columns (Broms 1993). The direction of rotation is then reversed, and the mixing tool is slowly withdrawn as quicklime is forced into the soil by compressed air pushing through holes located just above the horizontal blades of the mixing tool. The blades are inclined so that the stabilized soil will be compacted during the withdrawal of the tool. The rotary table and kelly are on a mast, which

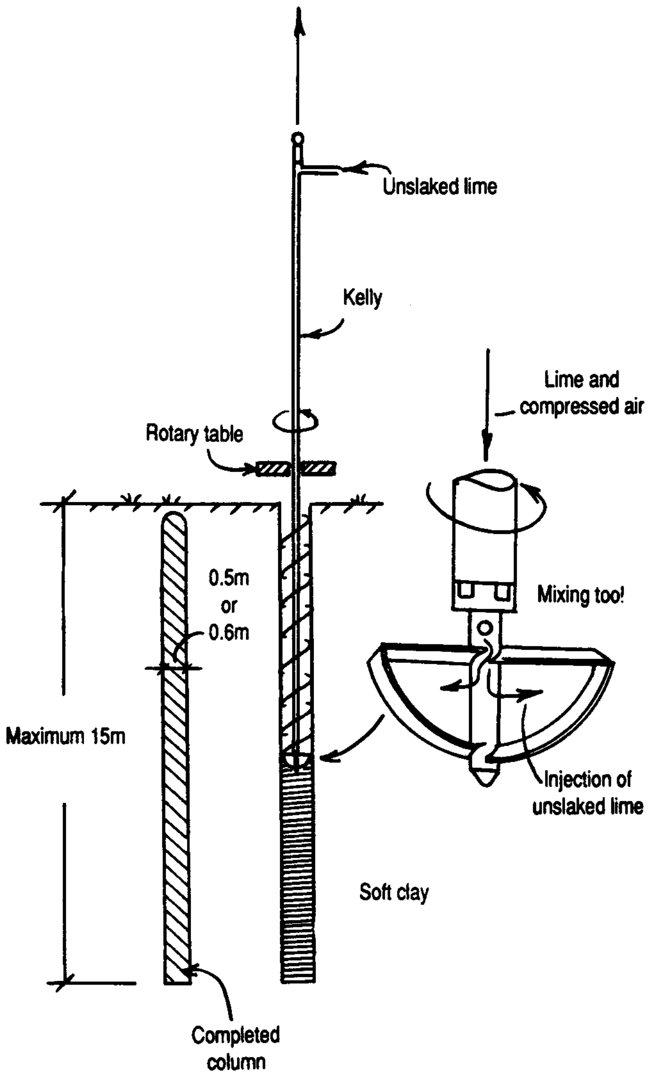


FIGURE 6A.239 Typical equipment used to manufacture lime columns (from Broms, 1993).

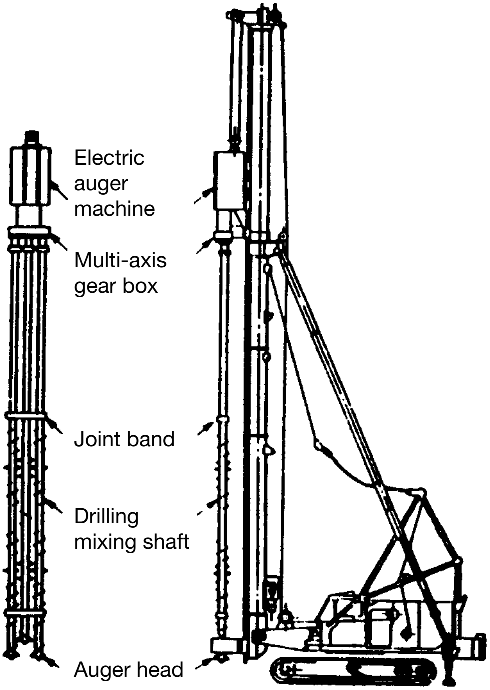


FIGURE 6A.240 Typical equipment used to manufacture deep soil-cement mixed columns (from Taki requiring the soil to be kept moist are to and Yang, 1991).

is mounted on either a standard front wheel loader or on a special off-road vehicle. The lime-stabilized columns are typically 0.5 to 0.6 m (1.6 to 2.0 ft) in diameter and can be constructed up to about 30 m (100 ft) in length.

Deep soil mixing can be used in soils ranging from soft clays to gravels (Broomhead and Jasperse, 1992; Taki and Yang, 1991). The equipment used in deep soil mixing consists of a crane-supported set of leads that guide a series of one to eight hydraulically driven mixing augers, as illustrated in Fig. 6A.240. The augers are typically 0.45 to 1.0 m (1.5 to 3.3 ft) in diameter, and columns of more than 60 m (200 ft) in length can be constructed. Each auger shaft is mounted with discontinuous auger flights and mixing paddles. The purposes of the discontinuous auger flights are to provide some vertical displacement for soil mixing and to prevent the transportation of soil to the surface. As penetration by the auger flights occurs, the soil is broken loose and the cement slurry is injected into the soil through the tip of the hollow-stemmed augers. The auger flights lift the soil and slurry to the mixing paddles where the materials are blended. As the auger continues to advance, the soil and slurry are remixed by additional paddles attached to the shaft. When multiple augers are used, overlapped soil-cement columns are created that can be extended to form any geometric shape or pattern.

Lime columns have been used extensively in Sweden, Norway, and Finland, with some applications in the United States, for the following purposes (Broms, 1993):

- To improve the total and differential settlements of light structures
- To increase the settlement rate and control the settlements of relatively heavy structures
- To improve the stability of embankments, slopes, trenches, and deep cuts
- To reduce the vibrations from, for example, traffic, blasting, pile driving

More than 3,000,000 m (10,000,000 ft) of lime columns have been installed in Sweden alone since 1975, with a production rate (1990) of about 600,000 m/yr (2,000,000 ft/yr).

Deep soil mixing has been used on more than 1500 projects in Japan and the Far East and was reintroduced in the United States in 1986 (Broomhead and Jasperse, 1992; Taki and Yang, 1991).

Soil-cement walls constructed by deep soil mixing have been used in the following applications:

- Cutoff walls for ground-water control
- Walls for excavation support
- Stabilization of soft or liquefiable soils

A spinoff technology to deep soil mixing has recently been developed and has been dubbed shallow soil mixing (Broomhead and Jasperse, 1992). This process uses a single auger 1.0 to 3.7 m (3 to 12 ft) in diameter and can be used to stabilize soils to a depth of 10 m (33 ft) or less. Shallow soil mixing has been used for both geotechnical and geoenvironmental applications.

Curing. Three conditions are needed for proper curing of soils stabilized with cementitious or pozzolanic admixtures:

1. *Favorable temperatures*. Temperatures greater than 40°F (4°C) are needed for the cementitious or pozzolanic reactions to occur at a reasonable rate. Therefore, construction of admixture stabi-

lized soils should not commence if the temperatures during the month or so following the completion of the construction will drop below the minimum required temperature for extended periods.

2. *Passage of time*. Usually at least a 3- to 7-day undisturbed curing period is specified before any appreciable load can be applied.
3. *Moist conditions*. The primary purposes for requiring the soil to be kept moist are to ensure that adequate moisture is available for hydration of the stabilizer and to keep the shrinkage cracking to a minimum. In lime-stabilized soils, moist conditions are also needed to prevent carbonation. If the stabilized soil is backfilled immediately after construction, no special measures are required for adequate curing. If the stabilized soil is exposed to the atmosphere for more than a few hours following construction, appropriate procedures should be taken to ensure that complete curing will occur. The procedures used are of two general types—moist or asphaltic curing. In *moist curing*, the surface is sprinkled with water to keep it damp. On small jobs, the stabilized soil may also be covered with plastic sheets. Light compaction with small smooth drum rollers or vibratory plate compactors may be used, as needed, to keep the surface knitted together. *Asphaltic curing* involves sealing the surface with a coat of cutback asphalt or bituminous emulsion.

Quality Control and Assurance. For admixture-stabilized soils prepared by off-site or onsite mixing methods, the procedures for quality control and assurance are similar to those for compacted untreated soils (Sec. 6A.3.5), with the additional requirements that the stabilizer content and uniformity of mixing be checked. The control of stabilizer content is a two-step process involving monitoring and spot-checking the amount of stabilizer that is added to the soil and conducting tests on samples obtained from the material immediately after mixing is completed. Standard chemical analyses for determining the cement or lime content of freshly mixed, treated soils are given in ASTM D2901 and D3155, respectively.

The uniformity of cement-stabilized soils can be checked by visually inspecting the exposed soil within a trench dug across the width of the treated area and to the desired depth of treatment. A satisfactory mixture will exhibit a uniform color throughout, and a streaked appearance indicates a nonuniform mixture. Checking the uniformity of lime-soil mixtures is more difficult because the visual appearances of the untreated soil and the treated soil are the same. Phenolphthalein, a color-sensitive indicator solution, can be used to check that the minimum lime content required for soil treatment is present. When the solution is sprayed on the soil, the soil will turn a reddish pink color if free lime is available (pH = 12.5). Short-cut strength methods can also be used.

The quality control and assurance procedures used for deep in situ mixing are necessarily complex because the soil is stabilized to substantial depths. Details regarding the methods used for lime columns are given in Broms (1993).

6A.8 MOISTURE BARRIERS*

As implied by their names, the intended purpose of horizontal or vertical moisture barriers is to prevent the seepage of moisture into or out of an expansive soil located beneath a foundation. Moisture barriers have been used as both preventive and remedial measures. Unfortunately, moisture barriers are frequently ineffective in controlling swelling in expansive soils for the following reasons (Jones and Jones, 1987):

1. Moisture may enter the protected area from below the barrier, such as by upward migration of water vapor from an underlying ground-water table.
2. Variations in moisture content of the protected soil will still occur in covered soils owing to changes in soil temperature (called hydrogenesis). Even in dry climates the air has some mois-

*Coauthored by Robert Wade Brown.

ture, which enters the subgrade soil. As the ground temperatures cool at shallow depths, the moisture in the air condenses and accumulates in areas that are covered.

3. Moisture barriers cannot prevent moisture flow through shrinkage cracks, slickensides, or permeable inclusions below the barriers.

The use of moisture barriers is not a viable preventative or remedial measure for expansive soils because the money spent could be better spent on an alternative solution that is both more effective and more permanent.

6A.8.1 Horizontal Barriers

Horizontal moisture barriers are intended to prevent or substantially reduce the moisture loss from an area of soil resulting primarily from evaporation. A typical design of a horizontal barrier using a polyethylene membrane overlain by a thin layer of gravel is depicted in Fig. 6A.241. Membranes made from polyvinyl chloride (PVC) and polypropylene have also been used. These plastic membranes typically range from about 4 to 20 mils (0.2 to 0.8 mm) in thickness. Other materials that have been used to make the impervious barrier include concrete and asphalt. Regardless of the type of barrier material used, construction techniques are important to the effectiveness of the barriers (Nelson and Miller, 1992). Care should be taken to seal joints, seams, rips, or holes in the barrier.

Jones and Jones (1987) indicate that the effectiveness of horizontal barriers can be improved in the following ways:

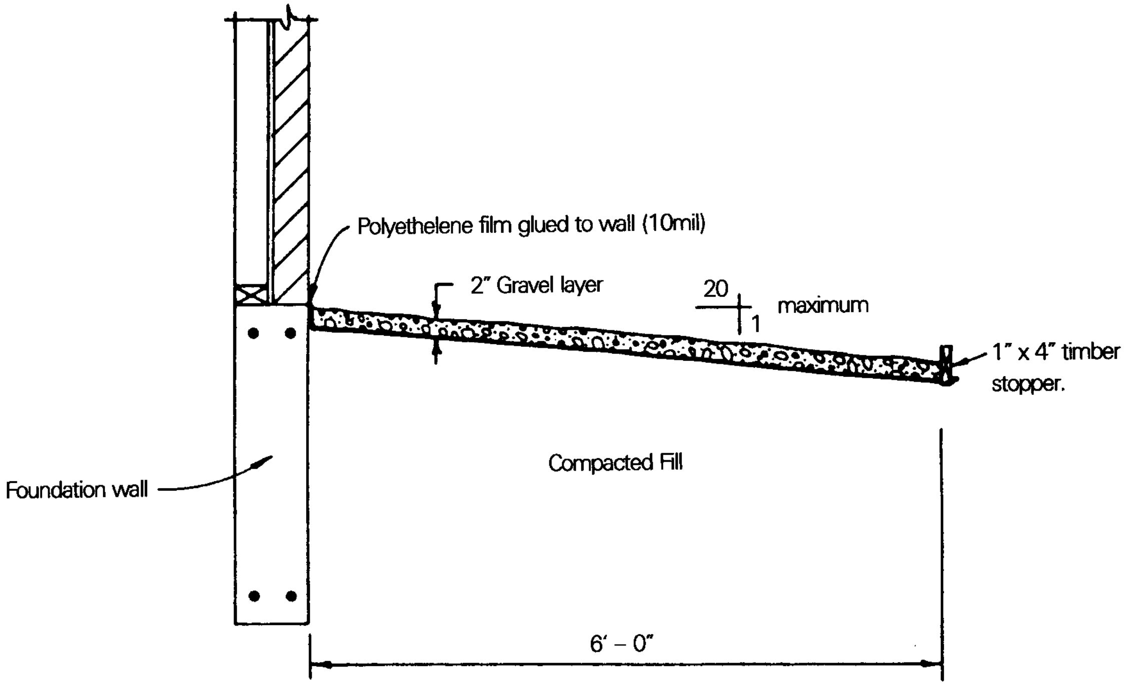


FIGURE 6A.241 Horizontal moisture barrier consisting of a polyethylene membrane overlain by a thin gravel layer (from Chen, 1988).

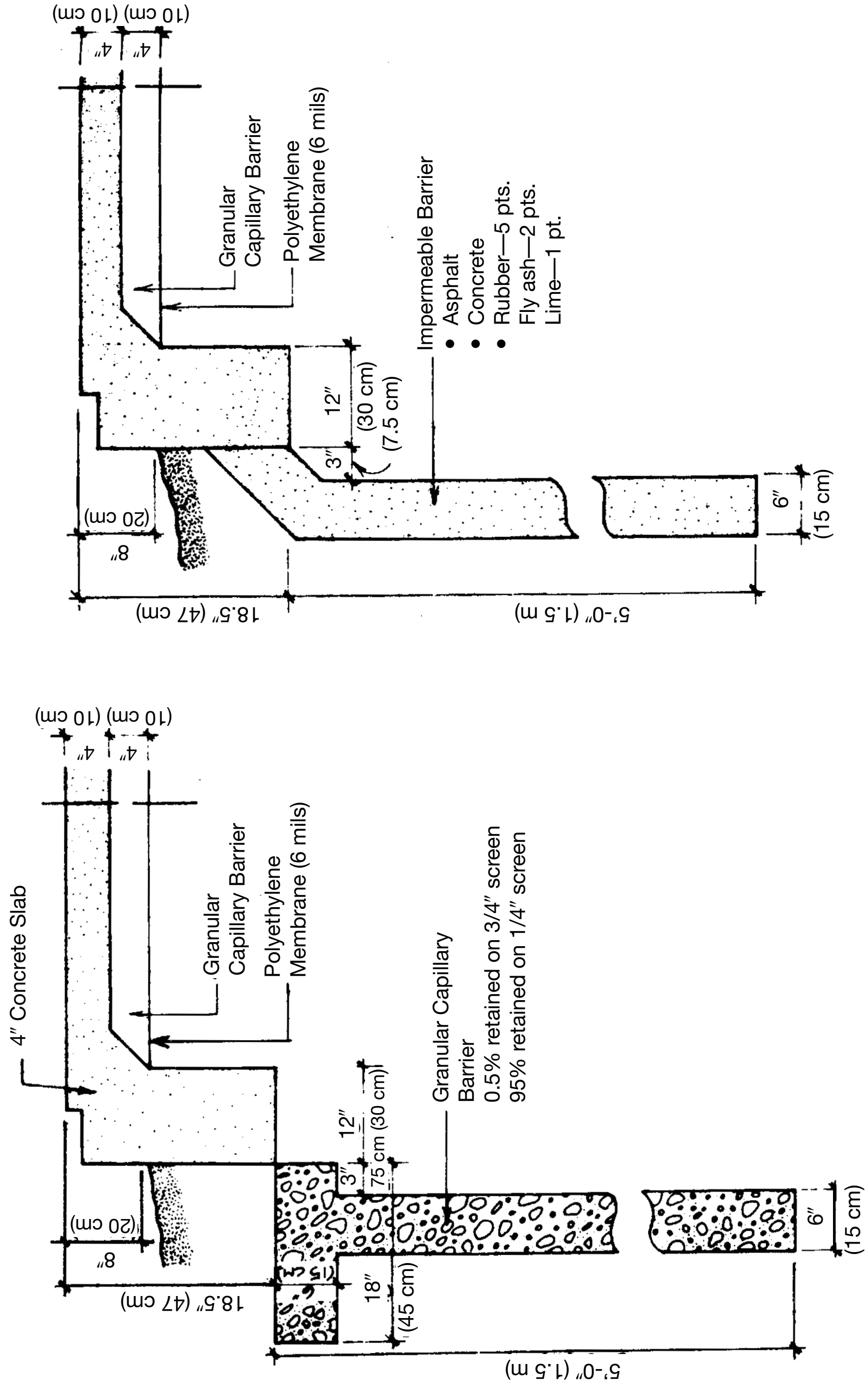


FIGURE 6A.242 Typical permeable and impermeable vertical moisture barriers (from Brown, 1992).

1. The barriers should be placed at least 2 ft (0.6 m) below the ground surface because at this depth, the daily variations in soil temperature are negligible.
2. The preconstruction moisture condition of the bearing soil and the soil underlying the barrier should be controlled to maintain a high water content that is uniform throughout the soil.
3. Open areas surrounding the barriers should be irrigated regularly using a sprinkler system to sustain a consistently high water content.

Even if the measures outlined above are taken, the behavior of a horizontal barrier is not reliably predictable. Discussions of data reflecting the success or benefit in using horizontal barriers can be found in Chen (1988) and Nelson and Miller (1992). Both sources seem to agree that a horizontal barrier does little to prevent or lessen the increase in moisture (heave) within a foundation soil over several years. However, the presence of a barrier does seem to cause the buildup in moisture to develop more slowly and to be more uniform. The pertinent question is whether these benefits justify the cost.

6A.8.2 Vertical Barriers

Vertical moisture barriers are intended to avoid or substantially limit the lateral transfer of water between the foundation bearing soils and those outside the perimeter. The theory is that if expansive soils are maintained at a constant level of moisture, no volumetric changes occur. The vertical barrier can consist of an excavated trench lined or filled with any impermeable material such as polyethylene membrane, fiberglass, asphalt, concrete, semihardening slurries, or sometimes even tar paper (see Fig. 6A.242). If polyethylene membrane is used, it should be sufficiently thick and durable to resist puncturing and tearing during installation. The trench should ideally extend as deep as the zone of seasonal changes in moisture (active zone). Sometimes a horizontal barrier is used with a vertical barrier to prevent wetting of the soil between the vertical barrier and the building.

The published data dealing with the effectiveness of impermeable vertical barriers are similar to those described for horizontal barriers. Vertical barriers tend to reduce increases in soil moisture (heave) during the first year or so; however, after 4 or 5 years, the results become nearly the same with or without the barrier. As with horizontal barriers, the rate of increase in moisture is generally slower and more uniform when a vertical barrier is used. Again the question arises: Does the result justify the cost? As stated by Chen (1988), ". . . in view of the high cost involved in the installation of a vertical moisture barrier, especially where great depth is required, it is doubtful that such an installation is of sufficient merit to warrant the expense."

A variation of this design is the permeable, vertical soil barrier, also shown in Fig. 6A.242. This technique is primarily intended to prevent or minimize external water from entering the bearing soil matrix, somewhat similar to a French drain. Published information on the performance of permeable vertical barriers is not readily available.

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SECTION 6B

GROUTING TO IMPROVE FOUNDATION SOIL

JAMES WARNER

6B.1 INTRODUCTION

Geotechnical pressure grouting consists of forcing grout under pressure into a subsurface formation. The grout can be a liquid solution, fluid suspension, slurry, or of a stiff mortar-like consistency. It will generally harden at some point after injection so as to become immobile. Prior to the last few decades, most grouting was done to fill discontinuities in rock, with the primary purpose of reducing water movement through the formation. Now, in addition to water control, grouting is also extensively used to strengthen soil, either permanently or temporarily as an aid to construction.

Grouting is not new. In fact, records abound of grouting projects throughout the 1800s and even prior to that. Most of the early work involved injection of aqueous suspensions or slurries, often containing cement, lime, or clay, into joints and seams in the bedrock underlying dams in order to reduce water leakage. As early practice involved filling of seams or voids, the grout had to be very flowable, and the maximum particle size considerably smaller than the thickness of the particular discontinuity to be filled. Because the pore spaces in soils are generally much smaller than typical rock joints, injection of particulate grout had limited success in soil.

Accordingly, low-viscosity chemical “solution” grouts, which could permeate soil formations and chemically harden, were developed. In 1887, a patent was issued to Jeziorski for a sodium silicate based formulation, which could be mixed on-site and injected. Unfortunately, the chemicals would react soon after mixing, requiring very rapid injection, and all too often, would harden in the injection hoses and delivery system. This limitation restricted the application of these early grouts in soils.

To overcome the problem of early hardening in the delivery system, Hugo Joosten, a Dutch mining engineer, developed a two-shot sodium silicate based grout system, which was patented in the United States in 1925. Therein, the sodium silicate base chemical was first injected into the soil, followed by injection of a reactant chemical, commonly calcium chloride, which would cause the silicate to harden. Although Joosten’s system was used until the late 1960’s, difficulty in achieving complete mixing of the two components in the ground discouraged it’s widespread use.

In the 1950s, large chemical suppliers started marketing a variety of solution grout systems, primarily focused at the reduction of water movement through soils but for soil strengthening applications as well. Many of those systems are no longer available due to environmental restrictions or economic considerations. Although commercially available chemical grout systems remain available, sodium silicate based grouts are the most widely used, especially for soil strengthening work. The earlier limitations of rapid setting have been overcome in modern formulations. Many different reactant systems for silicate base grouts are readily available, and it is common for the grouts to be proportioned on the job site by the specialist contractor, using his own particular reactant system and formulation. There are also a number of proprietary sodium silicate base grout systems commercially available. For water control grouting, more sophisticated chemistry is generally involved, and commercially available formulations are almost exclusively used.

Another significant contribution to the current state of the art in grouting of soils was the development of the "mudjack" machine in 1933. The original objective of the machine was the filling of voids under, and raising of, settled concrete pavement. As conceived, a mixture of "loam" or clayey soil was used. As experience with the machine was gained, it was found that the addition of portland cement resulted in a stronger and more durable grout material. Likewise, it was found that by varying the consistency of the grout, a wider range of work could be accomplished. As a natural outgrowth of their work, some of the more inventive operators of the mudjack attempted stabilization of the soil by various means, including pumping of relatively stiff mud mixes into predrilled holes. Although early work was performed on a somewhat "hit or miss" basis, and with little engineering input, a great deal of knowledge was gained and such work was actually a forerunner of the "compaction grouting" process that is now widely practiced.

In the 1940's it was found that mixing a source of calcium with clay soil was beneficial to strengthen and/or reduce the expansiveness of clay. High calcium hydrated lime became the material of choice for such mixing, due to its economy and wide availability. The practical depth for physical mixing was limited however, and a method for deeper mixing was needed. This resulted in use of pressure injection of lime slurries, using grouting techniques. Lime injection continues to be utilized, primarily in geographic areas where swelling clay soils predominate.

6B.1.1 Foreign Development

A very significant amount of our present grouting knowledge is the result of foreign development, primarily from the European countries, where extensive use of both solution chemical grouts and particulate suspension grouts has occurred. Unlike American practice, much European work is performed on a design–build basis, and very large projects are commonplace. This has resulted in large multidiscipline firms possessing the capability to perform research and development, design engineering, and the actual construction, as well as design–build and maintain the very specialized equipment often utilized. Such ability to integrate the various disciplines combined with strong financial capability has resulted in many technical advances. There has, however, been reluctance on the part of such firms to share their knowledge, and in fact, it is not unusual for their proposals to be completely void of any mention as to the exact procedures or materials to be used, but rather only the end product to be obtained.

Unique to European work is the use of large central grout plants (Figure 6B.1), often containing a number of different types of mixing and pumping units. Such plants are nearly always provided with automated continuous monitoring and recording equipment, and the operation is usually under the direct supervision of a professional engineer. With exception of techniques that rely on the use of massive equipment such as jet grouting, the European philosophy for grouting has remained largely unchanged in recent times, and although improved and refined, solution and fluid suspension grouts continue to be used almost exclusively.

6B.1.2 U.S. Development

Contrary to the European experience, development of soil grouting in the United States was somewhat erratic and primarily the work of small, widely dispersed specialty contractors. The more common grouting of rock for dam foundations was under the control of one of three large federal government agencies: The Army Corps of Engineers, Bureau of Reclamation, and Tennessee Valley Authority. These agencies generally designed and supervised their grouting operations in-house, to the point that the contractors basically furnished only labor, equipment, and materials, performing the work strictly as directed by the agency. This effectively discouraged research or development by the contractors, resulting in little interaction between those contractors doing the more traditional dam foundations and those performing work other than dam-related, which included virtually all grouting of foundation soil. Unfortunately, a shroud of secrecy, often embraced by the latter group,

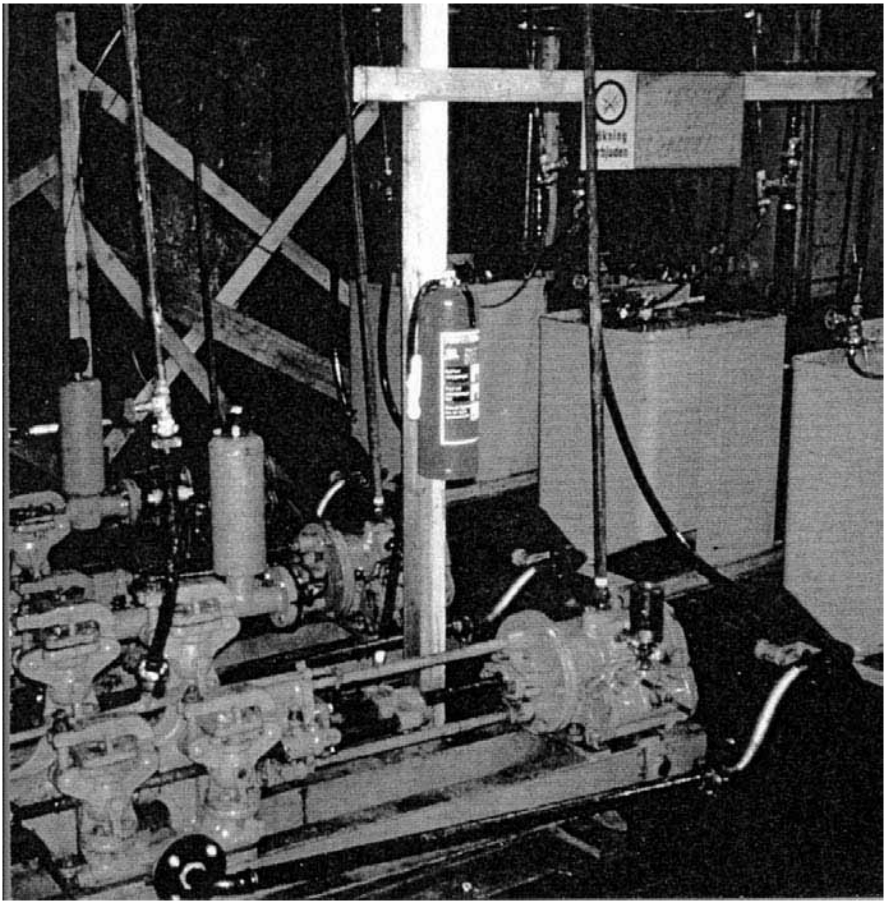

FIGURE 6B.1 Large European central grout plant.

prevented the sharing of experiences, so that neither widespread knowledge nor use of their developments occurred.

It is only within the last few decades that extensive knowledge of grouting technology has become available. A major contributor to this change has no doubt been the week-long Short Course on Fundamentals of Grouting, now sponsored by the University of Florida and held annually each year since 1980. Additionally, the American Society of Civil Engineers Geo-Institute has sponsored specialty conferences, as well as a number of sessions, at their gathering, on grouting. Sadly however, despite the dissemination and sharing of knowledge, some firms continue to maintain a "shroud of secrecy" in an effort to convey the idea that only they are capable of the "magic touch." Even worse is the sponsorship of publications and seminars that often provide incorrect information, favorable only to the sponsor. Such marketing efforts, often lack technical accuracy and incorrect information is frequently provided. Whereas foundation grouting knowledge, is now fairly well developed, many practitioners, especially design engineers, remain unaware and/or misinformed, of certain well-established technology.

6B.2 WHY GROUT IN SOIL?

There are many reasons for grouting in soil, but the most frequent is to strengthen it. This can be accomplished through densification, increasing the cohesion of granular soils, or in the case of clay, altering the chemistry. Another major area of work involves control of subsurface movements of water and reducing the permeability of soil.

6B.2.1 Densification to Prevent or Arrest Settlement and Mitigate Liquefaction

In the United States, compaction grouting is the most frequently used procedure for densification of existing soils. It is widely used for arresting settlement of structures of all types (see Figure 6B.2). It also is widely used for predensification of faulty soils, prior to construction, as well as soil densification under existing structures for mitigation of the liquefaction potential during earthquakes. An extension of compaction grouting, which involves essentially the same equipment and grout mixtures, is "groutjacking," which can accurately raise or level settled structures and effect other improvements.

Compaction grouting is uniquely American and is the only major grouting technique to originate in the United States. Its early development was in California in the mid to late 1950s. It is only within the last few decades however, that the procedure has been extensively used. Although much research has occurred and the technology is well documented, some practitioners fail to utilize the proper procedures, resulting in improper and often incompetent performance. It is unfor-

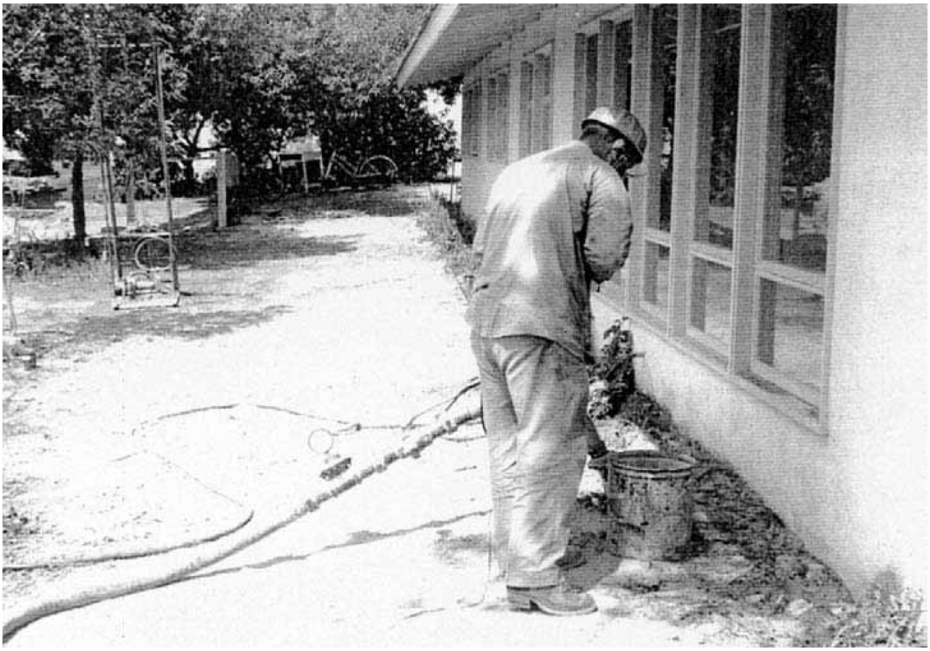

FIGURE 6B.2 Compaction grouting under a structure to correct settlement damage.

tunate that such poor performance has in many instances resulted in limited effectiveness, and sometimes in actual damage to the structure it was meant to repair. Although the greatest use of compaction grouting remains in the United States, it is now recognized and used worldwide, especially in Asia.

6B.2.2 Soil Solidification to Increase Cohesion of Granular Soils

Another form of strengthening is the solidification of sandy soils, wherein the individual grains are bonded together with a chemical solution or a fluid suspension grout (Figure 6B.3). There are a variety of chemical solution grouts, but they all work in essentially in the same manner. A base material, which usually represents the majority of the final mix and is often diluted with water, is combined with one or more reactant materials. The base component is usually fluid, whereas the reactants can be either liquid or powder, depending upon the individual grout formulation. At some period of time after mixing the base and reactants together, setting occurs, and the injected solution turns either to a foam, gel, or solid state. The setting time can be instantaneous to several hours, depending upon the individual system used and the environment at the work site.

Many chemical grouts are mixed and pumped as single solutions, whereas the different components of others are individually pumped and stream mixed at the injection point. There are several manufacturers of proprietary chemical grout formulations, especially those used for control of water movement within the soil or into underground substructures. For strengthening applications, especially where large quantities of grout are required, however, many specialty contractors tend to purchase the individual chemical components separately and mix them on the job site near the point of injection. Although some specialists tend to represent their formulations as something very special and "proprietary" (a word greatly overused, in the writer's opinion), the vast majority of grouts now used for strengthening of soil throughout the world involve sodium silicate as the base materi-

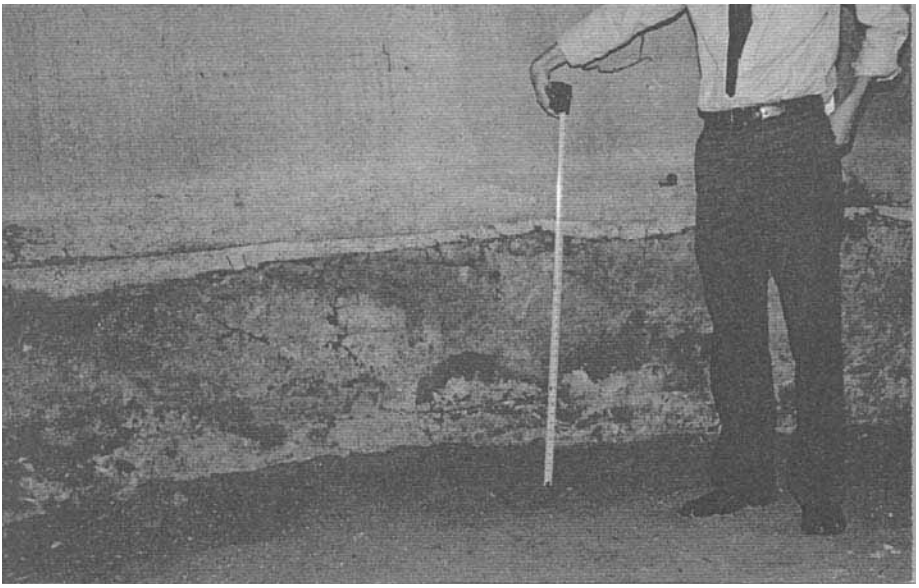

FIGURE 6B.3 Sandy soil solidified to a solid mass.

al. There are however, a wide variety of different reactant systems, most of which are well known and readily available.

With the increasing availability of ultrafine cements, many of which are able to readily penetrate the pores of even fine sands, suspension grouts of these cements are increasingly being used for soil solidification. They usually result in a stronger and more predictable solidified mass than do the chemical solution grouts, and are often less costly to apply. Also, unlike many chemical grouts, they do not lose strength over time.

European practice for soil strengthening continues to favor chemical solution and/or cementitious suspension grouts. Because solution grouts are usually much more expensive than their common cementitious counterparts, there is a natural tendency to use the latter if feasible. Recognizing that their grain size often exceeds that of the soil pore space, applicators have developed highly refined injection procedures wherein a discreet amount of grout is placed at regular intervals along a grout hole. The grout tends to fracture the soil but remains near its intended location, as only a small amount is injected at each interval. The amount of densification that results from any single hole is limited, as the quantities of grout injected are usually small and considerably less than typically used in compaction grouting. This is necessary, as once grout has created a fracture in the soil, it is virtually impossible to control its deposition distance or direction. Relatively close spacing of the grout holes and limitation of the grout quantity at each hole location is thus imperative in order to maintain control of the injection.

6B.2.3 Reduce Permeability, Water Control

Historically, cementitious suspension grouts have been used to stop the flow of water through cracks and joints in rock. The pore size of most soils, however, is significantly smaller than the width of typical rock joints, and except in the case of coarse sands and gravels, it is insufficient for intrusion of a common cementitious, particulate grout. Thus, water control in soils is almost always accomplished with either chemical solution grouts or suspensions based on ultrafine cement. Whereas hard rigid chemical gels are preferable for strengthening applications, some flexibility is usually desirable in water control work, and thus different chemical grout systems are in order. Dilution of the grout with groundwater is always a concern, and where rapid flow of the water occurs, the grout must be resistant to being washed away as well. This often means use of a rapidly setting grout. A variety of such materials are widely available that provide instantaneous setting, as shown in Figure 6B.4. These grouts are usually more chemically complex than those used for strengthening of soil. Some formulations present health risks and require special handling. They are thus most often obtained as proprietary systems from specialty grout producers.

6B.2.4 Stabilize and Reduce Expansion of Clay Soils

Unlike granular soils, clay consists of microscopically small, flat plate-like flakes. Clay platelets are generally colloidal in size, which means that they will float rather than sink in water. Water between the individual plates, thus contributes significantly to the volume of a clay soil. Because of the minute size of the individual particles, the movement of water in clays is very slow, and consolidation of clay soils, which results from expulsion of the water component, occurs slowly. Changes in the water content of clays can produce either expansion or shrinkage of the soil.

Mixing a source of cations such as calcium with clay has been shown to stabilize the water content, thus producing a more stable soil material. The most frequently used calcium source is lime, which is available in two forms—quicklime and hydrated lime. When these materials are physically mixed with clay soil, base exchange and pozzolamic reactions occur, which are fairly well understood. When lime slurries are injected at depth (Figure 6B.5), beneficial results are obtained; however the exact response is not predictable. Lime injection has been practiced in the United States for

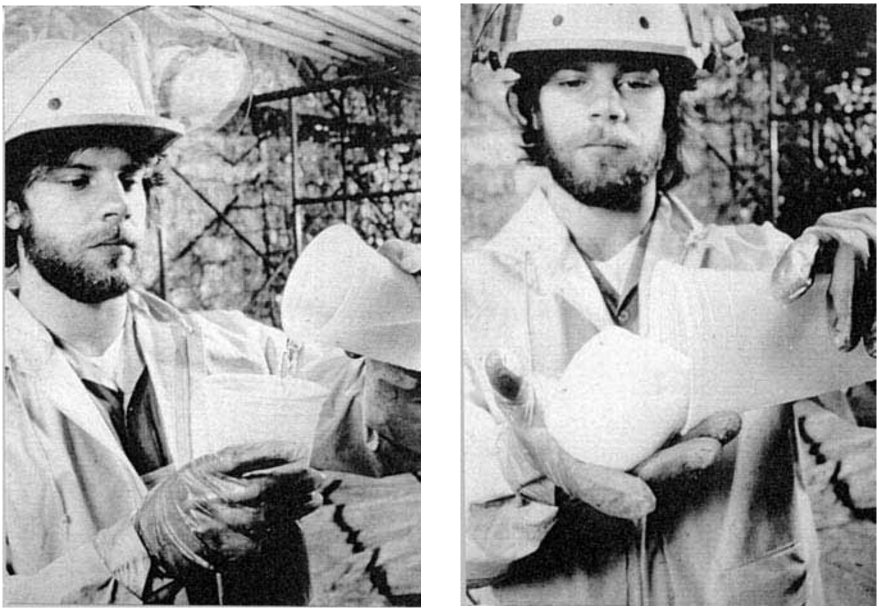

FIGURE 6B.4 Freshly mixed grout flows freely from one cup to the other (a) but gels midstream a few seconds later (b).

more than 50 years with varying degrees of success. The principal beneficial mechanism is believed to be an ionic exchange that improves the basic chemistry of the clay and facilitates stabilization of the soil moisture content, resulting in a more stable condition.

6B.2.5 Compensating for Lost Ground and Filling Large Voids

Grouting methods are often used to fill voids resulting form geotechnical construction in soil. Cementitious suspensions and slurries are widely used to compensate for lost ground adjacent to structures, resulting from loss of support; during soft ground tunneling; or in other underground construction. The work is usually referred to as *contact* or *compensation grouting*. Although the two terms are sometimes used interchangeably, *contact grouting* is preferable for reference to the filling of voids between the structure and the host formation, whereas *compensation* usually refers to larger voids, which may or may not be in contact with the structure.

Where massive voids, such as sinkholes or abandoned mines, are filled, neither specialized grout mixtures nor equipment are required, the work generally being performed with standard concrete pumps and using ready-mixed mortar or grout (Figure 6B.6). Grouting contractors often disguise concrete pumps with modified sheetmetal coverings and sometimes modified hoppers, to give the appearance of special technology. The fact is, however, that standard concrete pumps and ordinary ready-mix concrete and mortar mixes, as well as standard controlled low-density cementitious mixes, can perform quite adequately in such applications, which the writer prefers to refer to simply as *fill grouting*. Concrete technology is highly developed, and established ready-mix concrete suppliers are capable of furnishing mixes with a wide variety of special properties, using established admixture technology.

FIGURE 6B.5 Injection of lime slurry into a clay soil.

FIGURE 6B.6 Use of a standard concrete pump for filling a sinkhole with ordinary ready-mixed concrete.

6B.3 TYPES AND PROPERTIES OF GROUT

A wide variety of grout materials are available. These range from water-like fluids to very low mobility, stiff mortar-like cementitious mixtures, as shown exiting a grout hose in Figure 6B.7. Selection of the optimal grout material is one of the most important parts of any grouting program. Whereas some job requirements can be adequately accomplished with any of a number of different grout materials, others require very close control of a single one. Some grouting contractors are capable of successful application of many different types of grout, whereas others specialize in only one type of grouting or class of materials. As an example, there are a number of contractors who limit their work to water control grouting, using urethane chemical solution grouts. Others, although strongly favoring one particular procedure or class of material, will offer to perform the work with any that might be specified. Unfortunately, all too often, after a contractor has procured a job, he finds "need" to substitute his preferred grout material or system for that which was originally contemplated, even though it might not be in the best interests of the project (being either more costly or less effective). Available grouts can be divided into the following four classifications.

6B.3.1 Fluids and Solutions

These grouts are composed of two or more components that are either premixed and pumped as a single solution or stream mixed, wherein the various components are blended together as they enter the grout hole by way of siamese fittings at the collar of the hole. Some solution grouts are very thin water-like fluids, whereas others can have considerable resistance to flow. Most grout manufacturers and many grouting professionals refer to the *viscosity* of a particular grout, and infer that the in-

FIGURE 6B.7 Stiff mortar-like grout extruding from grout hose.

jectability is directly related to that property. This is incorrect, however, as viscosity is only one of the properties that contribute to a grout's injectability, which subject will be much more thoroughly discussed shortly. The injectability properties of a proposed solution grout must also be related to the grain size and resulting permeability of the formation to be grouted. As an example, a very highly penetrable grout can "run away" from its intended injection location in a very permeable formation, whereas a less-penetrable grout would remain where intended.

A search of the chemical solution grouting literature will uncover much discussion as to the effect that the soil permeability has upon the viscosity of the grout to be used. This can be quite misleading, however, as such data seldom considers pumping rates in its conclusions. As an example, the writer injected several chemical grouts with widely differing viscosities in a full-scale field test. The test was in a deposit of fine to medium sand containing trace amounts of silt. It was interesting to note that whereas an acrylamide grout with a viscosity of about 1 cps (similar to water) could be injected at a rate of about 7 GPM (27 LPM), a much more viscous sodium silicate based formulation, with a viscosity of about 18 cps, was able to be injected into the same formation, although at a slower pumping rate of about 2 GPM (8 LPM).

Setting times of chemical solution grouts can be instantaneous to several hours, although for most work it is desirable for the material to set soon after injection. This is especially true when working below the water table, or where the water is moving, in order to preclude dilution or washing of the grout beyond its intended place of deposition. Grouts that resist dilution are available and should be used where such a risk exists. Also, grouts are available with extremely short or even zero setting times, and in the case of some urethanes, an instantaneous expansive foam will develop when the grout first comes in contact with water. In instances where moving water is involved, these special grout materials, which can be quite expensive, become very useful and ultimately very cost effective.

It is important to understand the setting behavior of chemical solution grouts, especially where dilution is likely to occur. Some formulations will remain at or near their initial viscosity until shortly before setting (Figure 6B.8, Curve A). Other grouts will start to thicken immediately upon mixing, until they finally set (Fig. 6B.8, Curve B). Either of these setting behaviors can be advantageous, and might even be required, depending upon the particulars of the individual application. Some grouts are hydrophobic, that is, they repel water, while others are hydrophilic and attract water. Still others will immediately foam or gel upon contact with water. These properties can have a significant impact not only upon the injection operation, but the performance of the grout once in the ground. The ability to resist dilution, and/or extrusion or movement of the grout within the formation, or syneresis (a squeezing of the water from the gel), are also important criteria in selecting grouts to be used in saturated soils.

6B.3.1.1 Chemical Solution Grouts for Strengthening Soil

For the strengthening of soils, the most important parameters for selection of chemical solution grouts are safety, strength, and cost. Many of the chemical grouts historically used have been found to be toxic, and thus were removed from the market. In considering any new grout formulation, it is thus imperative to assess any possible risk that might exist in its use. Hard, rigid gels are desired as they provide the greatest stiffness as well as providing good strength. Cost can also be an important factor, because this type of work is often performed on a massive basis where very large quantities of grout are involved. With these factors in mind, grouts based on sodium silicate are the most frequently used for strengthening applications. Appropriately, the technical literature is filled with publications relative to the use of sodium silicate based grouts. Unfortunately, most of it is quite academic, and some downright misleading.

As an example, several publications present strength data for "sodium silicate grouts," but make no reference whatever to the reactant system. Whereas sodium silicate may represent the largest component of a chemical solution grout, it requires a reactant component to produce the hardened end product. There are many different reactant systems, which may consist of one or several components, as will be shortly discussed, and the properties of the resulting grouts will vary greatly with the different systems. The combination of sodium silicate with *some* reactant systems (such as for-

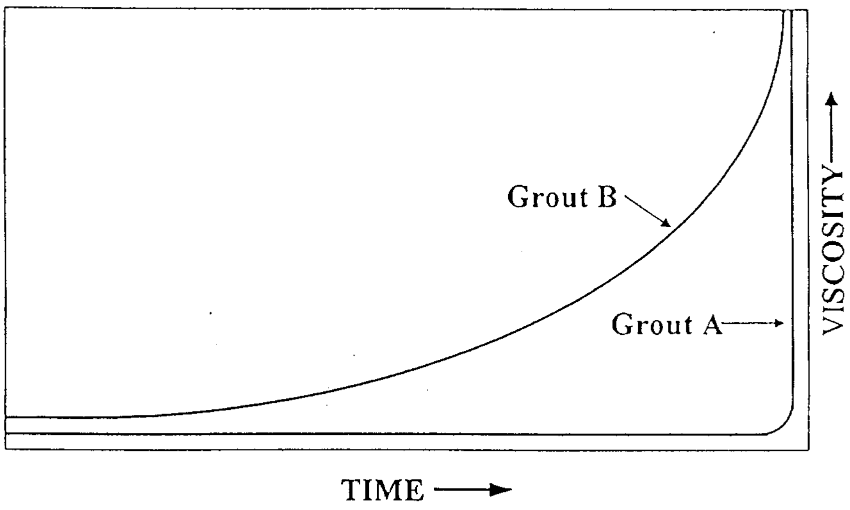

FIGURE 6B.8 Very different thickening modes for solution grouts.

mamide or inorganic acids) results in a grout that will break down and lose strength with time, while other mix combinations (such as dibasic esters or gyloxol) have proven to be durable over long periods of time. ASTM has produced a standard for the strength of grouted masses, D 4219, "Standard Test Method for Unconfined Compressive Strength Index of Chemical-Grouted Soils." This standard is addressed only to the "short term" strength, which certainly precludes its applicability to permanent installations.

Graf et al. (1982) reported on the long-term aging effect, of chemically grouted soil specimens that were aged from 9 to 11 years. They concluded that "Except for the very weakest samples, the strengths of the stabilized soils tested with no environmental change, showed no change or a modest increase over those measured after one to two years aging."

The strength of the sodium silicate concentration will also affect the final properties of the grout, including long-term durability. Some publications fail to recognize this crucial factor, or in some cases report incorrectly that sufficiently strong solutions either cannot be injected, or cannot obtain a satisfactory hardening time with a given reactant system. Bad results have been experienced in several instances where strongly diluted solutions were used. This not withstanding, the poor results were the result of faulty mix design, and not indicative of the behavior of properly designed sodium silicate formulations.

In 1972, the author reported results of a research program, which was ongoing for several years prior to 1971 (Warner, 1972). That work involved the testing of more than 2500 laboratory prepared specimens of chemically solidified soil, under a variety of both test and environmental conditions. The specimens were made with eight different chemical grout systems, and cured in three different environments. Sodium silicate was the base material in four of the systems. Included were strength test results of about 100 additional specimens, from sodium silicate grouted masses that were procured from a variety of actual field installations. These had been performed between 1965 and 1971, and included several permanent applications.

Although that work was completed a long time ago, many of the results remain pertinent. It was found that many different factors can drastically affect the indicated strength of the grouted specimens. It will vary enormously with different rates of load application. Faster loading rates will produce higher indicated strengths. Thus, comparison of reported strengths obtained from different reports are not valid unless the loading rate is known and identical for all reported results. The moisture level within a specimen also has a substantial influence upon the indicated strength. Dry specimens will always indicate a substantially higher strength than otherwise identical moist ones. But the most significant finding was that the strength that a given mass could withstand under continuous loading was but a small portion of that indicated by even relatively slow loading rates of laboratory compression tests.

The strength obtained under long-term continuous loads was referred to as the *fundamental strength*, and was within a range of 20% to 80% of the *ultimate* strength, indicated by the standard loading rate of the laboratory unconfined compression test. That rate was 20 psi per second as per the provisions of ASTM D 1663. The ultimate and fundamental strengths of several of the grouts evaluated at 5 days are shown in Figure 6B.9, which is reproduced from Warner (1972). Therein, G.V.S., SIROC, and Modified Earthfirm all contain sodium silicate as the base component. The writer has had extensive experience with the Modified Earthfirm, and has found the long-term, fundamental strength of that formulation, which is provided in Table 6B.3, to be on the order of 70% of ultimate.

Another finding of the research was a large variation of the total strain at rupture during loading. The poorest performing material was AM-9 Acrylamide grout, which had a strain of 19.7% at rupture. Although not now easily available, in earlier times this grout was often recommended for strengthening applications. Obviously, unless the solidified masses were fully restrained, less than satisfactory performance could be expected with such great strain deformations. The strain behavior of several different grouts is shown in Figure 6B.10.

Many of the formulations that were reported on were proprietary at the time of publication in 1972. Such rights have now expired, and the mix contents of the three main silicate base formulation are given in Tables 6B.1, 2, and 3. Although this research was performed a long time ago, sadly, nothing comparable has been since produced. Although a limited amount of strength data has been reported, important test criteria, such as loading rate, curing regime, etc., either varied from the previous work, or were not even reported, so that meaningful comparisons are not possible.

In 1983, ASTM released their standard, D4219, "Standard Test Method for Unconfined Compressive Strength Index of Chemical-Grouted Soils," which covers the determination of the *short-*

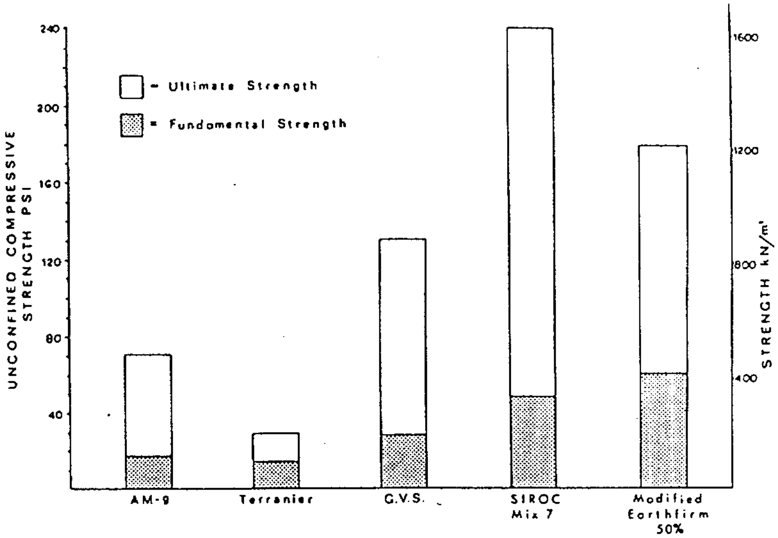

FIGURE 6B.9 Ultimate and fundamental strengths of five day cured specimens of various chemically solidified soils. (From Warner, 1972.)

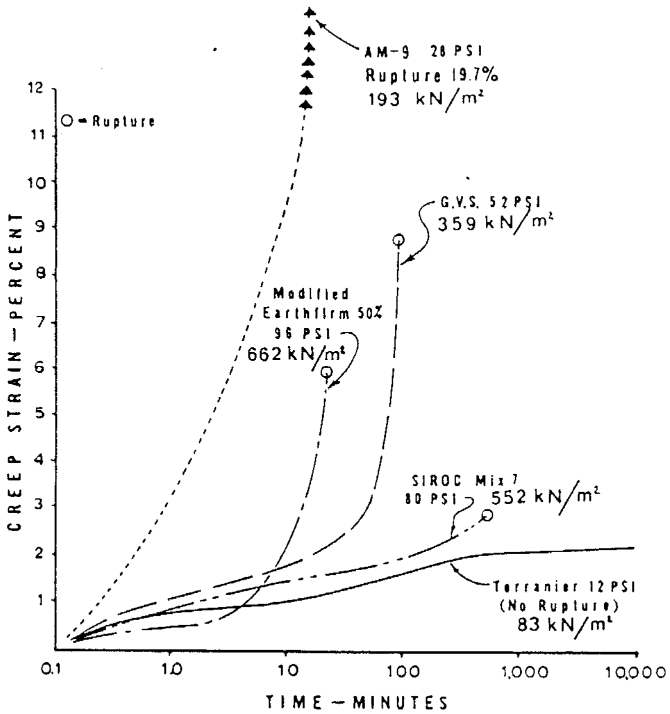

FIGURE 6B.10 Accumulated strain at rupture of five day old specimens loaded to 40% of their ultimate strength. (From Warner, 1972.)

term unconfined compressive strength. That standard provides for a constant but unspecified rate of load or deformation. The only limiting requirements are that failure is not to occur in less than two minutes and the maximum strain rate not exceed 1% per minute. Unfortunately, such wide latitude in the allowable loading protocol precludes meaningful comparison of different specimens, even though evaluated in conformance with the standard, unless identical loading rate were used and noted. An additional provision of the standard is that the reported strength is to be that obtained at rupture of the specimen, or an accumulated strain of 20%. This is a very large strain, and it is unlikely that many structures partially supported by such ground would tolerate the resulting deformations. It must also be noted that the standard pertains only to the *short-term* strength, and thus is not applicable to long-term or permanent strengthening.

There has been much discussion as to the appropriateness of the unconfined compression test

TABLE 6B.1 Mix Constituents for G.V.S. Chemical Solution Grout.

Ingredient	Laboratory batch	Field batch
Sodium silicate	350 ml	50 gallons
Calcium chloride	4.5 grams	6.5 pounds
Glyoxal	35 ml	5 gallons
Water	315 ml	45 gallons

TABLE 6B.2 Mix Constituents for SIROC Mix 7.

Ingredient	Laboratory batch	Field batch
Sodium silicate	350 ml	50 gallons
Formamide	63 ml	9 gallons
Sodium aluminate	4.3 grams	6 pounds
Water	287 ml	41 gallons

TABLE 6B.3 Mix Constituents for Modified Earthfirm, 50%.

Ingredient	Laboratory batch	Field batch
Sodium silicate	350 ml	50 gallons
Calcium chloride	6.72 grams	8 pounds
Ethyl acetate	14 ml	2 gallons
Water	336 ml	48 gallons

for such evaluations, and suggestions that a triaxial test, which provides lateral confinement to the test specimen, would be more suitable. This view has some validity *if* the solidified mass of the particular application is to be continuously confined. A large amount of chemical solution grout solidification, however, is performed for the retention of excavated soil faces, as illustrated in Figure 6B.11. In such cases, lateral confinement does not exist. Even in those situations where the grouted mass is completely confined, extensive laboratory studies by Ata and Vipulanandan (1999) have established little if any effect of the confining pressure on either the shear strength or

FIGURE 6B.11 Cohesionless dune sand solidified with a chemical solution grout enable support of vertical excavation.

modulus of the grouted soil. Use of the much easier and less costly unconfined test is thus certainly pertinent and justified.

Although, in earlier times, the use of grouts for strengthening applications other than those based on sodium silicate was not unusual, most of the contenders have fallen, due to environmental, economic, or performance limitations. Today, the vast majority of chemical solution grout strengthening applications involve sodium silicate based grouts. While glyoxal, formamide, and ethyl acetate, along with calcium chloride and other salts, continue to be used, the reactants that are now used most frequently, are dibasic and tribasic esters. These can be more easily combined with the sodium silicate, and are much simpler to compound and use than the multicomponent mixtures given in Tables 6B.1, 2, and 3.

Dimethyl esters, which are the most economical and commonly used, are a by-product of the manufacture of nylon. As recovered, they are a combination of dimethyl succinate, dimethyl gluterate, and dimethyl adipate. The proportions of these three components can be quite consistent or it can vary widely, depending upon the source. Whereas for many uses of the solutions, the exact proportioning is of little consequence, such is not the case for use in sodium silicate grouts. Both the gelling time of the grout and strength of the resulting injected mass are affected by the exact proportion of the three components. In order to obtain consistent setting properties of the resulting grout mixture, as well as uniform strength of the grouted mass, the tolerance of the components should be reasonably consistent.

The actual mechanism by which a sodium silicate grout hardens is a reaction between the sodium silicate and an acid. In the presence of water, the esters hydrolyze to acid, which then converts the silicate to a solid. The rate of hydrolysis of the three ester types, and thus the grout gel time, varies widely. Dimethyl succinate hydrolyzes in minutes, gluterate in hours, and adipate in days. The proportions of these three ester types in the particular reactant solution being used can thus have a dramatic influence upon the performance of the mixed grout. Temperature also has an effect upon setting time, but this is generally not a problem in the environments in which grouts are usually used. It can become a significant consideration, however, where ambient temperature extremes exist in either the soil or the surroundings.

The strength, stiffness, and the durability of sodium silicate grouted soils are all related to the concentration of the sodium silicate base component as well as that of the reactant. Sodium silicate solutions are available in a variety of densities and viscosities. The sodium silicate most used in the United States has a density of about 41° Be and a weight of about 11.6 pounds per gallon (5.3 kg/l) at 68° F. It is highly alkaline, with a pH of about 11.3, and as delivered is similar to light syrup, with a viscosity of about 180 cps. It usually comprises from 30% to 80% of the total grout volume. To assure long-term durability for permanent applications, a minimum solution of sodium silicate base of 50% should be used. The proportion of basic ester reactants is usually within a range of 5% to 10% of the total solution, although for strong and durable applications 8% to 10% should be used. The remainder of the grout mix is water.

A typical mixture containing 50% sodium silicate and 10% diester will have a viscosity on the order of 20 cps. Whereas this is much greater than some widely touted low-viscosity grouts, they are nonetheless, quite penetrable. The penetrability can also be enhanced by the judicious inclusion of a surfactant, which effectively reduces the surface tension within the sodium silicate grout. Although grouts employing the basic ester reactants are now widely used, the author is unaware of any documented data relative to their long-term durability, and would suggest that they not be used for permanent solidification until suitable long-term performance data become available.

6B.3.1.2 Chemical Solution Grouts for Water Control

Whereas hard rigid gels are best for soil strengthening, some flexibility of the gel is often desirable in water control grouts. Also, although long gelling times are often advantageous for strengthening operations, very rapid setting times can be an essential property when grouting in moving water. Those grouts, which are widely used for strengthening, are thus often not best suited for water control work. There are two main generic classes of chemical solution grout that are particularly suited for water control grouting. These are polyurethane and acrylamide/acrylate resins. There are several different formulations within each of these broad categories. The polyurethanes are usually used alone, whereas the acrylamide/acrylate systems are sometime filled with other finely divided materials such as diatomaceous earth, silica fume, and ultrafine cement.

There are many different types of polyurethane grout, which provide a wide range of different properties. They all react when mixed with water, to form either a gel, solid, or foam. The strength of the gel, or density of the foam, is dependent upon the particular formulation and the amount of water with which it has been mixed. Temperature also has an influence upon the final properties, though it is not as significant as the other variables. Some formulations can have a rather complete reaction upon simply coming in contact with water, whereas others require complete mixing therewith.

Urethane grouts can be divided into two main classes, *hydrophobic* and *hydrophilic*. Hydrophobic formulations repel water upon hardening. For this reason, their bond to wet surfaces is not very great. This is not much of a problem within a maze of interconnected soil pore spaces, but can be a limitation where an obvious crack in rock or concrete, or a void at a soil to hard surface interface, is to be filled to stop water flow. Some hydrophobic formulations will immediately foam on contact with water, as shown in Figure 6B.12. This can be very useful in stopping flows of moving water. Because reacted hydrophobic polyurethanes contain no outside water, they are generally free of shrinkage, even when allowed to dry.

Hydrophilic formulations also react with water but continue to attract it after completion of the initial reaction. These formulations will thus continue to expand if possible, forcing the resulting gel into pore spaces and voids, beyond those originally filled. Unfortunately, the strength of the grout is reduced as further water is absorbed. Obviously, they will develop a much greater bond to adjacent surfaces as long as they are not diluted excessively. However, they are subject to shrinkage upon drying out. They are thus best not used in areas that will not be in constant contact with water. Some

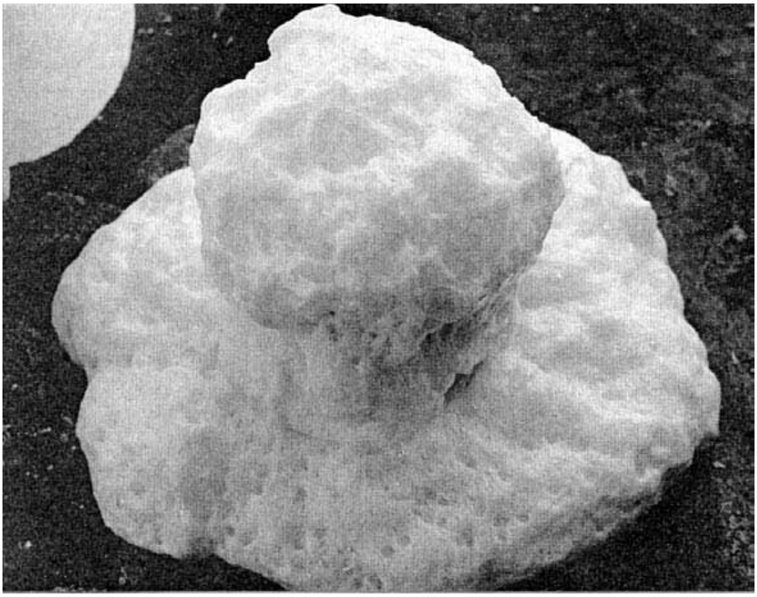

FIGURE 6B.12 Water dropped onto a small amount of hydrophobic urethane grout in the bottom of container resulted in immediate formation of a closed cell foam.

hydrophilic formulations require a minimum amount of water in order to react fully. This is typically delivered into the grout, during injection, by means of a suitable dual pump proportioning system.

Urethane grouts are of two different categories chemically—those based on toluene diisocyanate (TDI) and diphenylmethane diisocyanate (MDI). The designations TDI and MDI are universally used and well-established terms, and are often used alone for reference. The TDI based formulations, which usually are hydrophilic, will combine with all available water. The result will be a gel or flexible foam, the strength of which will depend upon the proportion of the combined water. They stick tenaciously to wet surfaces and are very hard to remove or clean up should they be deposited upon an exposed surface. As illustrated in Figure 6B.13, spills of hydrophilic urethane grouts are extremely difficult to clean up.

MDI formulations are generally hydrophobic, and while they require water to react, they will only accommodate a specific amount thereof, and will displace any excess. They will thus often not develop a good bond to wet surfaces. They can be formulated so as to provide either flexible or rigid foams, or solid masses.

Although the TDI liquids are generally safe to handle, the resulting vapor pressures in the air exceed established threshold limits. Full protective precautions are thus required with their use. MDI mixtures, on the other hand, have relatively lower vapor pressures, and are thus less hazardous. With either type, however, airborne droplets, such as might result from a leaking delivery line, are hazardous, although the cured gels are not. The safety aspects of working with urethane grouts are thoroughly covered in "Recommendations for the Handling of Aromatic Isocyanates" (1976).

Mixed acrylamide/acrylate grouts are water-like solutions that provide very high penetrability. They are capable of exceptional control of the gelling time. Upon setting, the gel is very soft, enabling it to be pumped out of the delivery system should it gel prematurely. While these factors are benefits from the injection standpoint, they can be limitations for the injected grout. The resulting gel is quite weak, and subject to extrusion within any significant voids or defects in the soil. This

FIGURE 6B.13 Hydrophilic urethane grout spills are extremely difficult to clean up.

limitation can be offset by inclusion of filler materials; however, the injectability goes down as fillers are added. This group of grouts finds its largest single use in the repair of leaking sewers and pipelines. Although the chemistry of the gelled grout is essentially the same, the origin of these materials is different.

Acrylamide grouts were one of the earliest commercially available chemical solution grout systems and were marketed by the American Cyanamid Company under the trade name AM-9. They were first marketed in 1955 and were extensively used for water control applications until 1978, when they were suddenly removed from the market. This was due to their manufacture being discontinued following a toxicity problem in Japan, which resulted in a ban on their use in that country. Although the cured grouts are not toxic, the individual components are, so the unmixed components or any mixed solution that fails to gel, due, for instance, to dilution after injection, could present a serious risk.

Although not widely promoted, acrylamide grouts are still available in the United States, and continue to see extensive use for sewer maintenance. They are generally sold only to firms that are properly equipped and have personnel who are especially trained in their safe use. The base material is available in either a dry granular form or a fluid slurry. The slurry is considered less hazardous than the dry form. The grout consists of separate solutions of the base material and the reactant system. These components are proportioned and impelled by separate synchronized pumps to the point of injection where they are combined and stream mixed (Figure 6B.14). Both the gel time and strength can be varied by adjusting either the strength of the starting solutions or the final mix ratio.

In order to enable variation of the final mix ratios, a variable proportioning pumping system must be used. The reactant portion of the grout is highly corrosive, so the mixers, pumps, and appurtenances on that side of the pumping system must be of plastic or stainless steel. Most of the work now being done with these grouts involves sophisticated pumping systems, requiring minimal handling of either the unmixed ingredients or the resulting grout.

FIGURE 6B.14 Stream mixing of grout at the point of injection.

Acrylate grout was developed as an exact replacement for the acrylamide grout when AM9 was taken off the market. It uses the same catalyst system and offers many of the advantages of the acrylamide mixtures. The acrylate, which forms the base solution, however, is a prepolymer and is not neurotoxic as is the acrylamide monomer. The mixing and pumping equipment requirements are the same for both grout types.

6B.3.2 Fluid Suspensions

Fluid suspension grouts were the first to ever be used, and they continue to be widely employed. The largest area of use for these grouts continues to be the filling of cracks and joints in rock, although their use has been extended greatly and includes controlled injection into soil. In the simplest form a suspension of ordinary portland cement and water is mixed. If all of the cement particles are to be thoroughly dispersed, high-shear mixing is required and the mixed grout must be continuously agitated to maintain the individual grains in suspension.

Cement grout mix design is expressed as the ratio of water to cementitious solids, either by weight or by volume. Volume measurements have been the most often used traditionally, as they were very easy to make when using common bagged cement, which contains one cubic foot of dry powder per 94 pound bag. Where bulk cement is used, proportioning by weight is more convenient. In the United States, grout mixtures as thin as 10:1 by volume have been extensively used historically. Many, including the writer, consider such thin grouts to be little more than dirty water, and extensive testing and research by the author and others (Houlsby, 1982; Weaver, 1991) have shown such thin mixtures to provide little durability. Current thinking suggests that thicker mixtures (maximum W/C 3:1 by volume) are far more appropriate for most applications. In a well-documented discussion of appropriate water to cement ratios, Houlsby (1982) concluded that W/C 2:1 by volume has been used on thousands of projects and is the optimal for general use.

Although unmodified common cement–water suspensions continue to be widely used, they suffer from settlement of the solids, resulting in accumulation of mix water at the top of unagitated grout. This is known as *bleed* and is an important factor to consider in any grout formulation. In this regard, the amount of shear energy imparted during the mixing of the grout has an important bearing upon the resulting bleed. When cement and water come together, the cement grains tend to clump or floc together. To minimize the amount of bleed of a particular grout, high shear mixing will break up the flocks and separate the individual cement grains. This factor was thoroughly discussed by (Kravetz 1959). The essence of his conclusions, as reported by Houlsby (1990) are:

1. Cement grains, when mixed with water, tend to aggregate and form clumps. This slows the wetting process, as does air attached to the grains. The effect of high-speed shearing or laminating, plus the centrifugal effect, is to thoroughly break up the clumps and to separate air bubbles. As a result, each individual grain is rapidly and thoroughly wetted and put into suspension.
2. During cement hydration, needlelike or springlike elements of hydrates form on the superficial layer of each wetted grain of cement. In a high-speed mixer, the laminating effect and high-speed rotation keeps breaking these hydrates away from the grain of cement, thus exposing new areas to the water and consequently causing the formation of new elements. These hydrate elements are of colloidal size, and as the amount of these elements in the mixture increases, the grout becomes colloidal in character.

In reality, virtually no common cementitious suspension grout is truly colloidal, as even well-dispersed cement grains will settle in water unless special admixtures are used, as will be subsequently discussed. High-shear mixing will greatly reduce and can in thick grouts nearly eliminate bleeding. In some places, but generally not the United States, a grout that exhibits little or no bleed is regarded as "stable." Stable grouts are generally defined as those which exhibit a total of no more than 5% bleed. European practice commonly calls for "stable" grouts, which typically include a few percent of bentonite to act as a suspension agent. Whereas bentonite inclusion will tend to lessen the cement grain settlement and resulting bleed, there are significant disadvantages to its use. Modern admixture technology offers a variety of different compounds that can minimize bleed without the undesirable aspects of admixed bentonite. These will be subsequently discussed in more detail.

6B.3.2.1 Soil Strengthening with Cementitious Suspensions

As previously discussed, the grain size of ordinary portland cement grouts is too great to allow injection into all but the coarsest of sands and open gravels. Extending work of the Corps of Engineers, Technical Memorandum 3-408 (1956), and the work of a number of others, relative to the permeation of soils with cement grouts, King and Bush (1961) concluded that grout would pass freely through a granular mass if the diameter of the particles in the grout was not greater than 6.7% that of the material to be grouted. Their conclusions were the result of an extensive study, which considered not only the largest sized particle in the grout, but also the variation and distribution of grain sizes, as well as consideration of the pore structure to be grouted.

The work was based on the premise that whereas two grains of cement wedged across a pore opening would form a stable bridge, and thus cause blockage, three grains would not be as stable. Although their work was based largely upon spherically shaped particles in the mass to be injected, it did consider the variation in pore structure that could be expected. They performed a number of injection tests in which a neat cement grout was injected into a specimen of natural, round-grained aggregate. Although they included many aggregates in their test program, the finest was a sand, which was completely retained on a No. 8 sieve but passed 100% through a No.4 sieve. It should be noted that currently available common cements are ground somewhat finer than the cements evaluated by King and Bush.

In more recent times, there has been considerable discussion by a number of authors relative to the filling of soil pore spaces with particulate grouts. The discussion has been concerned mainly with mixtures containing ultrafine cement. Theoretical ratios of the grout particle size to the grain size and thus porosity, of a given soil into which a particular cement grout can be injected are emphasized. There has not been a great deal of agreement as to the appropriate ratios, and especially whether they should be based on the mean or maximum particle size of the cement. Although grain size has been emphasized, it has been the author's experience that grain size alone is only one important aspect. The grain shape and surface condition, as well as the overall rheology of the grout, especially the thorough dispersion of the cement, are all significant. Also, of importance are the grout pumping rate and sustained pressure level.

Suspension grouts are often injected under fairly high pressures, and pressure filtration, that is, loss of water from the solids due to the injection pressure, must not be allowed to occur. This is best prevented by inclusion of a high-range water-reducing admixture in the mix, combined with high-shear mixing action, so as to maximize dispersion, and uniform wetting of the cement particles. Whereas in the filling of fissures or voids, the maximum grain size of the grout is not as important as the overall grain size distribution, which will be discussed in more detail shortly, grain size is crucial when injection is into a granular deposit. This is because the particles can form a filter cake where they first enter the deposit, with the largest particles being restrained near the grout hole, followed by those which are progressively smaller, until permeation all but stops.

This is an important consideration, in that ultrafine cement suspension grouts, which are highly penetrable and can form nearly colloidal mixtures, are seeing ever-increasing use for the solidification of sandy soils. Shimoda and Ohmori (1982) described the development of a new ultrafine cement in Japan and reviewed laboratory research as well as two significant application case histories. Suspension grouts made with the then new cement were injected into fine to medium sands.

In more recent times, several manufacturers have introduced ultrafine cements, which are now readily available. Figure 6B.15 shows the grain size distribution for common portland as well as several ultrafine cements. It should noted that there are no standards regulating the actual grain size distribution for common cements. The curves shown in Figure 6B.15, for type I and III cement, are for that produced in one particular batch at a given plant. ASTM C 150, "Standard Specification for Portland Cement," specifies only the Blane fineness, which is a measure of the specific surface area of all of the grains in a given volume of cement. Type III portland cement is required to be finer than type I or II; however, the maximum grain size or grain size distribution, are not specified.

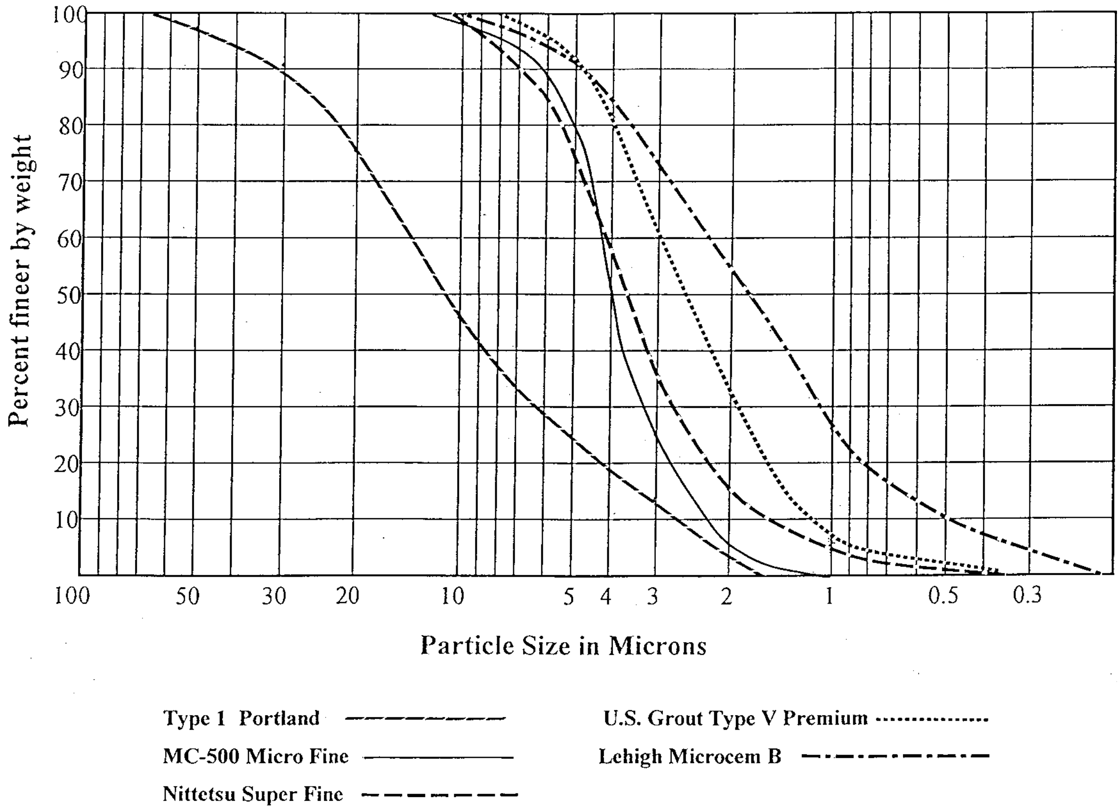

FIGURE 6B.15 Particle size distribution of several cements used in fluid suspension grouts.

There are two main types of ultrafine cement: those based on slag and those based on portland cement. Although the two are comparable in many respects, there is a fundamental difference in the span of hydration and the resulting setting time. Because of the very high specific surface of ultrafine cements of portland cement origin, hydration activity is high, resulting in rapid setting and strength gain. For this reason, retarders such as citric acid are sometimes included. Portland cement–citric acid combinations are extremely sensitive to temperature changes, so the proportions of the additive must be matched to the temperature of both the working environment and the medium being injected. Problems with flash setting and difficulties with both setting time and rate of strength gain have been reported.

Cements based on slag, however, tend to set and gain strength much slower than their portland cement cousins. Thus, retardation is seldom a problem, but accelerating admixtures may be required in some instances. Well-established accelerators are available and their behavior is quite predictable, so setting times are not much of a problem with the slag ultrafine cements.

These factors were well illustrated during a demonstration, which was part of the 21st Annual Short Course on Fundamentals of Grouting, held in Denver, Colorado, September 12 to 17, 1999. Therein, vertical, transparent tubes, eight inches in diameter and five feet high, were filled with sand (Figure 6B.16). The sand was of quartz origin and was graded such that 100% passed a number

FIGURE 6B.16 Sand-filled columns used to evaluate cement grout penetrability.

30 mesh sieve and 40% passed a number 40 sieve. It was carefully poured into the tubes such that all tubes were filled equally and had equal density.

Five different grout mixtures were injected into the sand, from the bottom of the columns. The injection was made at a pressure of 10 psi, which was maintained for twenty minutes on each column. The degree of penetration of the various grouts was noted. All of the grout mixes had an identical water to cement ratio of one. Mixture content and penetration rates for the various grouts are shown in Table 6B.4. Interestingly, the greatest degree of penetrability was not achieved with the finest cement. This is consistent with real-world experience on many projects where ultrafine cement grout has been injected into sandy soil. Obviously, there remains much to be learned about these cements, and certainly, particle shape and surface characteristics are major contributors to the penetrability of grouts in which they are used.

The thickest grout that can be readily mixed with the mixers commonly used in grouting is on the order of 0.5:1 by volume. Lower W/C ratio grouts result in a mixture with pastelike consistency, generally referred to as a slurry. Slurry consistency grouts are sometimes used for filling small voids (0.3 to 3 inch) as well as for filling of controlled fractures in soil. They have also seen some

TABLE 6B.4 Penetration of various ultrafine cement grouts into experimental sand columns.

Grout mix/cement type	Penetration from base (inches)
Type I Portland	4
Type I Portland + 1% naphthalene superplasticizer	8
Portland based ultrafine	9
Slag based ultrafine	60 (flow from top in 7 minutes)
Portland–pumice based ultrafine	57

use in filling of large voids, such as abandoned mines (although this is inappropriate in the writer's opinion). Slurries are generally too thick to be mixed in commonly available high-shear mixers. They are thus most often prepared in horizontal-shaft paddle mixers, such as those commonly used to mix plaster and mortar, and thus are often particularly sensitive to bleed.

6B.3.3 Mortarlike Low-Mobility Grout

Very stiff mortarlike grouts were first developed in the United States in the mid 1950s. The original use of these very low mobility grouts was for the then developing technology of compaction grouting. With the advent of the modern concrete pump in the 1960s, it became practical to pump and place under pressure typical concrete and mortar mixes. In recent times, admixture technology has provided the ability to alter conventional concrete and mortar mixtures, creating a wide range of special properties. Of these, greatly increased plasticity, cohesion, and resistance to bleed are most germane to grouting.

Although many consider all mortarlike grouts to be equal, subject only to the slump, as determined by ASTM C 143, such thinking is erroneous. As an example, the low-mobility grouts used for compaction grouting must exhibit high internal friction in order to provide placement control and prevent hydraulic fracturing of the soil into which they are being injected. Conversely, conventional concrete and mortar mixtures ideally possess relatively low internal friction, in order to provide maximum workability. Stiff mortarlike grouts of the same slump are not necessarily equal in mobility when under pressure, either in the delivery system or in the ground. Whereas slump by itself may be a sufficiently accurate measure for conventional concrete or mortar, and whereas such mixes are quite satisfactory for fill grouting of large voids, the mix requirements for compaction grouting are much more demanding, as will be discussed subsequently.

6B.4 GROUT RHEOLOGY

Simply stated, rheology is the study of the deformation and flow of materials. Perhaps more germane to grouting, the American Concrete Institute, in their publication ACI 116 *Cement and Concrete Terminology* (1990), defines rheology as "The science dealing with flow of materials, including studies of deformation of hardened concrete, the handling and placing of freshly mixed concrete, and the behavior of slurries, pastes, and the like." Satisfactory rheological properties of a grout in both the as-mixed as well as in the hardened state are fundamental to successful completion of any grouting project. The mixed grout must provide appropriate properties to enable proper injection and travel within the particular formation to be improved. Obviously, the durability and long-term performance of the hardened grout is fundamental to achieving the intended performance. The flow properties of grout materials vary widely, and a basic understanding of both fluid and plastic flow is requisite to either the study or understanding of grout flow.

Many grouts are subject to thixotropy, which is the performance of a material as an immobile paste or gel when at rest, but as a fluid when energy is exerted on it. A common example of thixotropic behavior is the pouring of catsup from a bottle. The catsup has a high resistance to flow out of the bottle at first attempt, but upon rapid shaking, flows readily. Many grouts are thixotropic in that they require a positive pressure of some magnitude to initiate movement within the delivery system, but flow freely at lower sustained pressure, once movement is initiated. This factor is important in considering allowable injection pressure for a given application, and an adequate initial pressure level to start flow must be provided. The flow behavior of fluids is a complex science. Here, only those properties that are fundamental to a basic understanding of grouting will be explained in as simple a manner as possible. Those who might desire a more technical explanation are referred to Tattersall and Banfill (1983) and Mehta and Monteiro (1993).

Fluid and paste flow are described as either Newtonian or Binghamian. In Newtonian flow, the shear stress, which is the force required to move the fluid, is essentially constant, regardless of the rate of movement, which is technically referred to as the shear strain. Water is an example of a Newtonian fluid. Bingham fluids, on the other hand, possess some thixotropy, and require a measurable force (shear stress) to start movement. The magnitude of that force is usually referred to as the Bingham Yield Value, and sometimes, in reference to grout, the *cohesion*. With such grouts, the shear stress typically increases as the rate of shear (shear strain) increases. Typical stress–strain curves for both Newtonian and Bingham flow are shown in Figure 6B.17. The slope of the Bingham curve indicates viscosity, which property will be dealt with in more detail later. Whereas some chemical solution grouts perform as Newtonian fluids, most fluid suspension and other grouts exhibit Binghamian behavior.

Behavior of non-Newtonian grouts, is not always as simple as indicated by the straight line relationship of the applied shear stress and the shear rate or strain, as shown on Figure 6B.17. Although the relationship of shear stress to shear rate can remain constant, as shown, it can also vary either up or down, depending upon the tendency of the material to thicken or thin with an increase in the shear rate. Figure 6B.18 demonstrates an instance of shear thickening where resistance to flow (pressure) increases with an increase in shear or flow rate. Some grouts will behave quite differently, wherein their resistance to flow will decrease as the shear or flow rate increases (shear thinning), as shown in Figure 6B.19.

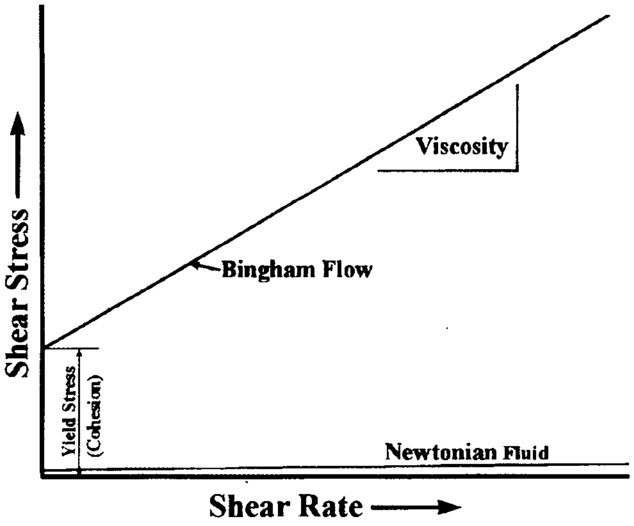

FIGURE 6B.17 Newtonian and Binghamian flow indicating Bingham yield stress (cohesion).

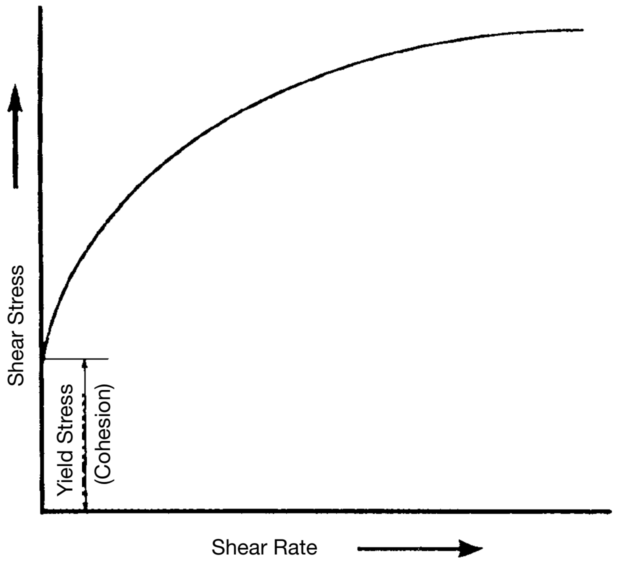

FIGURE 6B.18 Shear thickening.

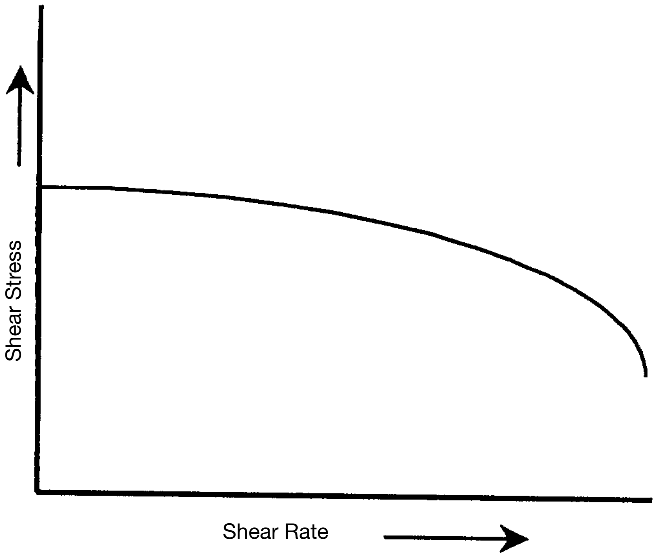

FIGURE 6B.19 Shear thinning.

Although incorrect, all too frequently only one or a few of the rheological properties pertaining to a particular grout mixture are used to describe the performance of that grout. One of the most frequent errors, is using the *viscosity* alone to define the injectability, when in fact knowing the viscosity is of little value unless other important properties such as surface tension and the resulting wetability are known. Grout rheology is very complex, and one must consider many important properties. The writer suggests that the use of all-inclusive, simple terms to describe the properties of a grout are of more practical value, and recommends the descriptive terms *mobility*, for relatively thick grouts, and *penetrability*, for more fluid grouts, that are intended to fully permeate soil or rock. Each of these terms include many rheological influences and effects that must be evaluated in total in order to rationally evaluate the propriety of that grout for a particular application.

6B.4.1 Mobility

Mobility denotes the ability of a grout material to travel through the delivery system and into the desired voids or intended deposition area within the geological formation being grouted. Of, equal importance is the ability to limit travel, so as to *not* flow beyond the zone of desired deposition. Thus, for proper performance, the grout should be of sufficient mobility to penetrate and fill the *desired* geotechnical defects, but it must be sufficiently limited so as to *not* flow beyond the desired injection zone. When injection is made into soils, the mobility will also affect the manner in which the grout behaves during injection.

When grout is placed in soil under pressure, it acts in one of three ways:

1. As a penetrant filling pore space, if the pumping rate is equal to or less than the rate at which the pore structure will accept the grout
2. As a fluid causing hydraulic fracturing of the formation if the pumping rate exceeds the permeation rate, or if the grout consistency does not allow it to permeate
3. As an expanding mass pushed by fluid-like pressure at the source of injection, so as to compress or compact the soil (Warner et al., 1992)

A common misconception is that a thick grout is always of low mobility, whereas a thin grout is always highly mobile. Depending upon the individual mix constituents, thin grouts can be of relatively low mobility, whereas even very thick low slump grouts can be highly mobile and behave in the soil as fluids. An example of the former would be fluid suspension grouts that contain blocking agents such as sawdust, mill feed, etc. (Technical Memorandum 646, 1957). Prior to development of the ability to pump low-mobility plastic consistency grouts, such low viscosity fluid grout mixtures, designed to restrict grout travel, were quite common.

Even very thick or essentially no-slump grout mixtures can be of sufficiently high mobility as to act like a fluid when injected into soil. Such behavior would typically be expected of grouts containing clay components or admixtures such as some concrete pumping aids. When a grout injected into soil behaves as a fluid, causing hydraulic fracturing, control of the grout placement is lost, and negative occurrences are likely. This subject will be much more extensively discussed subsequently in relation to compaction grouting.

In 1980, the American Society of Civil Engineers published the "Preliminary Glossary of Terms Relating to Grouting," wherein they defined compaction grout as:

> COMPACTION GROUT—Grout injected with less than 1 in (25 mm) slump. Normally a soil–cement with sufficient silt sizes to provide plasticity together with sufficient sand sizes to develop internal friction. The grout generally does not enter soil pores but remains in a homogeneous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures, or both."

Although, a standard test for "slump" was not defined by ASCE, many grouters have assumed that the well-established slump cone used in ASTM C 143, (see Figure 6B.20), which is the most common

FIGURE 6B.20 ASTM C 143 slump test in progress.

slump test, was intended. Many have ignored the inclusion of "sand sized" and "internal friction" in the definition, and defined any grout with a low slump using the C 143 slump cone to be appropriate.

Notwithstanding the definition by ASCE, or the interpretation by grouters, the *slump* of a grout alone, is *not* a valid measurement of its rheological properties or appropriateness for compaction grouting, or, for that matter, any grout application. The test was originally developed for appraising the workability of concrete that includes only clean sands and large aggregate. Even for use with that material, for which the test was developed, ASTM states in their document, ASTM Special Technical Publication 169B (1978),

> Slump Test—The slump test . . . is the most commonly used method of measuring consistency or wetness of concrete. It is not suitable for very wet or very dry concrete. It does not measure all factors contributing to workability, nor is it always representative of the placeability of the concrete.

A mixture with a slump of only one inch is certainly "very dry," and is a mixture that, according to ASTM, the slump test is "not suitable" for. Typical low-mobility grouts, such as those used for compaction grouting, contain silt-size particles, which tends to make them sticky. It is nearly impossible to properly fill the slump cone with such material, let alone obtain repeatable results. Unless one is dealing with concrete that is not "very wet or very dry", the slump test should not be used, and it most certainly is *not* appropriate for judging the rheology of most thick mortar-like grout mixtures.

6B.4.2 Penetrability

Penetrability defines the ability of a grout to permeate a porous mass, such as a sand or soil, or fill fractures or small voids. As previously discussed, viscosity alone is sometimes equated to the penetrability of a given grout. In actual practice, however, penetrability of a grout into a given soil is dependant upon a combination of viscosity *and* wetability. Within the viscosity ranges commonly found in solution grouts, wetability, which is defined by the contact angle of a drop of the fluid with the receiving surface, is a far more important and significant property than viscosity. Wetability is a function of the surface tension between the formation or granular surface and the solution at their interface.

This surface tension depends on the chemical composition of the grout being used and the chemistry and physical conditions of the individual soils being treated. The affinity of the grout, which results from its wetability behavior in contact with the formation into or thorough which it is being driven, is thus of critical importance. This affinity, which includes the grout's surface tension properties, is fundamental to its penetrability. A simple example of the concept is the relative immobility of a drop of water placed on a newly waxed surface, compared with the rapid dispersion of a drop of the same water placed on a similar but heavily oxidized surface.

An excellent example of this occurred during an experimental test program to develop the optimal grout for injection into several thousand feet of ⅜ inch OD tubing embedded within the embankment of a earth dam. The tubing had been installed in the embankment during the original construction, as part of an extensive instrumentation system. The test program was to include pumping of grout through several thousand feet of tubing, to determine injection parameters that would be required for the actual work. Although the original tubing was made of saran plastic, such material was no longer available, so a PVC plastic tubing of identical dimension was used.

As part of the evaluation, an eight foot length of tubing was secured to a slanted board, as shown in Figure 6B.21. Various trial grouts were then run through the tubing, with the flow times being noted. Because of a concern that the different tubing materials might react differently to the grout, an eight foot long piece of the original saran tubing was removed from a gallery in the dam. It was secured to the slant board adjacent to the PVC tube, and fed with grout from a common funnel, as illustrated in Figure 6B.22. Interestingly, the grout flowed more easily in the PVC tubing, as can be

FIGURE 6B.21 Tubing attached to slant board for grout flow evaluation.

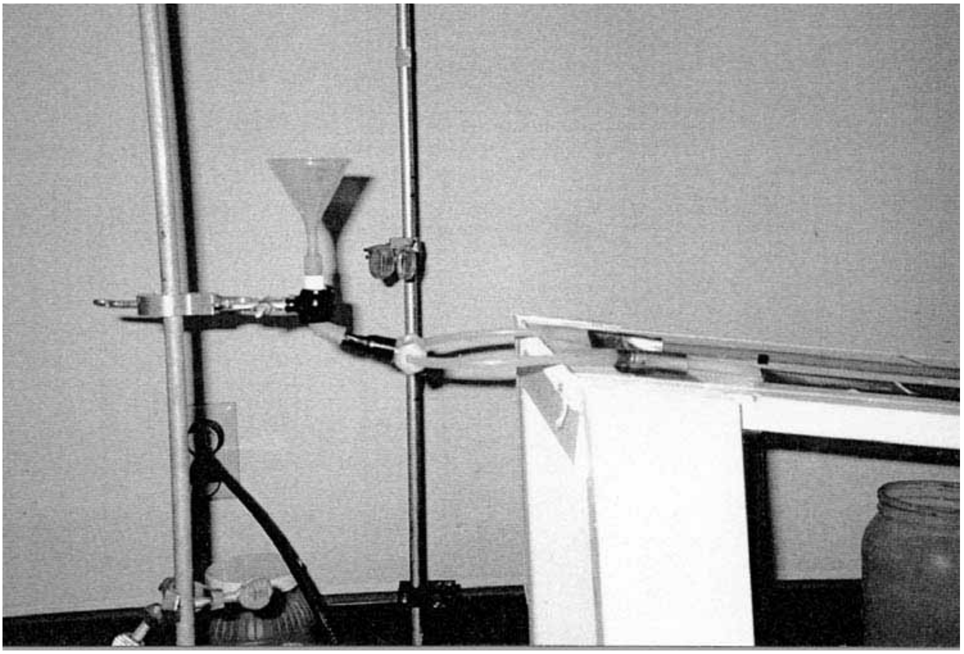

FIGURE 6B.22 Both tubes are simultaneously filled through a common funnel.

seen in Figure 6B.23, wherein the graduated cylinder on the right is filled much higher than that of the left, below the saran tubing.

The most promising grout mixes were then pumped through several thousand feet of the PVC tubing, Figure 6B.24. As a result of the slant board tests, a correction factor was developed for the saran tubing, allowing an accurate calculation of the grout pressure that would be required to inject the saran tubing existing in the dam. As confirmed by these tests, the penetrability of a grout is not solely dependent upon its viscosity. Low-viscosity grouts can have poor penetrability, whereas highly viscous grouts can be very penetrable, in a given delivery system or formation.

In the case of grouts that contain solids, such as fluid suspensions and slurries, the grain size of the solids also affects the grout's ability to penetrate. Theoretical research and discussion abound, relative to the ratio of maximum grain size that can penetrate a given crack and/or joint width in the grouting of rock. Although the subject is somewhat contentious, in rock grouting there is general agreement that the minimum dimension of the defect must be at least three to five times that of the maximum grain size in the grout. Although the maximum grain size unquestionably affects the penetrability of an obvious defect, the author suggests that it is only one consideration, and perhaps not the most important.

As an example, should a grout contain only a small percentage of the maximum size particles, and those particles were well distributed so they were not adjacent to or in near proximity of each other, the maximum size would not be of great significance, as long as their dimension were less than that of the void being filled. Conversely, if the grout contained a larger proportion of the large-size particles and/or they were in near proximity of each other, the risk of blockage would be much greater, and thus the maximum size of the particles more significant. Regardless of the grain size, of particular importance in all cases is the amount of shear energy applied during the grout mixing. It

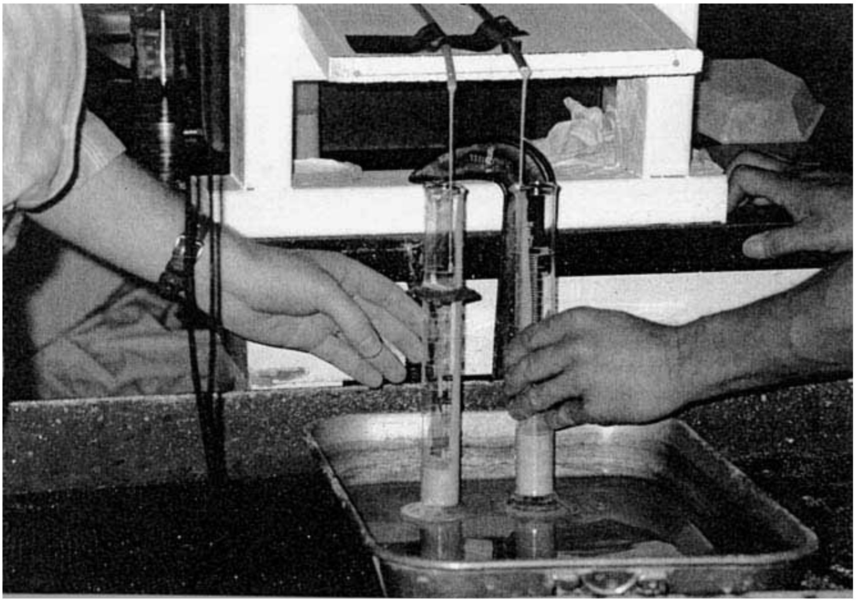

FIGURE 6B.23 Cylinder on right is filling faster than that on the left.

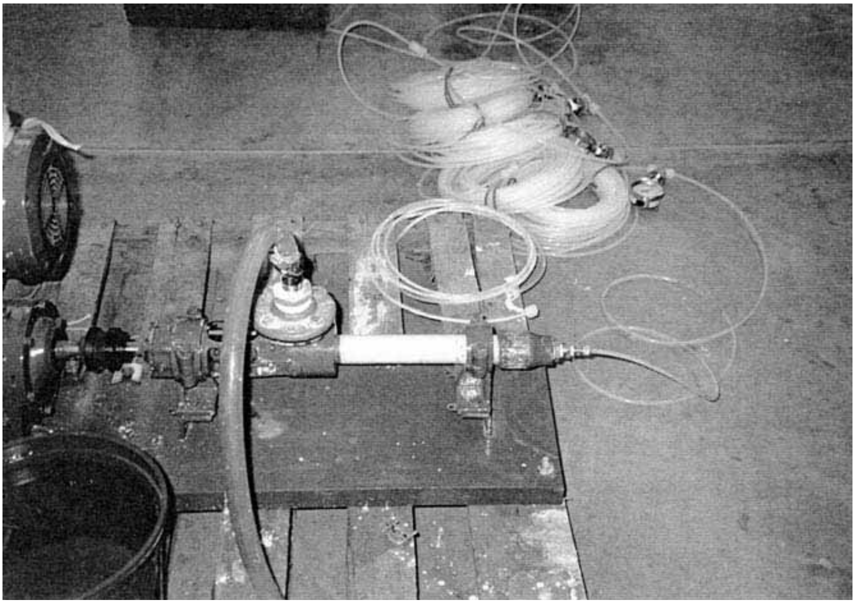

FIGURE 6B.24 Test grouts were pumped through several thousand feet of coiled tubing to simulate conditions actually existing in the dam.

has an important influence on the dispersion of the individual grains, and thus the penetrability of the resulting grout.

The shape of the particles and condition of their surface are also of immense importance. Rough, angular particles are far more likely to form blockages within a pore space or defect than are smooth round ones. Experience has repeatedly shown that cementitious grouts containing large amounts of pozzolanic materials, such as fly ash or silica fume, are more penetrable than those containing only cement. This is no doubt the result of the near spherical shape of most pozzolans. Round shaped grains tend to roll past and over each other, rather than bind together as would rough angular shapes.

Although the terms mobility and penetrability are perhaps the most all-inclusive for describing the rheological properties of a grout, there are other factors that must also be considered to assure the proper selection and applicability of a grout mix.

6B.4.3 Cohesion

Lombardi (1985) reported on a simple field test to determine the "cohesion" of a grout. What Lombardi is actually evaluating is the Bingham Yield Value for the grout, albeit with a simple test. It involves dipping a 10 cm square, slightly roughened metal plate approximately of ⅛th inch thick (Figure 6B.25) into the grout and measuring the weight of the grout that remains on the plate after removal.

6B.4.4 Bleed

When at rest, the individual particles of a fluid suspension grout will tend to settle out of the solution. This is referred to as bleed. To prevent bleeding of the grout prior to injection, it is usual-

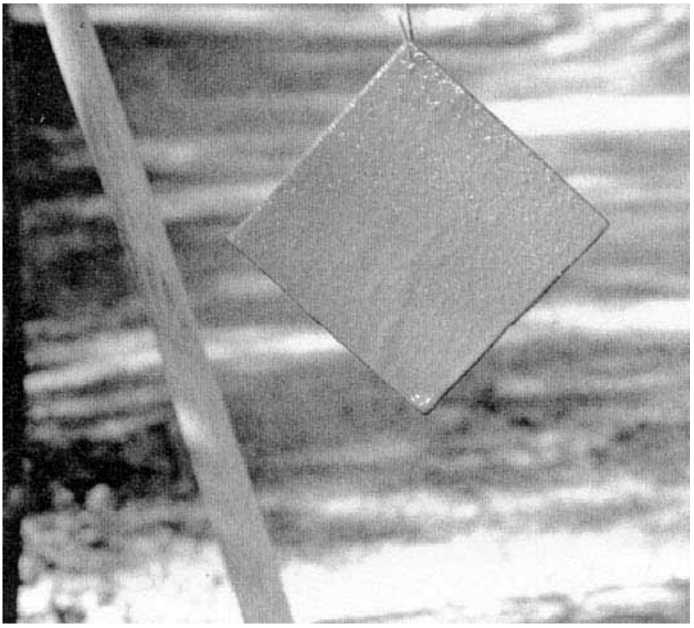

FIGURE 6B.25 Lombardi cohesion test.

ly continuously agitated after mixing. Whereas excessive bleed will not occur under proper agitation, it can occur when the grout is essentially at rest, either during or after injection. This is of particular significance where large quantities of grout have been injected into a single location or void, or where the injection rate is very slow. As bleed occurs, the solids will settle to the bottom of the void or defect being filled, and the top portion will be filled initially with water, and eventually with air, leaving void space. The bleed potential of a grout can be greatly reduced by very through, high-shear mixing, as well as through thoughtful mix design, including the judicious use of admixtures.

6B.4.5 Setting Time

In many cases, control of the time required for a grout to set or harden can be crucial to proper performance. It is imperative that grout injected into a given hole has set or become immobile prior to the drilling of adjacent holes. Rapid setting times are usually desirable when injection is into moving water, so the grout mass will set before it is washed away. Even where the water is not moving, a rapid setting will minimize the opportunity for dilution of the grout. In many cases of soil strengthening or groutjacking, rapid strength development is required in order to limit downtime of a facility. Conversely, where injection is to be made through a very long delivery system, or into large linear void spaces, extension of the setting time might be required, so as to prevent hardening of the grout within the delivery system or the initially filled portions of the void. Admixtures are commercially available that can either accelerate or delay the setting time of all cementitious grouts. In the case of chemical solution grouts, the setting time can often be controlled through judicious proportioning of the mix ingredients, or the use of set controlling components.

6B.4.6 Solubility

When mixing a solution grout, rapid dilution of all the components is beneficial and shortens both the amount of time and energy required for mixing. If the grout is to be placed into a saturated formation or void, however, the ability of the mixed grout to easily dissolve is undesirable. Under such placement conditions, and especially where the water is moving, a grout of low solubility in water is desirable. It is also important to assure that the particular grout to be used will not be adversely effected by the chemistry of the formation or ground water into which it is to be injected. An example would be attack of the grout curtain under a tailings dam by retained acidic water.

Properties of grout in the hardened state are also often very important. Is the purpose of the grouting short term, such as solidification of sandy soils to permit tunneling or excavation, or must the improvement be permanent? If strengthening of the soil is desired, the grout or the grouted mass must be of sufficient strength and/or stiffness. In many applications, the dimensional stability or shrinkage of the grout must be considered. Nearly all chemical solution grouts shrink when allowed to dry, as do most cementitious compositions.

Durability of the hardened grout, under the conditions that exist at the injection location, should also be evaluated. In this regard, chemistry of both the soil and the ground water, to which the grout might be exposed, should be appraised, as well as any unusual chemical exposures that might occur during the lifetime of the installation. Exposure to unusual temperature variations must also be considered, as the setting time of virtually all grouts is temperature dependent, as is the durability of many. Both the strength and durability of chemically grouted masses are dependent upon their moisture state. The ability to resist syneresis (the exudation of liquid from the grout) or extrusion of the grout from its intended location is also of importance, especially when the purpose of the grouting is the blockage of high heads of retained water.

Most reacted water-control grouts will shrink upon drying. Some will swell upon being rewetted; however, there is often a substantial time lag for complete expansion, and often times it will not be to the full volume that existed prior to drying. Although this is of little concern where the grout will always be in contact with water, it is an important consideration in arid regions or other locations where drying might occur.

6B.5 TESTS FOR EVALUATION OF GROUTS

Simple field test methods to assure the quality and propriety of the grout are unquestionably one of the greatest needs in geotechnical grouting. Unfortunately, few simple tests exist for either very thin or very thick grouts. A number of tests do exist, however, for mid-range grout consistencies, which are generally fluid suspensions.

6B.5.1 Flow Cones

Several different configurations of funnel devices have been proposed or used to evaluate the flow properties of grout, two of which have become somewhat commonly used. The "flow cone," Figure 6B.26, originally developed in the early 1940s, has been used for many years by the U.S. Army Corps of Engineers under the designation CRD-C611, for both research and field quality assurance. In 1981, it was adopted by ASTM as part of their C939 "Standard Test Method for Flow of Grout for Preplaced Concrete Aggregate." The Marsh funnel, Figure 6B.27, long used to evaluate the flow properties of drilling fluids, is now also used for pourable grouts.

Flow evaluations are made by filling the funnels while holding a finger over the outlet. The finger is then released, allowing the funnel to empty. The time is noted to the nearest second, providing the number of seconds required for the cone to completely empty. This is known as the efflux time. The C939 cone is typically made of cast aluminum, whereas the Marsh funnel is usually made of

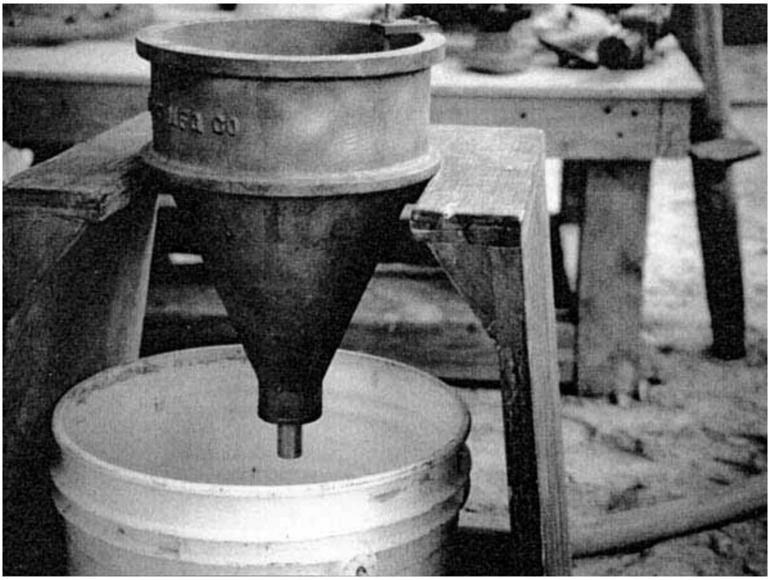

FIGURE 6B.26 ASTM C939 Flow Cone.

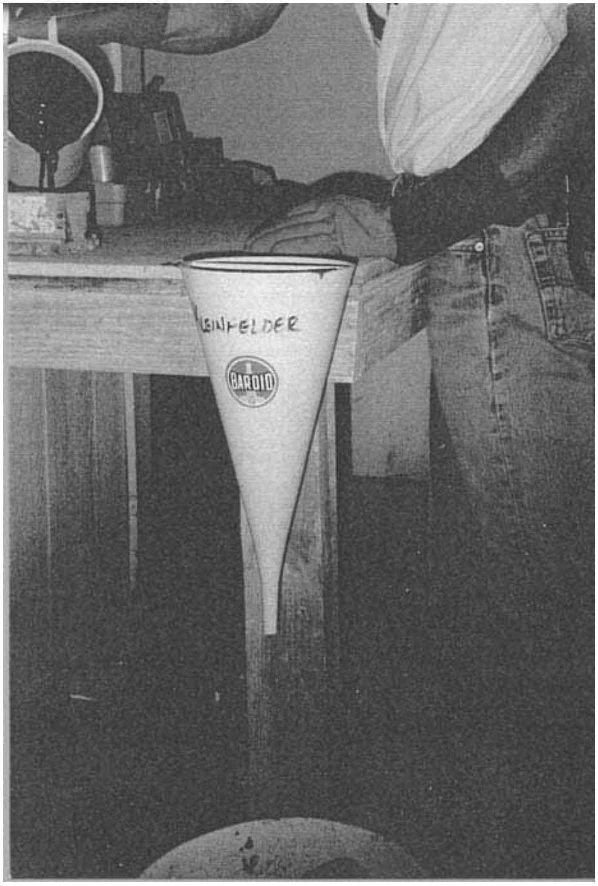

FIGURE 6B.27 Marsh funnel.

plastic. Neither material can have wetability properties at all similar to the wide variety of geomaterials into which a grout might be injected. They do, however, provide for easy confirmation of the uniformity of different batches of grout and are widely used in quality assurance testing.

6B.5.2 Specific Gravity

Another testing device that was originally developed for the evaluation of drilling fluids is the Baroid Mud Balance, Figure 6B.28. It is a simple and very rugged device for determining the density of drilling mud or grout. In application, the cup and beam are removed from the fulcrum, and the cup is dipped into the grout until it is completely filled. Any excess grout that has come in contact with the beam or outside of the cup is removed. The beam and cup are then placed on the fulcrum, and the weight is slid along the beam until the bubble of an attached spirit level is centered. The density is then read directly from a scale on the beam. The mud balance is a particularly useful device for quality assurance testing of cement–water suspensions, in that the water to cement ratio can be easily and accurately determined.

FIGURE 6B.28 Baroid mud balance used for evaluation of grout density.

6B.5.3 Evaluation of Bleed

Controlling the bleed of suspension grouts is also important, especially when slow pumping rates are used or large voids are being filled. The bleed potential of a suspension grout can change quite markedly with changes of water to cement ratio, shear mixing energy, or changes in properties of the cement or other grout constituents. Bleed evaluation is easily accomplished by filling a transparent tube or jar with the grout and observing the amount of clear water that collects on the top after about two hours. Where accurate determinations are desired, 1000 ml graduated cylinders, as shown in Figure 6B.29, can be used. These can be used in a manner similar to that provided in ASTM C 940, "Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory." Whereas this procedure is useful for laboratory evaluations, the required settlement time often renders it impractical for the control of injection parameters in the field.

6B.5.4 Pressure Filtration

Pressure filtration is essentially bleed forced by the pumping pressure imposed upon the grout. It can be evaluated for common cement grouts with a standard Gelman pressure filter. The 47 mm diameter test chamber shown in Figure 6B.30, fitted with a disposable glass fiber filter, works well on

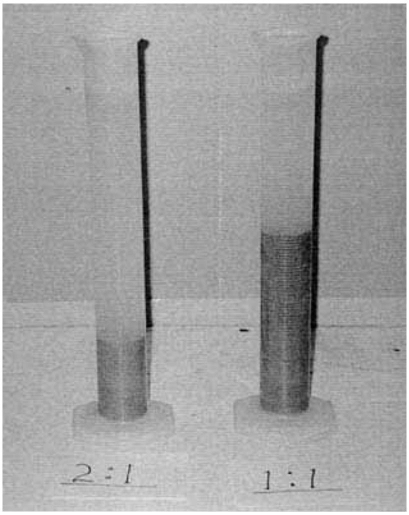

FIGURE 6B.29 Evaluation of the bleed of two cementitious water suspension grouts with different water to cement ratios.

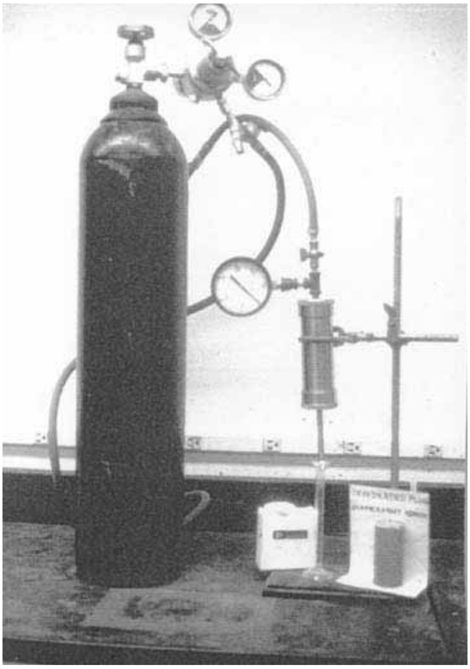

FIGURE 6B.30 Pressure filtration evaluation with Gelman filter apparatus.

grouts based on common cements. Various applied pressures on the grout can easily be imposed by pressure regulation of bottled nitrogen gas, which is typically used.

6B.5.5 Slump

As previously discussed, standard slump tests are not appropriate for either very wet or very dry mixtures, and certainly not for the rather sticky mixtures that are common in grouting. Except for the rather rare instances of *fill grouting*, in which standard concrete or mortar is being injected, the slump test should be neither specified nor used. Where it is mentioned, however, the particular standard should be provided. There are several different established slump tests that involve molds of different sizes and configurations than the most common ASTM C 143 test, shown in Figure 6B.20.

6B.5.6 Strength Tests

In most cases of soil grouting, grout strength is of little concern as long as it is at least that of the soil. In fact, a mistake frequently made is specification of cementitious grout strength, based on typical concrete or mortar standards. Specifying unnecessarily high strength for the grout is in most cases of no benefit, can present difficulties for the contractor, and usually results in increased costs. In those cases where strength is of importance, either standard unconfined or triaxial compression tests can be made. Specimens for grout are usually standard 2 by 4 inch cylinders or two inch cubes.

In the case of permeation grouted masses, it is not the strength of the grout that is of importance, but rather that of the resulting solidified soil mass. In such cases, any laboratory tests, to be meaningful, must involve saturation of the proposed grout into a confined mass of the particular soil. ASTM Standard D 4220, "Test Method for Laboratory Preparation of Chemically Grouted Soil Specimens for Obtaining Design Strength Parameters," is applicable for the preparation of such specimens. Because virtually all laboratory tests rely on remolded specimens, the indicated strengths may not compare closely to that of the in-place grouted soils, however the indicated strength should give a good idea of the strength levels that can be reasonably expected.

In order to verify field strength performance, specimens should be carved from the actual grouted mass for evaluation. This will require excavating to the grouted zone for access. Core drills can be used to obtain specimens of many masses of cement grouted soil and those soils that have been solidified with higher strength chemical solution grout, provided the soil zone where the core is taken has been completely permeated and essentially all of the pore space filled. Grouted soils are usually of much lower strength than grouted rock, and some erosion and disturbance of the core surface often results from the drilling operations. It is sometimes useful to procure a core of greater diameter than is required for the test and then carve it down to the required size. Be aware, however, that very often it is not possible to obtain a proper core specimen of grouted soil. This is especially so where weaker grouts are used or the soil pore space is not completely filled. The strength of many grouts and virtually all chemical solution grouted masses are significantly affected by their moisture condition. It is thus crucial to preserve the moisture condition of the specimen as obtained, and often precludes the use of core drilling for specimens that are above the water table.

Because chemically grouted masses are subject to significant creep, and the fundamental strength is often significantly less than that indicated by the rather rapid loading of the ASTM D 4219 test, which was previously discussed, variable strain, constant stress compression tests, such as shown in Figure 6B.31, are preferable. The simple device illustrated operates on compressed air, and the imposed load level is easily controlled by varying the air pressure with a regulator, such as shown on the right, bottom, of the figure. Such simple equipment can easily be taken to the job site and tests run immediately upon retrieval of the specimens.

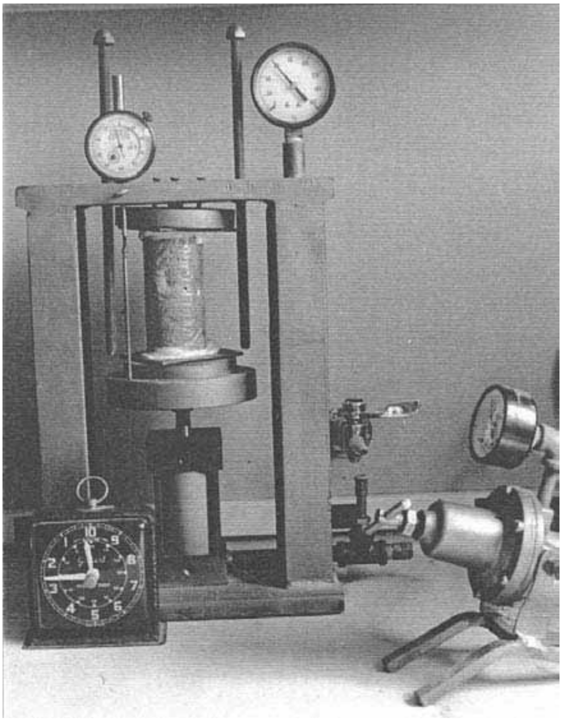

FIGURE 6B.31 Constant stress testing apparatus with test in progress.

6B.6 TYPES OF GROUTING

Classification of the many different types of soil grouting is a difficult task at best. This is due to the wide variety of different grout materials and end uses, as well as the many different injection methods and procedures. Additionally, much confusion results from the promotion of a variety of "proprietary" terminology by grouters, in an attempt to make their activities appear to be something special. Perhaps the best classification for different methods of soil grouting was presented by Xanthakos, Abramson, and Bruce (1994). Therein, four different categories of soil grouting are identified:

1. Hydrofracture (or claquage)
2. Compaction
3. Permeation
4. Jet (or replacement)

6B.6.1 Fracture/Claquage Grouting

Fracture grouting involves the intentional hydrofracture of the soil by a fluid suspension or slurry grout, with the intent of producing a network of interconnected grout-filled lenses to act as reinforcement. The procedure was developed in France (origin of the term claquage), in an effort to overcome the limited ability of most soils to accept the particles common in suspension grouts due to insufficient pore size. In theory, the procedure should be quite effective; however, in practice, control of the direction and extent of the fracture system is nearly impossible. Due to this limitation, the procedure has not been extensively used outside Europe, although some notable exceptions exist. To create the hydrofractures, high pressures are generally required. In the experience of the writer, pressures of up to 1200 psi (1.7 MPa) have been used, although pressures as high as 2760 psi (4 MPa) have been reported.

Because uncontrolled hydrofracture can be very damaging to both the soil and adjacent structures, it is crucial to maintain control of the grout deposition to the greatest degree possible. This requires a strict limitation on the quantity of grout injected at any one location (not more than about two cubic feet), necessitating relatively close spacing of the grout holes and injection intervals. In fracture grouting, *sleeve port pipes* (tubes-à-manchette) are used; they are simply tubes with drilled ports, spaced at regular intervals, typically within a range of one or two feet, as shown in figure 6B.32. The tubes are typically made of 1¼ to 1½ inch plastic pipe. The regularly spaced ports are covered on the outside with a thin rubber sleeve so as to prevent any soil or encasement grout from entering the tube. They are placed into oversized drilled holes. A weak grout, which is typically of cement–bentonite composition, is used to fill the annular space between the tube and the oversized hole.

For grout injection, a double packer is used to isolate any one of the ports for injection, which can be used in any desired order. The injection pressure breaks the rubber sleeve and the containing grout seal, such that the grout exits at the desired vertical interval. The writer has investigated many

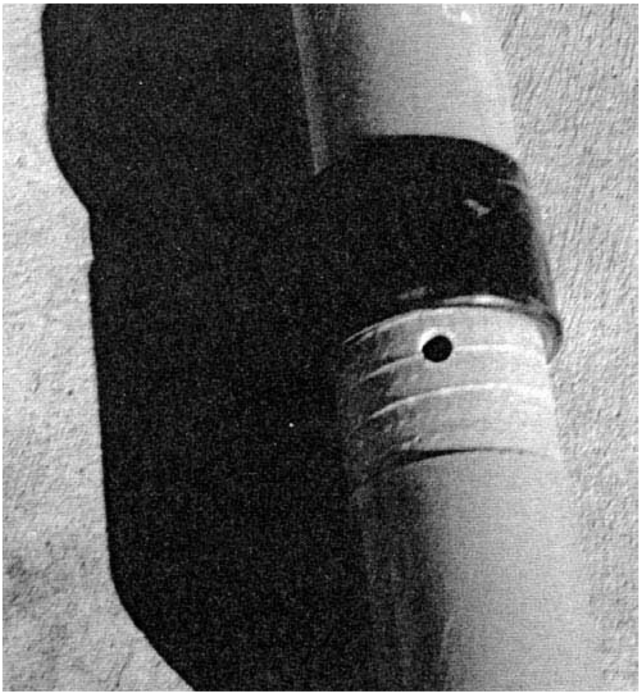

FIGURE 6B.32 Portion of a sleeve port pipe. Rubber sleeve has been folded back to expose one of the drill hole ports.

instances of damage resulting from uncontrolled hydrofracture grouting, and is of the strong opinion that where performed, strict limitations of grout quantities, combined with meticulous observation and control of the grout pressure and other injection parameters, are essential. Fracture grouting can be performed in virtually any soil and to any depth.

There are other grouting methods, which are usually both faster and more economical than closely controlled fracture grouting, for the strengthening of most soils. A clear exception where fracture grouting is applicable however, involves the pressure injection of hydrated lime slurry to stabilize the moisture in, and reduce the expansive potential of, clay soil. Because the chemical reaction requires contact of the lime with the soil, closely spaced fractures are needed. This type of injection is usually done on a fairly large-scale basis by contractors who specialize in it. The equipment tends to be massive, possessing several probes with perforated pointed tips, which are pushed into the soil simultaneously, with either continuous or discrete injections of the lime slurry (Figures 6B.33 and 6B.34). As can be observed, the process can be quite messy, and avoiding lime deposition on either the equipment or surrounding surfaces is nearly impossible. Although the slurry has a distinctive white color, and is quite alkaline, with a pH of about 12, it presents no serious health or environmental risks, and is fairly rapidly diluted by rainfall or washing, and then absorbed into the surface soil. The high alkalinity, however, can cause burns on human flesh if allowed to remain in contact for an extended time. Spills should therefore be promptly washed off the skin.

The depth of treatment depends upon the expansive potential of the subject soil. It is usually on the order of 5 to 20 feet, but applications as great as 40 feet, have been reported. The slurry consists

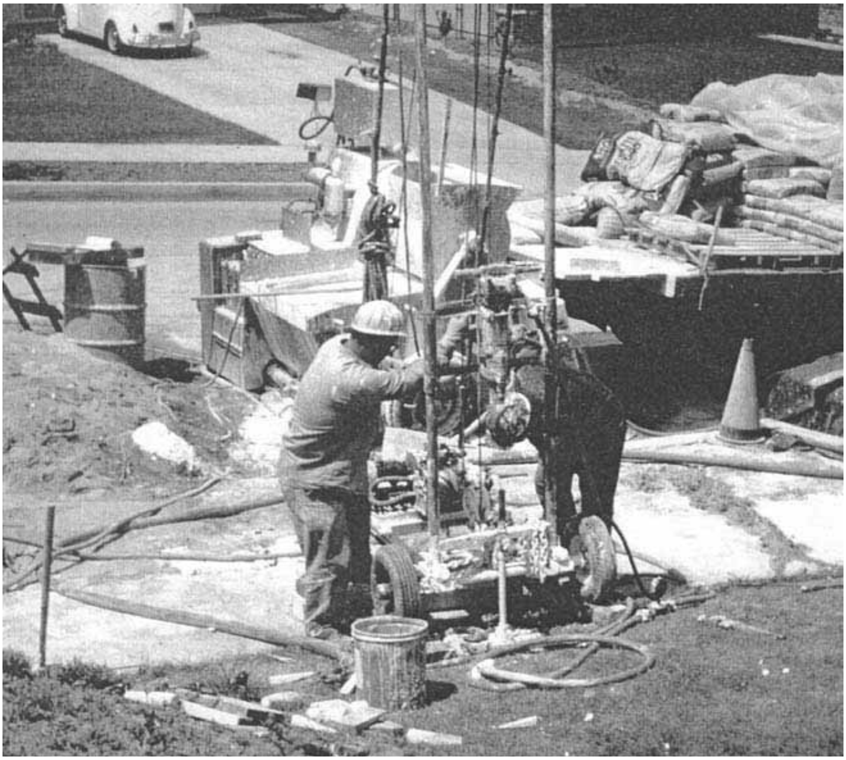

FIGURE 6B.33 Small-scale lime injection

FIGURE 6B.34 Large-scale lime injection.

simply of water mixed with about 25 to 30% by weight of dry hydrated lime. A surfactant wetting agent is often included to improve penetrability of the slurry. It is typically injected at pressures within a range of 50 to 200 psi. Injection is continued until either refusal at the maximum pressure or surface leakage occurs, or the predetermined amount of slurry, which is typically about 10 gallons per vertical foot of hole, has been placed. Because the injection slurry consists primarily of water, which can be absorbed into the clay, some accelerated expansion can take place, immediately after injection, unless the initial soils moisture content was near its maximum level. This expansion tends to stabilize shortly after the injection, however, usually within a period of two or three weeks.

Lime injection has been subject to considerable promotion by those who perform the work as well as the National Lime Association, a trade organization for manufacturers of commercial quicklime and hydrated lime. The association has made available a publication by Boynton and Blacklock (1986) that includes a comprehensive bibliography of 51 references.

In 1967, the writer was involved in the remediation of damage to the slab on grade floor of three large dormitories at the Hollygrove Orphanage, in Hollywood, California. An investigation disclosed that the floor slab had experienced upward movement of as much as 1.5 inches. An additional heave of 2.75 inches was predicted. The cause of the upheaval was determined to be a layer of highly expansive clay, underlying the floor at a depth of two to seven feet. Because funding was restricted and moving the occupants out of the structure was not possible, lime injection was adopted as the most reasonable remedial method, although it was considered somewhat experimental at the time.

As previously mentioned, lime injection is inherently messy, so great care was taken to perform the work with as little damage to the structure as possible. Injection holes were located on a three foot grid, normal to the module of the existing 12 inch square floor covering tiles. The work was accomplished with only minor disturbance and the structures were occupied throughout the work. Limitation of floor covering replacement to only those tiles at the hole locations, and waxing of the entire floor upon completion of the work, resulted in a large cost savings to the owner. The replacement tiles were of bright colors to provide an accent, which actually improved the esthetics of the original construction.

The slurry consisted of high-calcium hydrated lime mixed with water, at the rate of about 4.2 pounds of lime per gallon of water, to make a nearly 50% mixture by weight. The injection rate was held to a relatively low value of about 0.6 cubic feet per minute, in order to minimize leakage. Injection pressure varied between 50 and 250 psi. Reading from a report prepared after completion of the work:

> The slurry was injected from the top down through special injection needles made especially for this work. The needle was driven downward from the surface until the clay layer was "felt," after which a metered quantity of 0.25 cubic feet (7 L) of slurry or 7.5 pounds (3.4 kg) of lime was injected. The needle was then continued downward in 6 inch increments, the metered amount of slurry injected at each, until the bottom of the clay layer was "felt" or minus 7 feet was reached. In some cases it was not possible to inject the full measure of slurry due to uplift of the structure walls or floor.

Several elevation surveys had been taken prior to the work. Others were taken immediately following completion and at 30, 90, 180, and 360 days thereafter. Immediately following the injection, an additional upward movement of about 0.25 inch (6 mm) was recorded. No further movement was noted thereafter, and, in fact, after one year a portion of the original 0.25 inch (6 mm) of heave was recovered.

6B.6.2 Compaction Grouting

Compaction grouting involves injection of very stiff, mortar-like, low-mobility grout at high pressure into discrete zones of soil. Because of the low mobility of the grout, much higher pressures are used than in traditional grout injection. It is not unusual to experience pressures of several hundred psi when injection is made at depths of only five or ten feet. Properly placed, the grout remains in a homogeneous, expanding mass, which displaces and thus compacts the adjacent soil, increasing its density. Fundamental to the success of the procedure is deposition of the grout in such a manner that it remains in a globular mass at the injection location, with a distinct grout–soil interface. In deed, one of the primary advantages of the technique is that absolute control of the grout deposition location is possible.

The work is virtually always done in stages; that is, only a few feet of the grout hole are injected at any one time. The staging can be from the top down (downstage) or from the bottom up (upstage). The upstage method is the fastest and most economical, and thus, the most frequently used, especially for deep injection. For shallow injection (less than about 15 feet), working downstage has the distinct advantage, in that each injected stage provides additional restraint and containment for those that follow. Thus, higher pressures, which enable a greater quantity of grout to be injected, and thus greater densification, can be used in the deeper stages, after the overlying soils have been strengthened. Whereas upstage injection is nearly always accomplished in one continuous operation, when working downstage, each stage is allowed to harden before the next one is drilled and grouted.

Grouting upstage involves:

1. Drilling a hole to the bottom of the zone to the grouted.
2. Placing casing to within a few feet of the bottom of the hole. The casing should be a snug fit and may require pushing or driving into place. Sometimes it is driven entirely, the predrilling being eliminated.
3. Injection of the grout is continued until essential "refusal" is reached. Refusal is usually considered as: a) a slight movement of the overlying ground surface or improvements, b) injection of a

predetermined amount of grout, or c) reaching a given maximum pressure at a given pumping rate.

4. Raise the casing a regular increment, usually one or two feet (0.3 or 0.6 m)
5. Resume grout injection until refusal.
6. Repeat steps 4 and 5 until the top of the zone to be grouted has been reached.

Grouting downstage involves:

1. Drilling an oversize (usually about three inch diameter) hole, to the top of the zone to be densified or a minimum of about four feet (1.2 m) deep.
2. Insert a casing (usually two inch internal diameter), into the hole and fill the annular space outside the casing with rapid setting grout.
3. Drill through the casing and advance the hole several feet for the first stage. Typical stage lengths are on the order of three to six feet (1 to 2 m).
4. Inject grout until refusal is reached, as described above.
5. Repeat steps 3 and 4, after the previously placed grout has hardened, until the bottom of the zone to be injected is reached.

Grout holes are usually spaced on a grid of six to twelve feet, although closer spacing is occasionally used. Alternate primary holes should be injected before the intermediate secondary holes. As a general rule, the injection should start at the outside of the area to be improved, working toward the interior portions. When grouting near a down-slope or retaining wall, the holes nearest these features should be injected first. Holes should generally be vertical, as inclined holes provide a greater horizontal effective area (Figure 6B.35). This results in refusal, due to surface heave, at lower grout pressure, and thus less injected grout and resulting compaction. Also, a vertical column of grout and compacted soil provides better support than one that is inclined. Where inclined holes are used, they should generally not be more than about 20 degrees off vertical.

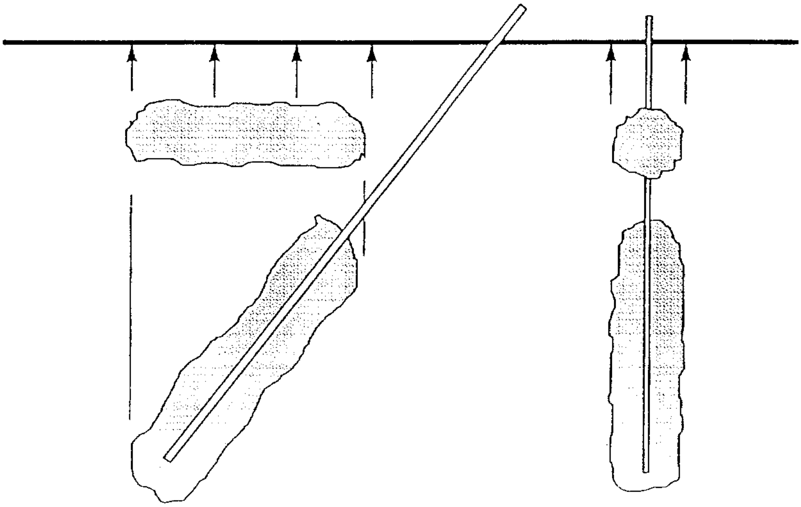

FIGURE 6B.35 Effect of grout hole inclination on surface uplift. Total uplift force is the product of pressure times *horizontal area.*

Compaction grout casing is typically two to four inch (5 to 10 cm) ID steel tubing. Some contractors use proprietary casing, while others use either standard flush wall drill casing or iron pipe. The outside surface of the casing should always be of a uniform diameter, which usually means a flush wall threaded coupling. When working upstage, the casing, and especially the couplings, must possess sufficient strength to resist the considerable extraction forces that are often required. Extraction, is most often provided by pairs of hydraulic jacks (Figure 6B.36). From the standpoint of grout effectiveness, larger casing can be used. However, in casing larger than about three inches, the pressure at the bottom of the hole resulting from the weight of the grout will usually exceed the restraint provided by line friction and may result in excessive pressure being exerted at the tip. A few contractors attempt to perform the work with casing of an internal diameter less than two inches. This is not advisable, however, at it is sometimes difficult to establish grout flow in such a small hole and the risk of hole blockage is significant. Also, if working upstage, casing smaller than 2 inch ID, often lacks sufficient strength to withstand the extraction force, and joints are more prone to breakage.

Because of the very low mobility grout that is used in compaction grouting, all valves and fittings must provide full flow openings. Appropriate gage savers must be supplied for all pressure gages and should have a minimum dial size of three inches, so they can be easily read. Standard pipe fittings should be avoided, and wide sweep bends used as required. A high-pressure, two inch internal diameter hose is the most often used, however one and one half inch hose is sometimes adequate. Where especially long runs are required, the use of rigid pipe will decrease the resulting line pressure. Appropriate quick connect couplings, such as those used for concrete pumping, should be used. Figure 6B.36, shows a typical connection to a hole during grouting.

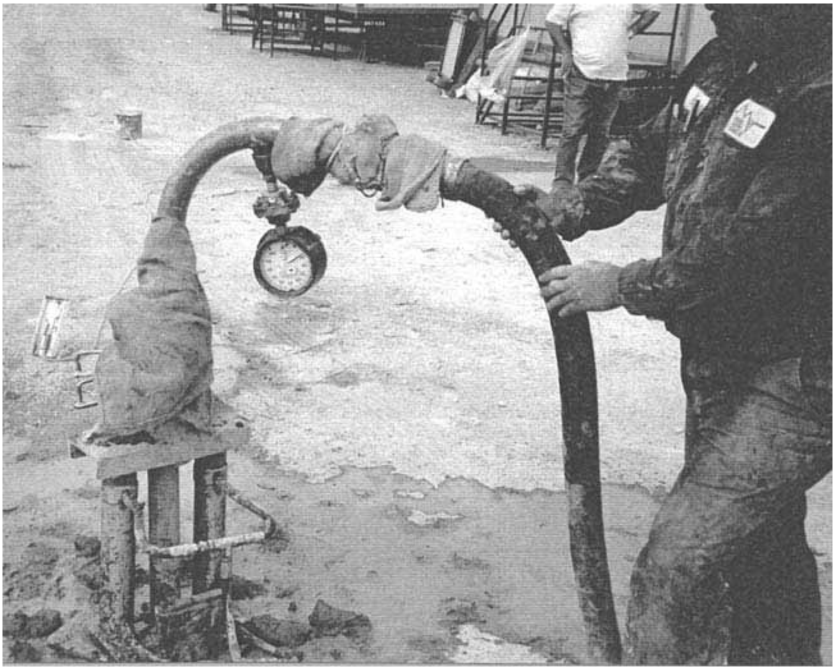

FIGURE 6B.36 Typical connection to a grout hole during upstage injection. Note pair of hydraulic jacks used for casing extraction.

Regardless of the method used or the slump or stiffness of the grout, should it be overly mobile or act as a fluid under pressure, hydraulic fracturing of the adjacent soil can occur, which will result in loss of control of the zone of compaction. Hydraulic fracturing not only interferes with orderly compaction, but can also result in unacceptable displacement or other damage. Should it occur in near proximity to a down-slope, retaining wall, or other substructure, or within a water retaining embankment, severe displacement or complete failure could result. Compaction grouting has been successfully used in most types of soil and to depths of more than 400 feet. Injection into saturated clays is dependent upon their ability to drain, however, and very slow injection rates are usually required. This increases the amount of time required to perform the work as well as the cost and often precludes its use. The procedure also has limited effectiveness in clean, coarse sands and gravels.

Compaction grouting originated in California, where the procedure has been practiced for more than forty years. Its application is now extensive throughout North America, and it is often used in many other countries, especially in Asia. The first publication on the procedure (Graf, 1969) presented a theoretical description of the process, hypothesizing that the injected masses would be generally spherical and would densify the injected soil radially, in all directions. The first report describing the actual mechanism, and providing data derived from the excavation and removal of full-scale test injections, was made by Brown and Warner in 1973. It reported columnar injected masses, with essentially horizontal, radial densification.

Subsequent to this early work, a large number of masses have been injected, exposed, and evaluated, in research demonstrations as well as in connection with actual or proposed projects. Evaluations have included determination of the increase in resulting soil density, utilizing standard penetration, cone penetrometer, and laboratory tests of split spoon specimens, both before and after grout injection. Large-scale load tests and long-term elevation monitoring of structures overlying treated soils have also been performed on many applications. Additional studies have been directed at the effect of lateral forces exerted in the soil mass and upon adjacent structures. This substantial research and investigation has resulted in compaction grouting becoming one of the best understood grouting technologies. The process is especially suited to the densification of soils, which is its only use.

The single most important parameter to assure effective soil densification is obtaining a globular grout mass, which will typically be either columnar or pear shaped. Cracking or hydraulic fracturing of the soil mass, with resulting thin lenses of grout, should generally be avoided. One measure of regularity of the obtained grout mass is the Travel Index (TI) of the grout. The TI is the maximum radial travel of the grout from the point of its injection, divided by the minimum radial distance to a grout–soil interface, and is thus representative of the grout's propensity to remain in a controlled mass and at the intended location. Grouts with a low travel index (less than about three) remain in relatively symmetrical masses, with clear interfaces of the surrounding soil. Loss of placement control and hydrofracture of the soil, however, is virtually assured with grout travel indexes exceeding about five.

Importance of the shape of the injected grout mass was first reported by Brown and Warner (1973). They described a research program performed in the 1950s that involved the injection and subsequent excavation of more than 100 test grout masses. The effort involved a variety of different mix designs, consistencies, sand materials, and injection rates. A photograph of two of the excavated test grout columns depicting desirable shape was provided and appears here as Figure 6B.37. It is interesting to note the much smaller diameter of the lower portion of the mass shown on the right. Whereas the upper deposits of the test site consisted of mixed soils, the grout holes extended into an underlying clean sand layer, which was not subject to the same degree of compaction as the upper mixed soils. This resulted in a much smaller mass diameter in the sand. The paper concluded with examples of many projects, successfully completed with grouts, that were found to be optimal. The grouts were reported to consist of: ". . . fine sand combined with about 12% cement and water to form a very stiff mortar like mixture." Emphasized were criteria such as "The greatest amount of grout injected, and thus greatest densification achieved, resulted from the use of very stiff mixtures," and "A slower pumping rate resulted in significantly higher grout takes."

The same authors expanded on their experience in a further report, "Planning and Performing Compaction Grouting," (Warner and Brown, 1974). Therein, the significance of the grout composi-

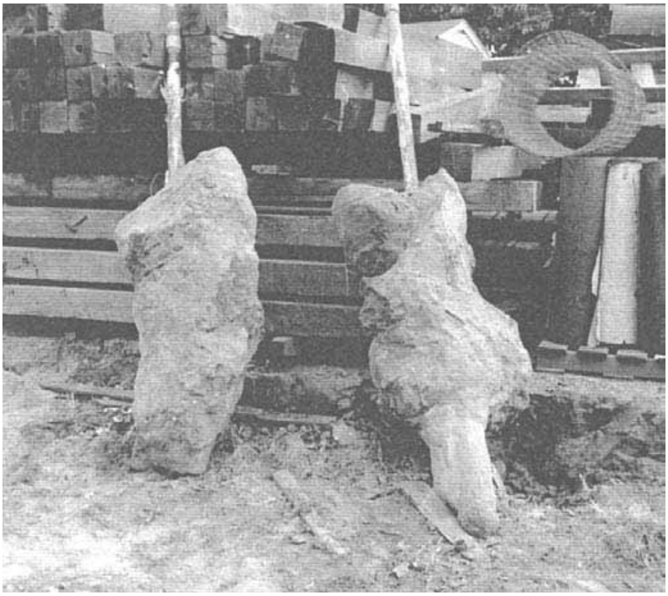

FIGURE 6B.37 Grout "columns" from two test injections (Brown and Warner, 1973).

tion, and in particular, a strict limitation of any clay content, was further emphasized. An entire section was devoted to describing the "sand material," which included a gradation envelope, "Preferred Limits of Gradation for Sand Used for Compaction Grout," given here as Figure 6B.38.

Noteworthy is the zero allowance for clay size constituents. The significance of grout consistency was again emphasized: "It is preferable to use the least amount of water that will provide a very stiff plastic consistency grout. A rule of thumb is the stiffer the grout, the more effective its injection will be."

Included was a photograph of such grout extruding from a grout hose, included here as Figure 6B.39. Further emphasized were the risks of excessive pumping rates, which could result in "rupture" of the soil. The first two of their concluding statements for proper performance stressed the importance of the aggregate gradation and resulting grout rheology, as well as the injection rate: "Proper gradation of the sand material which accounts for 80%–90% of the total grout volume is imperative" and "Absolute control of grout rate is imperative . . . within a range of 0.3 cu ft (0.009 m^3/min to 2 cu ft (0.06 m^3/min. . . ."

Quite surprising to this writer, the salient conclusions of that early research have proven to be applicable to the present day. Furthermore, that applicability has been substantiated by extensive recent research and literally thousands of successfully completed projects.

As part of the grouting demonstration included in the 12th annual short course, Fundamentals of Grouting, now sponsored by the University of Florida, and held in Denver, Colorado in 1990, ten grout test injections were made and exposed. Two different grout mixtures, designated "A" and "B," were utilized. They were identical except that 5% bentonite by weight of the sand material was included in the "B" mixes. Notwithstanding the recognized inappropriateness of the slump test for such grouts, such tests were made in as careful a manner as possible. The two grouts were injected at ASTM C143 slumps of one, two, three, and four inches (25, 51, 76, and 102 mm), and at a constant injection rate of 1.5 ft^3 (0.04 m^3) per minute.

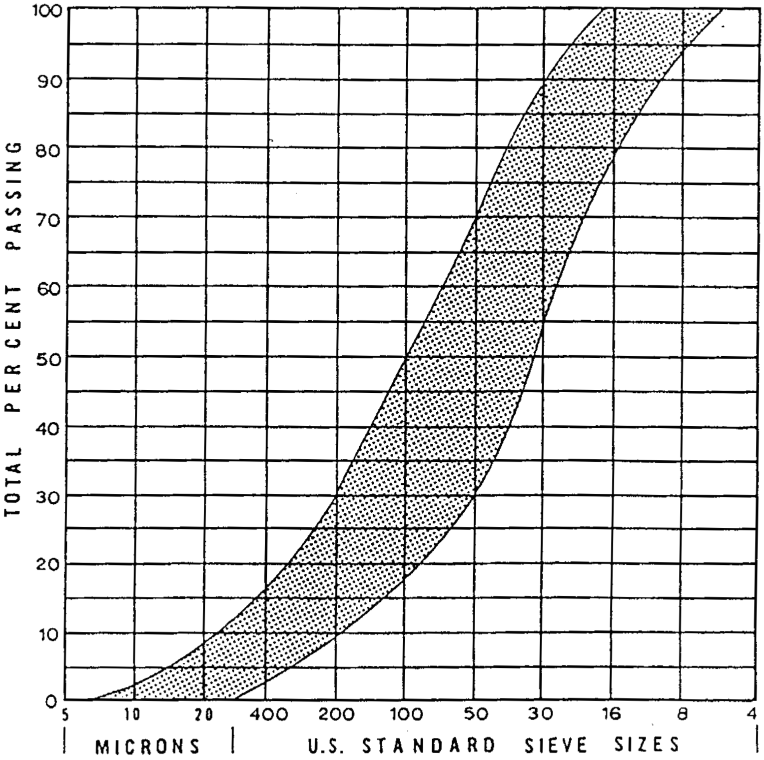

FIGURE 6B.38 Preferred limits for sand material used for compaction grouting (Warner and Brown, 1974).

FIGURE 6B.39 Very stiff plastic consistency grout (Warner and Brown, 1974).

The test site consisted of highly stratified stiff to dense clayey silts and sands, which exhibited negligible settlement potential. Untreated, they would provide adequate support for most normal foundations, and thus provided an extreme for evaluation of compaction grouting. Several of the injections resulted in hydraulic fracturing of the soil; however, the incidence and extent of the fracturing, was directly related to the grout rheology. All of the "B" mixes resulted in hydraulic fracturing and revealed very high TI's. Of particular interest was the markedly better performance of the "A" grout at a four inch slump than the "B" grout at a slump of only one inch.

An extensive and very well documented research effort was conducted in San Diego, California in 1991 (Warner et al., 1992). Eighteen grout masses were injected, with three different grout mix designs, using different grout consistencies and injection rates. The soils at the test site were predominantly fine to medium grained sands, with approximately 20% of the material passing a #200 (0.007 mm) sieve. The minus #200 (0.007 mm) portion consisted of approximately 70% silt and 30% low-plasticity clay. The soils were of waste fines from a quarry, and were deposited as an uncontrolled fill. Their in-place densities varied from 79% to 96% of maximum dry density as determined by ASTM D 1557.

The grout mixes, which were designated A, B, and C, were all identical except for the aggregate fraction, which was of different gradations. The A and B aggregate as obtained from the pit were identical. They contained about 7% clay, and the minus #40 sieve (0.04 mm) fraction was plastic, with a liquid limit of 35 and plasticity index of 10. The B grout mix contained the material as received from the pit, whereas the A mix had about 30% minus ¾ in (19 mm) gravel added With the exception of a component of about 4% nonplastic clay, the C aggregate was close to the gradation envelope recommended by Warner and Brown (1974) (Figure 6B.38). Each of the grout mixtures was injected using three different injection rates: one, two, and four cubic feet per minute (.0283, .0566, .085 and m^3/min).

Of interest was the development of three differently shaped masses of the injected grout. These were closely related to the aggregate material gradation, especially the clay content. They were radially symmetrical columnar, as shown in Figure 6B.40; four vertical "wings," extending out at about 180 degrees from a columnar mass at the hole alignment (Figure 6B.41); and two wings, with or without formation of an initial grout column, resulting in hydraulic fracturing, as illustrated in Figure 6B.42. Details pertinent to the resulting grout masses are provided in Figure 6B.43, and the travel indices are m shown in Figure 6B.44. The detrimental effect of clay in the grout was again clearly illustrated by the thin wings of grout, observed in Figure 6B.42. further evidence of hydraulic fracturing is clearly delineated by the high travel indices, enumerated in Figure 6B.44.

In 1994, the 1991 research effort of was extended with an additional eleven grout injections. These were made on the same site, immediately adjacent to the 1991 work. In order to establish the influence, if any, of the beginning soil density, the entire test site was excavated and backfilled, with minimal compaction being exerted. As in 1991, the grout mixes were identical except for the gradation of the aggregate. Three different aggregates, designated D, E, and F were used, which contained 0%, 1%, and 4.5% bentonite clay, respectively. In spite of the obvious benefit of including gravel in the grout aggregate, as demonstrated by the earlier work, because many contractors do not have the ability to pump the larger aggregate, it was not included. Exposure revealed grout masses very similar to those of the previous work. Again, a close correlation between the clay content of the aggregate and the shape of the resulting grout masses was illustrated. Figures 6B.45 and 6B.46 show typical examples of the D and F grout masses, respectively. Note the very thin, long wing length of the mass in Figure 6B.46, which resulted in hydraulic fracturing and a very high travel index of 20.

In South Africa, a mill building at a diamond mine had experienced serious differential settlement. The structure was founded on a marginally compacted mine waste fill, composed of sandy silt containing about 6% gravel. Compaction grouting was determined to be the best remedial method, but since the technique had not been previously used in the area, initial injections were excavated to assure proper performance. The grout used was a very stiff mixture of aggregate falling within the envelope of Warner and Brown (1974) mixed with about 10% cement. It was injected at a rate of about (1.5 ft^3) 0.04 m^3 per minute. As can be observed in Figure 6B.47, the resulting grout mass was in a nearly perfect column.

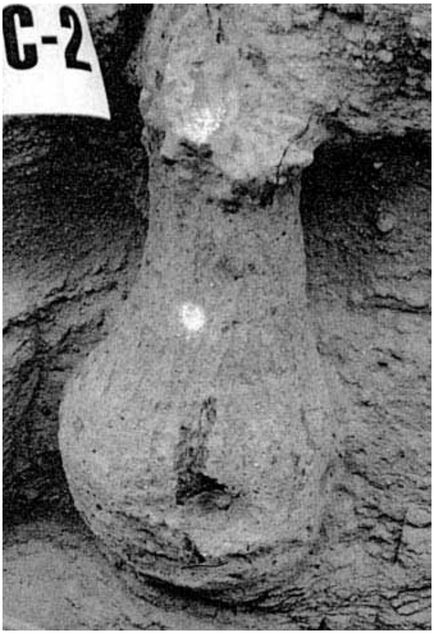

FIGURE 6B.40 Symmetrical grout mass.

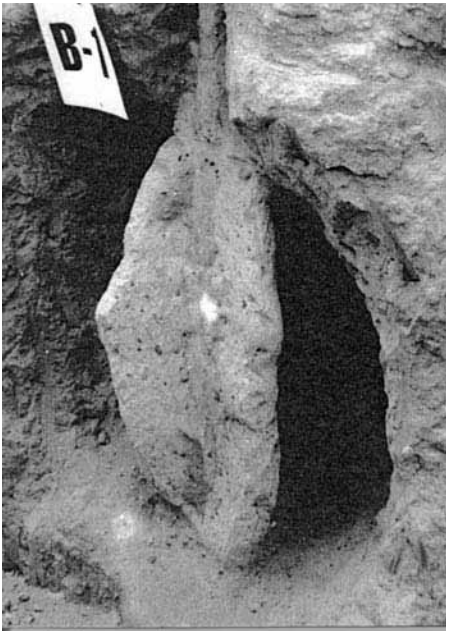

FIGURE 6B.41 Four wing grout mass.

FIGURE 6B.42 Two wing grout mass resulting in hydraulic fracturing of the soil.

Hole No.	Maximum Radial Travel		Maximum "Wings" Width		No. "Wing" Sets	Minimum Column Radius		*
	in	mm	in	mm		in	mm	
A-1	14	356	5-7	127-178	2	7.5	191	P/S
A-2	14	356	6-7	152-178	2	6	152	P
A-3	22	559	5-7	127-178	1	5	127	P/S
A-4	16	406	5-7	127-178	2	7	178	P
A-5	34	864	12	305	1	Negative		P/S
A-6	36	914	8	203	1	Negative		P/S
B-1	12	305	4	102	2	6	152	S
B-2	20	508	3-4	76-102	2	4	102	P/S
B-3	15	381	2-5	51-127	1	3.5	89	P
B-4	24	609	6	152	2	3	76	P/S
B-5	21	533	4	102	1	2	51	P/S
B-6	38	965	6	152	1	Negative		P/S
C-1	6	152	N/A		0	6	152	S
C-2	12	305	N/A		0	12	305	P
C-3	22	559	N/A		0	10	254	S
C-4	21	553	N/A		0	11	279	P
C-5	16	406	5	127	2	5	127	P/S
C-6	18	457	3	76	2	4	102	P/S

* Sequence - P = Primary; S = Secondary; P/S = One adjacent hole completed.

FIGURE 6B.43 Grout mass properties (Warner et al., 1992).

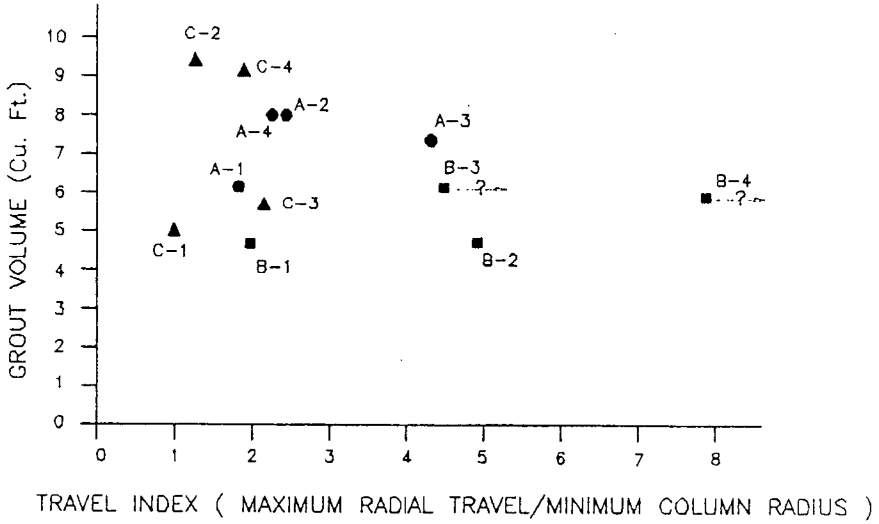

FIGURE 6B.44 Grout travel index (Warner et al., 1992).

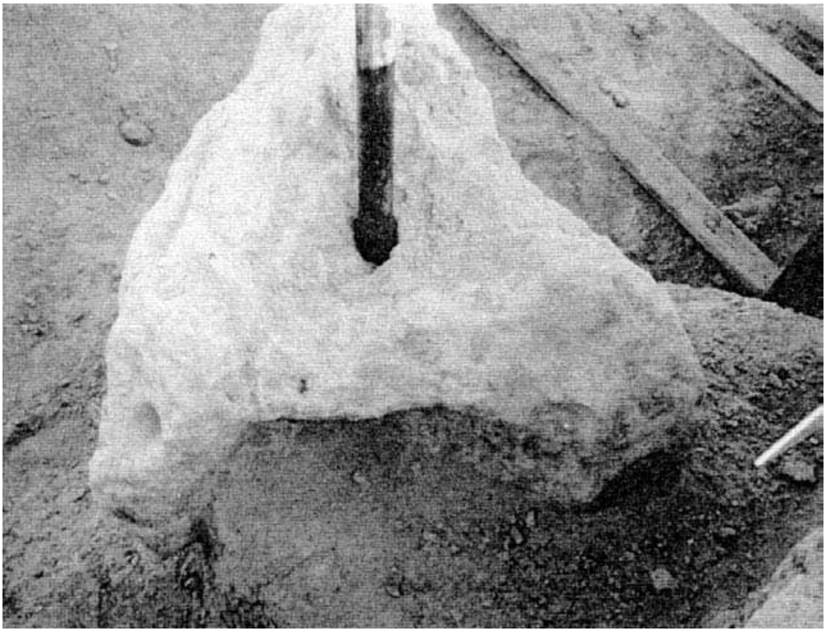

FIGURE 6B.45 "D" grout mass.

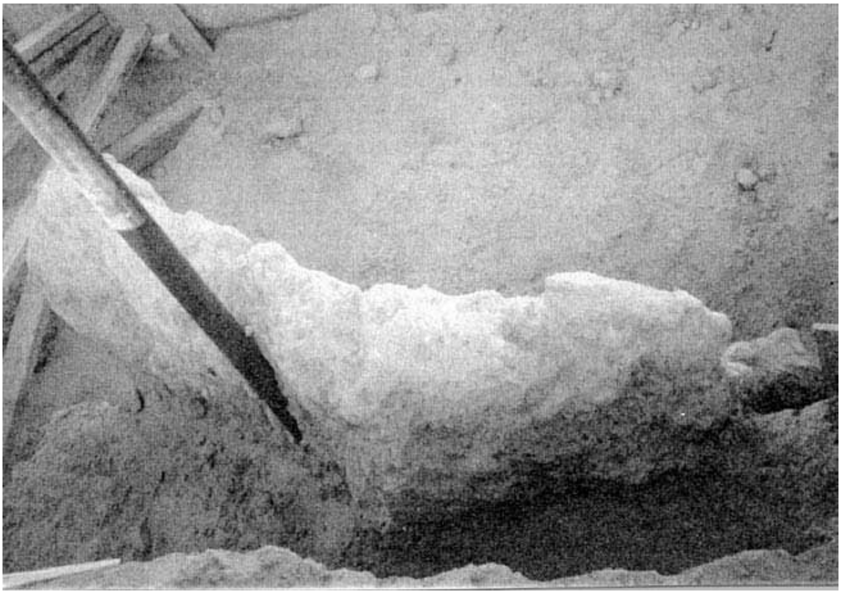

FIGURE 6B.46 "F" grout mass.

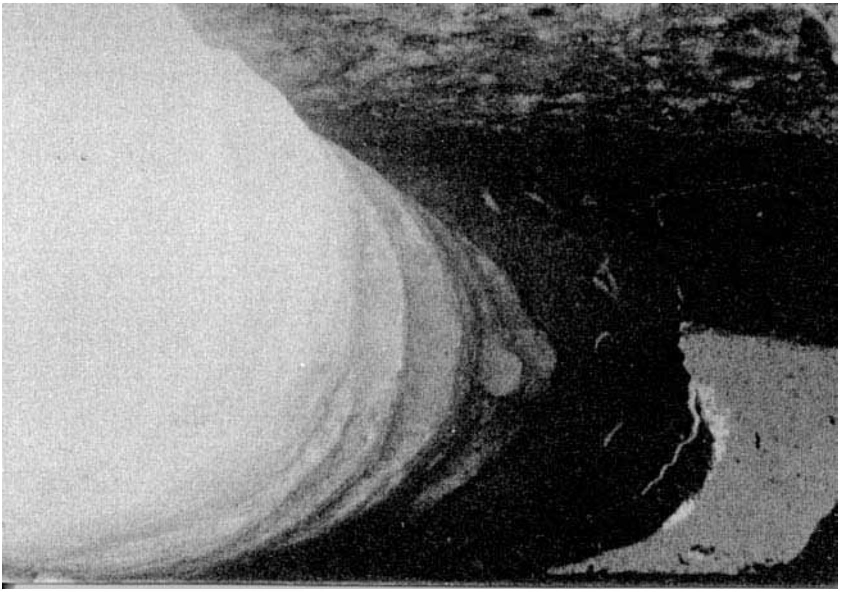

FIGURE 6B.47 Near-columnar grout mass.

Recent experimental work (1996) in Korea to develop a method to contain underwater bay mud has involved many experimental injections, both on dry land and underwater. The grout typically used is a very stiff mixture similar to that reported by Warner and Brown (1974), and is injected at rates less than 0.06 m^3 (2 ft^3) per minute. Resulting columnar masses, of nearly one meter (3.3 ft) in diameter have been routinely obtained, as illustrated in Figures 6B.48 and 6B.49. Figure 6B.48 shows an underwater grout mass that has been excavated and is being raised by a derrick barge. The grout mass shown in Figure 6B.49, was injected into loose sand. Upon exposure, the sand fell off, revealing a near perfect column.

The writer has witnessed a large number of excavations made on a variety of compaction grouting projects and visually inspected the particulars of the resulting grout masses. In the early use of the procedure, this was frequently done to increase knowledge of the technology, as well as for quality assurance. In many situations, examination of either full-scale test injections or early production work has been done as a requirement for qualification of the procedure for a particular project. In several instances, where grout injection work has failed to perform as expected, excavations allowing visual inspection of the grout masses have also been made, in order to better understand the cause of the poor performance.

In applications that are in near proximity to a down slope or retaining wall, a risk of displacing the slope or wall always exists. Damage of this type, which results from an inappropriate injection sequence, use of excessively mobile grout, or too high a pumping rate, has unfortunately been experienced on many projects within the last few decades. The initiation of such displacements will *al-*

FIGURE 6B.48 Grout mass from underwater injection.

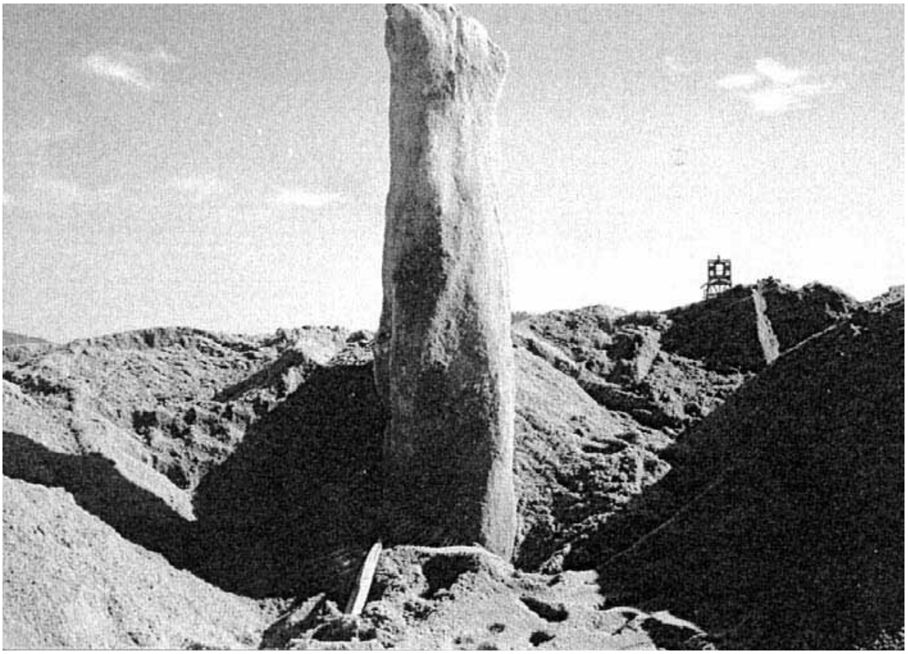

FIGURE 6B.49 Near-perfect column.

ways be indicated by a sudden loss of injection pressure. Competent grouting crews will thus promptly check for such movement at every pressure loss, especially when working in sensitive locations. If movement is detected, corrective procedures such as halting injection or reducing the pump output (which lowers the pressure) should be immediately taken, precluding the occurrence of destructive movements.

Unfortunately, not all compaction grouting projects have been satisfactory. The writer has investigated many instances where far less than acceptable performance has occurred. In virtually every instance, the poor performance was directly linked to one of four faults:

1. Failure to treat the faulty material for its full depth. Compaction grouting adds considerable weight to the treated zone. It is imperative that grout not be deposited over a soil formation that is unable to support the combined weight of the original soil and new grout. In this regard, experience has continually revealed the culprit soil zone in apparent fill failures to be in the bottom of the fill or in the original soil immediately thereunder.
2. An inappropriate injection sequencing. Soil settlement often results in lateral spreading, which causes open cracking on the ground surface or structures thereon. Initial injection should always be started at the furthest limits of the soil, which has influence on the surface spreading. As an example, settlement occurring in near proximity to a retaining wall or downslope generally has lateral movement in the direction of that feature. Initial grout injection should thus be in rows of holes nearest to the wall or slope. It is usually possible to push the soil laterally so as to close such cracks.
3. Excessively mobile grout. As previously discussed in detail, grout that is excessively mobile, or acts as a fluid in the ground under pressure, will cause hydraulic fracturing. Such a fracture will always be parallel to and near an area of weakened restraint, which is most often a retaining wall or downslope.

4. Too rapid a pumping rate. Soil deformation (compaction) is time dependent. Forcing it to occur too rapidly with grout under pressure will cause disruption and possible hydraulic fracturing. This will also be in the direction of least restraint.

Use of grout aggregates containing clay, or the deliberate addition of clay, to improve pumpability has been a continuing problem within the grouting industry. This is often done to impart lubricity or water retention to the grout, in order to allow an otherwise inadequate pump to function. Unfortunately, use of excessive injection rates are always a temptation for both contractors and their working crews, as less pumping time means earlier job completion and higher profits. Sadly, poor performance, is not often discussed by those involved, and the results of investigations as to the cause are often sealed as a result of legal proceedings.

Deficiencies routinely encountered include the displacement of down-slopes and retaining walls. The slope illustrated in Figure 6B.50 was displaced to such an extent that the exterior wall of the adjacent building (Figure 6B.51) was dislodged some eight inches toward the slope. Clay in the grout, an excessive pumping rate, and probable poor injection sequencing resulted in lateral displacement of the end of the building. The occupant, who was present at the time of the damage, stated to the writer: “The pumping was going real good in the middle of the room when all of a sudden the floor split and everything opened up.”

In a similar case, the building shown in Figure 6B.52 was seriously distorted, requiring extensive structural repair. This happened as a result of a retaining wall (Figure 6B.53) some eight feet away being blown out by an incompetent grouting crew. Again, clay in the grout and an excessive pumping rate were to blame.

In another case, a residential structure continued to settle even though a very large amount of grout had been injected under it, about a year before. While inspecting the structure, the writer happened to observe an excavation being made on an adjacent property. There exposed was a vertical fracture filled with grout extending from a previous grout hole (Figure 6B.54). It ran from the point of injection, a distance exceeding 12 ft (3.6 m), onto the adjoining property. Legal considerations

FIGURE 6B.50 Displaced slope caused by excessive pumping rate and clay in the grout.

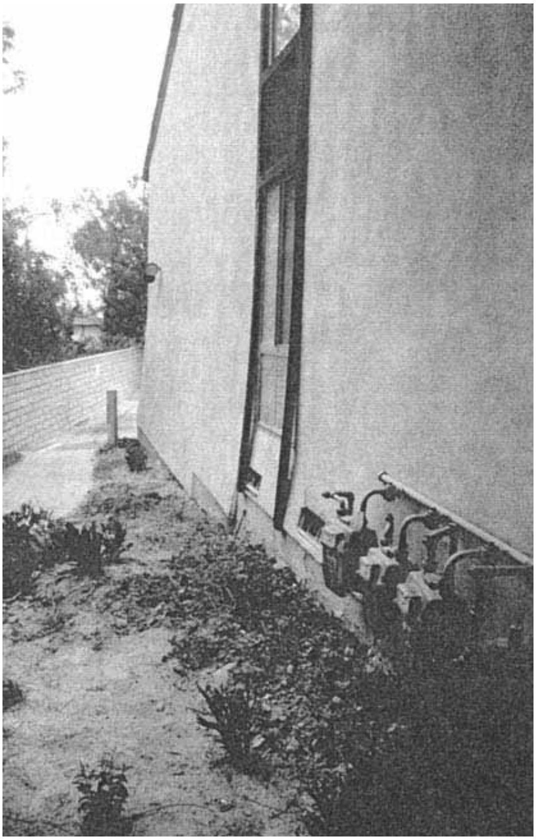

FIGURE 6B.51 Exterior wall of structure dislodged when slope was blown out.

precluded following it further, but it was obvious that most of the grout that had been injected "under" the structure actually traveled a long distance therefrom. No records of the grout injection had been kept, but it was known that the work was done with ready-mixed grout using a standard concrete pump. Because the fines contained in a proper compaction grout aggregate tend to make a somewhat sticky mix that builds up on the fins of a truck mixer, it is virtually impossible to obtain compaction grout of the proper rheology therewith. It is thus a reasonable assumption that the above example was the result of using an inappropriate grout in combination with an excessive injection rate.

The relatively high grout pressures used in compaction grouting would suggest the development of high lateral forces in the ground. Such is often given as an excuse for damage, as discussed above, and the refusal of some contractors to work in such situations. This notwithstanding, experience with properly performed compaction grouting, in literally hundreds of applications in near proximity to retaining walls or unsupported down-slopes, would indicate otherwise. In fact, a common requirement of the procedure is densification of faulty backfill material.

Satisfactory compaction grout mixes can be made with aggregate material conforming to that indicated on Figure 6B.38. Research and experience have proven, however, that increased control re-

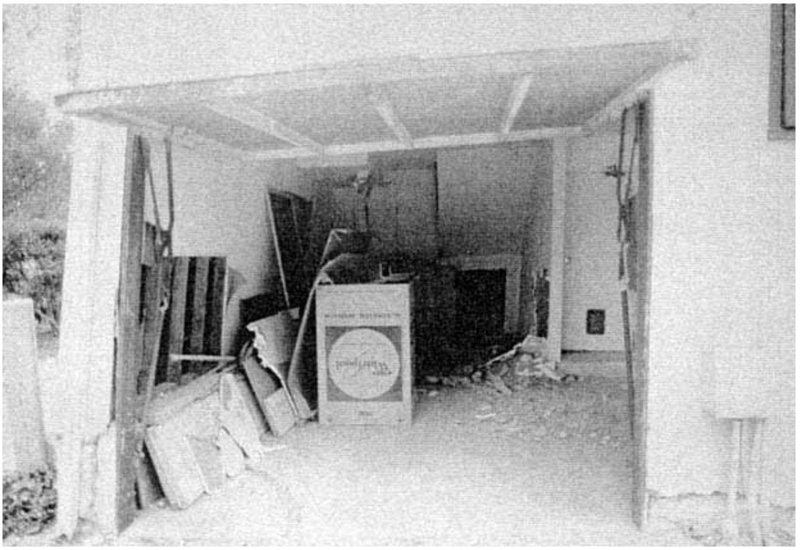

FIGURE 6B.52 Structural distortion caused by blow out of a retaining wall.

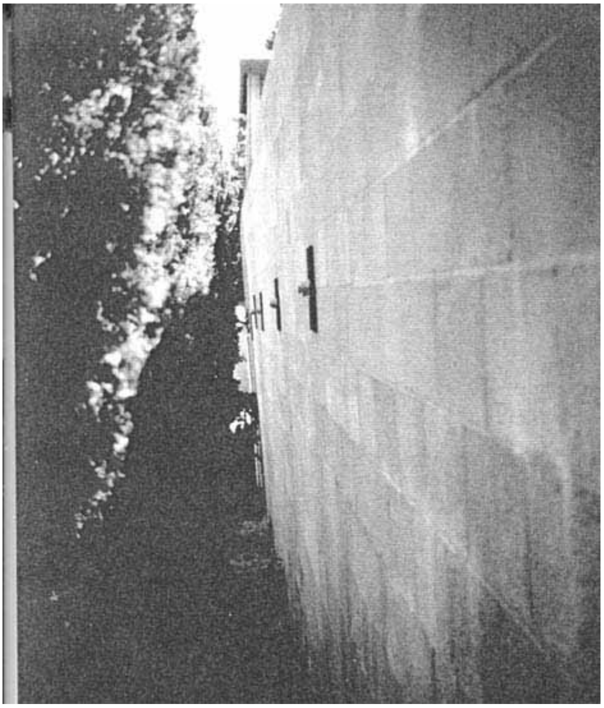

FIGURE 6B.53 Retaining wall displaced by inappropriate grout injection.

FIGURE 6B.54 Thick hydraulic fracture some 12 feet from the intended grout injection location.

sults from the provision of gravel in the aggregate. It has also been found that the inclusion of significant gravel will to some degree mitigate the propensity for high travel indices and resulting hydraulic fractures caused by clay in the grout. Because it is often difficult to obtain sands with the required silt content that is totally free of clay, the inclusion of gravel, which can be either blended into the sand or separately batched into the mixer, becomes even more advantageous.

The benefits of this were vividly observed during grouting of a test hole, where a large diameter injection casing was used. During injection, a distortion of the pressure behavior indicated some sort of a grout leak. When the casing had been raised about thirty feet, the cause of the distortion was readily evident. The casing had split, allowing grout to escape in an uncontrolled manner as shown in Figure 6B.55. Washing the grout from the casing with a water spray revealed gravel tightly packing those areas of the split that were less than about an inch wide, as shown in Figure 6B.56. The preferred range of gradation for grout material, which includes the gravel fraction, is provided in Figure 6B.57. Use of the preferred material is strongly recommended where suitable pumping equipment is available, and its use should be mandatory on sensitive projects.

For most compaction grouting, about 10% common portland cement is mixed with the aggregate. The water is limited to that which will provide a very stiff mortar-like consistency, as shown in Figure 6B.58. Such a mixture will provide an unconfined compressive strength of 400 psi (2760 kPa) or more, which is more than adequate for most work. Where required, higher-strength grouts can be formulated by increasing the cement content and using an aggregate with reduced fines. Such is seldom justified, however, and it is not recommended for most work. It must be recognized

FIGURE 6B.55 Grout escaping from split casing.

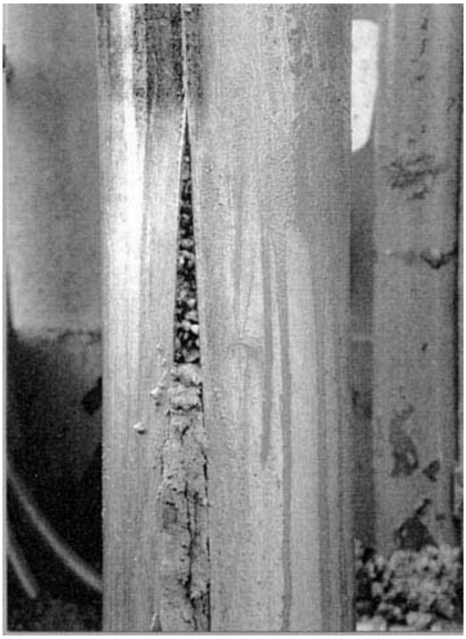

FIGURE 6B.56 Gravel bridges top of split.

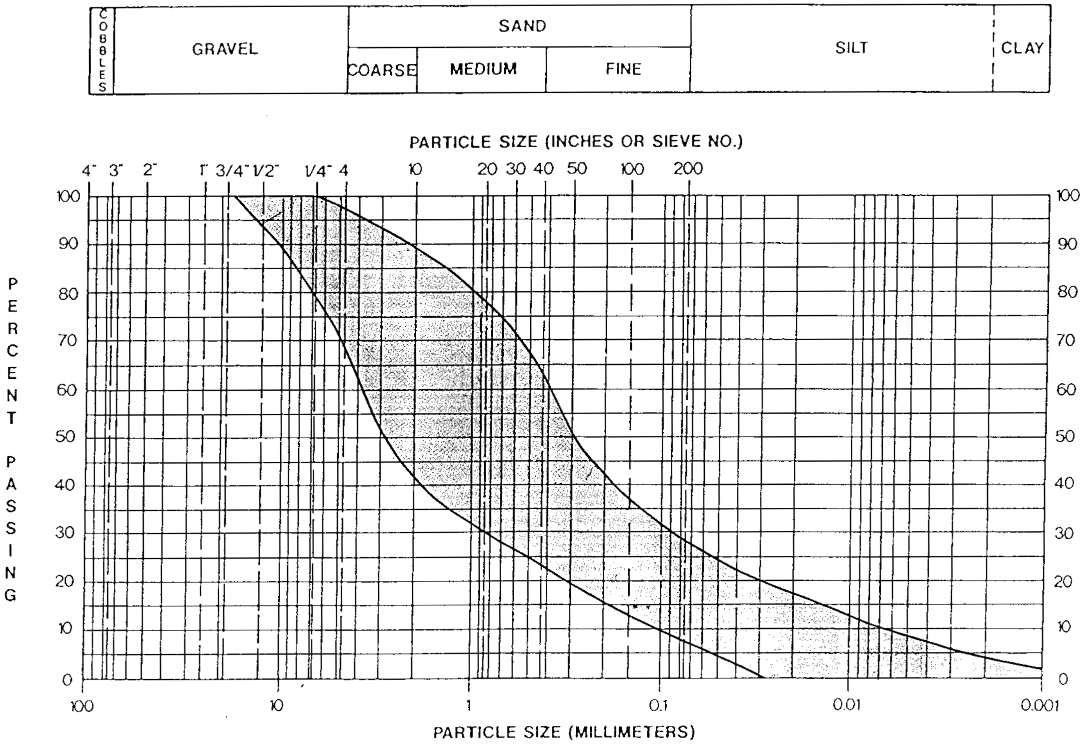

FIGURE 6B.57 Preferred gradation for compaction grout aggregate.

that a sufficient amount of fines in the aggregate is necessary to provide the required pumpability. Very high grout strengths are seldom justified, and their unnecessary specification is not advisable.

In those instances where only a small amount, or even no cementation is desired, another fine material such as hydrated lime, or a pozzolan, can be used in place of the cement. It is also possible to compound a completely suitable grout with no cementing material at all by increasing the fines content of the aggregate. Increasing the fines will, however, require more mix water, so trial mixtures with the available materials, should be assessed before specifying such a mix.

Because the grout injected by properly performed compaction grouting stays in a homogeneous mass near the point of injection, the process allows for controlled soil improvement. It has the advantage of execution without creating a great deal of mess or interference with the normal operations of a facility. Large equipment is not required near the injection location, which allows the work to be performed in areas of confined or poor accessibility. This results in a very wide range of advantageous applications, and no doubt accounts for the procedure's widespread use in North America, where it originated.

Whereas the greatest use of compaction grouting is in connection with the correction of settled buildings, significant applications have been performed on a wide variety of other structures. The process also is used for site improvement, prior to construction of new structures, such as the mitigation of the potential for soil liquefaction. By far, however, the most common applications are in connection with the repair of structural settlement. Because groutjacking involves essentially the same equipment and grout material, settled structures are usually raised to their proper elevations as part of the program to improve the underlying soils (see also section 7.B.2).

FIGURE 6B.58 Very stiff, mortar-like consistency grout.

Faulty foundation soils and resulting settlement of literally thousands of light residential structures have been corrected with compaction grouting. Included have been many large and even very heavy buildings, including structures founded on both pile and large continuous raft foundations. Many successful applications have been made where settlement caused structural damage and resulted in vacation and in some cases condemnation of the structures. The four year old building of Manitou Springs Junior High School in Colorado had to be vacated in 1980, due to severe structural distress resulting from several inches of differential settlement. Compaction grouting not only improved the underlying faulty soil, but also jacked the building back to its proper elevation.

A portion of a five story high wing of a concrete building in Rapid City South Dakota had settled several inches, resulting in serious structural distress. In 1986, faulty soils to a depth of fifty feet were remediated under the shallow foundations, and the settled areas raised to their original grade while the structure remained fully occupied. In 1991, the four story high Inage Welfare Center building in Chiba City, Japan remained fully functional and occupied during the correction of nearly three inches of differential settlement.

The first use of compaction grouting in connection of with a pile supported structure occurred in 1966, in Hollywood, California. Settlement of the seventeen buildings of an apartment complex still under construction (Figure 6B.59) was occurring. The structures were built on a deep canyon fill, which had been deposited over a period of several decades. The fill consisted primarily of a variety of uncompacted soils, but also included many large boulders, and a minor amount of organic waste. The foundation consisted of concrete grade beams supported by end bearing, bell bottom, cast in place concrete piles. Investigation revealed that several of the piles, which varied in depth to more than eighty feet, did not extend to a competent bearing layer as planned, and some actually terminated in massive boulders embedded in the fill. It was considered imperative that no grout be deposited above the pile tip elevation, as this would act negatively and increase the load on the already overloaded end bearing piles.

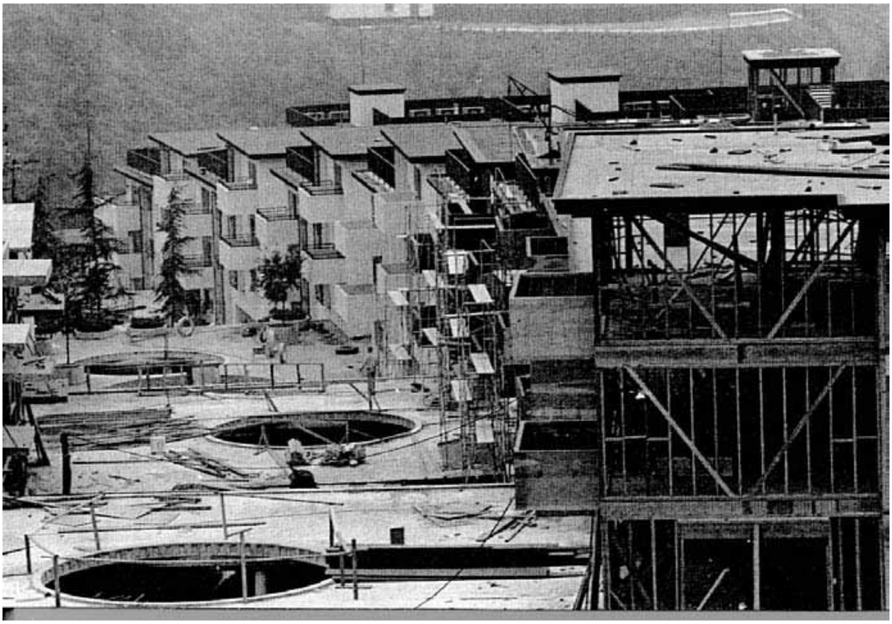

FIGURE 6B.59 Differential settlements of several inches occurred in the 17 buildings of this apartment complex while still under construction.

Accordingly, prior to adoption of a full-scale grouting program, several holes were grouted adjacent to four selected piles. The grout holes were spaced six feet apart, and placed either in a triangular pattern of three holes or a square pattern of four. Grout was injected from the top down, in vertical stages of four feet, starting at the level of the pile tip. Between 12 and 22 cubic feet of grout was injected for each foot of depth. Following grout injection, large test borings were excavated adjacent to the four piles, and extending beyond the pile tips, allowing visual inspection. The photograph shown as Figure 6B.60, was taken from one of the borings, which was about 60 feet deep. As can be observed, grout was under the pile tip, as required, and only extended above that elevation by less than one foot.

Because this was a landmark project in the development of the compaction grouting process, extensive other testing and observation was performed. Load tests (Figure 6B.61) to a level 1.5 times the design capacity were performed on several of the grouted piles, with virtually no further settlement. Regular inspections by the structural engineer and second-order optical surveys were made for a period of 10 years following completion and occupancy of the buildings. No structural damage was found during that period, and the maximum vertical settlement observed was less than 0.1 inch.

In projects such as this, where the grout deposition zone must not extend above a given level, it is important to "seal off" the boundary by at least two stages of grout, injected from the top down. This is especially important in such instances as that cited, as injected grout will naturally tend to flow into the weaker soils. Where a considerable depth under the piles needs improvement, bottom-up procedures can be used once the upper boundaries have been established.

In another noteworthy case, friction piles were used for support of the West Orange County, California, Municipal Courts Building (Figure 6B.62). The original geotechnical investigation provided for the piles to extend from the surface to depths of 25 to 40 feet. Subsequent to the original investigation and prior to construction, the building was relocated approximately 100 feet to the west. It is

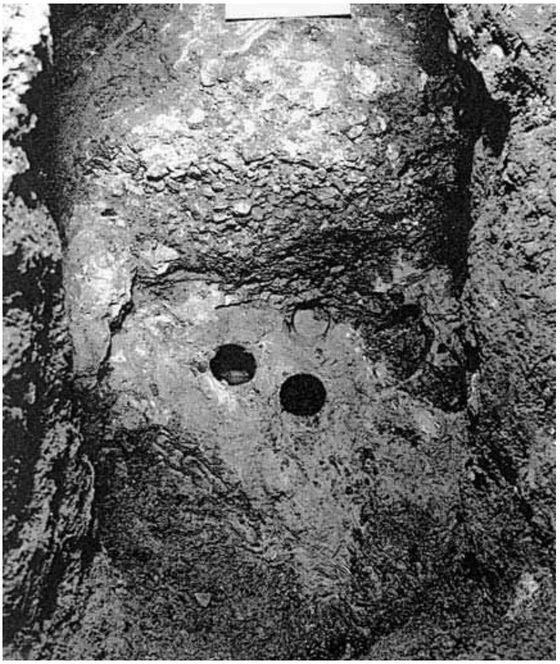

FIGURE 6B.60 Grout under pile tip.

FIGURE 6B.61 Load test in progress.

FIGURE 6B.62 West Orange County Municipal Courts building underlain by faulty soil.

of contemporary design, basically a reinforced concrete frame with both concrete and masonry walls. The floor and roof slabs are of reinforced concrete. Approximately two-thirds of the structure is one-story with a slab on grade floor, the remainder consisting of two structural floors above a basement. The foundation system consists of friction piles, cast in place, in driven corrugated steel shells. At the time of construction, the groundwater level was approximately at the basement floor elevation and, in fact, an extensive underfloor drainage system was installed to prevent the development of uplift pressures.

Distress was noted shortly after completion of construction. The building nonetheless remained in full service until a major reinforced concrete roof girder fractured, eight years later. An investigation disclosed that five inches of differential settlement had occurred, and that the structure was suffering from significant structural overstress. By moving the building from the originally planned location, the west half had inadvertently been placed over a wedge of very low density peat that existed only a few feet deeper than the pile tips, as illustrated in Figure 6B.63. As part of the investigation, individual piles were test loaded in increments up to twice their design capacity. Even with supporting the imposed loads for five days, no significant deflection of the piles was observed. It was concluded that the settlement involved not only the structure and its pile foundation, but also the entire block of soil above the peat layer.

Compaction grouting was thus performed in the peat layer underlying the pile tips. Two inch I.D. casing was placed to the top of the peat layer, and grout injected in stages from the top down, until good bearing soil was encountered. Interestingly, the building remained open and in full service during the work. The work was divided into five phases, each involving one courtroom and ancillary facilities. Access was through windows in the office areas adjacent to the courtrooms. Drilling of the grout holes, which were as deep as 65 feet (m) was by hand-held, rotary wash, equipment (Figure 6B.64), which could be quickly moved and operated throughout the otherwise restricted area. The grout pump remained outside the structure, and up to two hundred feet of hose was used to reach the furthest holes (Figure 6B.65).

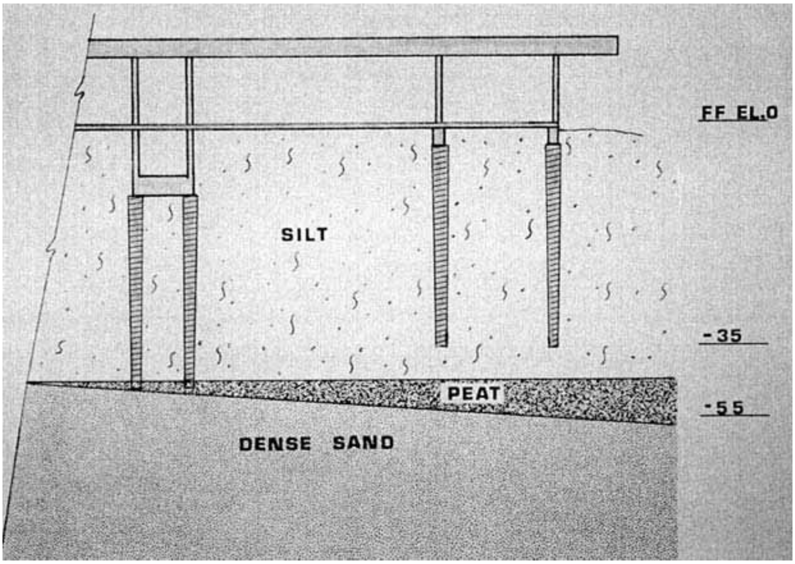

FIGURE 6B.63 Cross section of building showing peat layer underlying pile tips.

FIGURE 6B.64 Drilling in restricted interior area with compact hand-held drills.

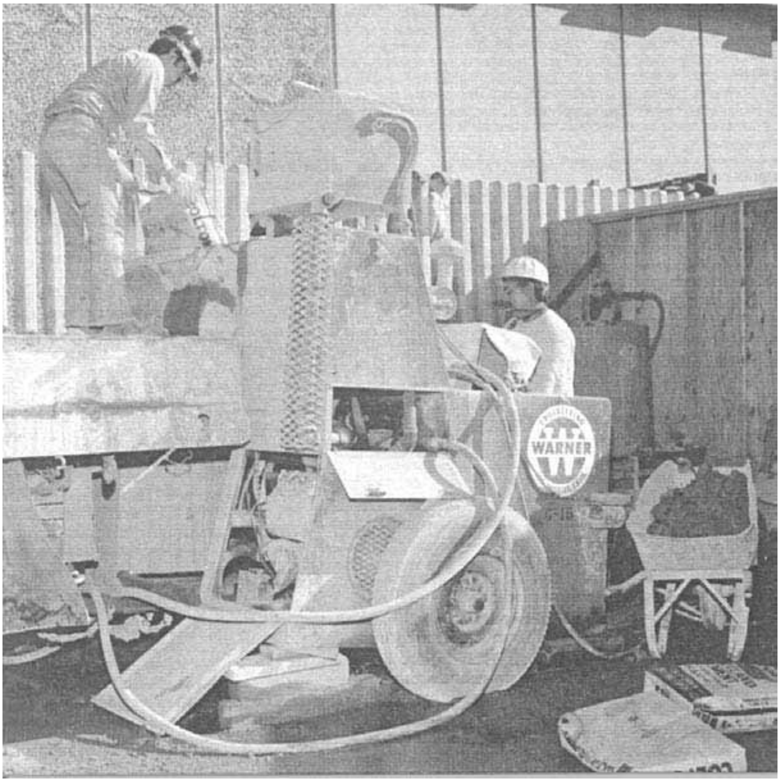

FIGURE 6B.65 The grout pump remained outside the building.

Early in the settlement investigation, survey monuments were placed into the roof slab, over each pile location. They were monitored throughout the repair program and three, six, and nine months after completion. Although the original intent was to monitor them for several years following the repair effort, as no detectable movement occurred, further monitoring was canceled after the nine month survey. A visual inspection by the writer in 1998, 22 years after the work was performed, found no evidence of distress or the prior major settlement damage. In fact, none of the occupants had been there when the work was done, and all expressed surprise at the idea that the building once had a serious problem.

In 1978, a newly constructed, cantilevered concrete sheet pile sea wall (Figure 6B.66) began to fail as backfill was initiated. The wall was composed of pretensioned, interlocking concrete piles, twelve inches thick by four feet wide. They had been jetted into the ground, underlying the water, utilizing a barge-mounted crane. The wall was to function as a continuous cantilever structure, as illustrated in Figure 6B.67. Investigation determined that a silt layer under the sea floor had been badly disturbed during the jetting operation, along the entire length of the 11/2 mile wall.

Because the work was underwater, and any cracks or displacements that developed on the sea floor could not be readily seen, a very conservative grouting program was conducted. Grout holes were located adjacent to the existing bulkhead faces, at a spacing of 2.4 m (8 ft). Grout conforming to the Brown and Warner (1973) criteria was injected into vertical stages of 0.6 to 1.8 m (2 to 6 ft) from the top down, at a maximum rate of 0.03 m^3 (1 ft^3) per minute. Grout pressures varied from about 0.34 to 1 MPa (50 to 150 psi) but were most often within a range of 0.34 to 0.48 MPa (50 to

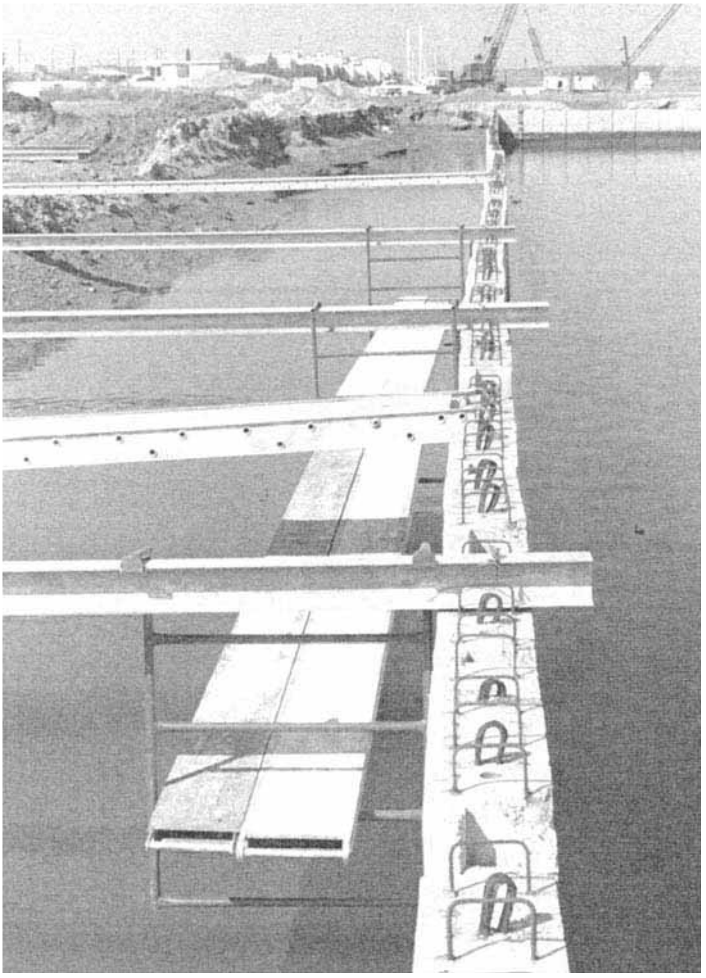

FIGURE 6B.66 Cantilever sea wall tilted outward when backfill was initiated.

70 psi). Approximately 0.3 m^3 (10 cu ft) of grout was injected for each linear meter (3.0 ft) of bulkhead wall.

Two rows of cone penetrometer test probes were placed on the water side of the wall, where dense soil was required for the cantilever to function. They were made both before and after the grouting. The first row was approximately 1.5 m (5 ft) from the face of the bulkhead, with probes being made every 4.6 m (15 ft). The second row was 4.6 m (15 ft) off of the bulkhead, with probes at 7.6 m (25 ft) intervals. The work was accomplished using a 10 ton capacity penetrometer secured to a floating barge, and the total penetration force included that portion of the weight of the barge lifted by the force acting on the penetrometer rods. When the rods could no longer penetrate, the barge was lifted partly out of the water, which established refusal.

Cone tip resistance prior to grouting was less than 40 kg/cm^2 (41 T/ft^2) to a depth of 7 m (23 ft),

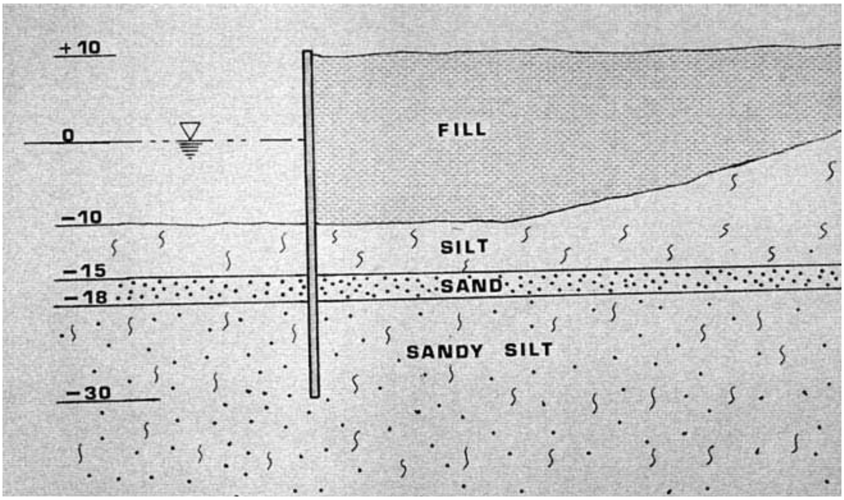

FIGURE 6B.67 Cross section of sea wall installation as designed.

and decreased to about 20 kg/cm^2 (20 T/ft^2) below that depth in the first row. In the second row, beginning values were less than 5 kg/cm^2 (5 T/ ft^2) to a depth of 7 m (23 ft), and about 20 kg/cm^2 (20 T/ft^2) below that depth. The postgrouting value increased with depth in both rows to nearly 50 kg/cm^2 (51 t/ft^2) at about 6 m (20 ft), below which refusal was reached at around 80 kg/cm^2 (82 T/ft^2) at about 8 m (26 ft).

Compaction grouting was an especially suitable method of repair, as it did not require the use of heavy equipment at the wall location, which was surrounded by water. Grout holes were drilled with hand-held pneumatically powered drills (Figure 6B.68). The grout pump remained on the adjoining grade, with the delivery line supplying a standard header (Figure 6B.69). Extremely close control of the work was required, as deformation of the piles and resulting cracks could not be tolerated. Cracks in the piles would result in rapid corrosion of the prestressing tendons, due to the salt water environment. Load cells, anchored to land-side deadmen, were used to monitor forces on the wall. A diver was used continually during grout injection to monitor the sea bed for any displacements or grout leakage. Because of the sensitivity of underwater injection, the pumping rate was limited to less than one cubic foot per minute. Grout pressures varied between 150 and 300 psi. Following the remedial work, the wall was backfilled, and the planned construction of a commercial center progressed. The writer has visited the site several times since the work was done, and has always found it to be performing well, with no indication of the former distress.

Compaction grouting is especially well suited for the densification of backfill material that was not properly compacted when originally placed. Many projects have been completed where faulty fills behind retaining walls, or around, and over buried pipelines have been improved. Figure 6B.70 illustrates such a case, where settlement of the fill behind the basement wall of a shopping mall is being densified. As can be seen, the operation is fairly orderly, and there is relatively little interference to the shoppers, as the work is being done while the stores remain open.

Shortly after the original opening of the Japanese Pearl Divers attraction at Sea World in San Diego, California, unacceptable leakage of a man-made lagoon threatened closure of the facility. The problem was the result of insufficient compaction of backfill materials surrounding the underwater viewing areas during original construction. Compaction grouting was used to densify the faulty soils (Figure 6B.71). Most of the work was under raised decks and shops over the lagoon, with only about three feet of overhead clearance. Two inch diameter holes were drilled through the

FIGURE 6B.68 Drilling of underwater grout holes.

wood decks to allow drilling of grout holes to proceed from above. Once the hole at a given location was completed, wood plugs were placed to restore the deck. The grout mixer and pump were located outside the park, about 600 feet distant from the work (Figure 6B.72), and the grout delivery hose line was picked up at the end of each work shift, which was at night and the early morning hours prior to opening of the attraction. With exception of the absence of water in the lagoon, there was thus no indication of a problem to the visitors of the attraction, which remained open and fully operational during the work. The project was monitored for several years following the work, with no further leakage or other distress noted.

The 1991 failure of a large sewer conduit in Houston, Texas, resulted in collapse of a road. Investigation established that the backfill around and overlying some 8000 feet of the pipe was of very low density. The silty and clayey sand backfill was found to have Standard Penetration Test "N" values averaging 12, with many below 10. Compaction grout was injected through rows of holes on each side of the pipe, at ten foot centers. Verification that the specification requirements of improvement to an average Standard Penetration Test N value of 20 with no test below 15 were easily

FIGURE 6B.69 Grout injection header.

FIGURE 6B.70 Grouting faulty backfill in shopping center.

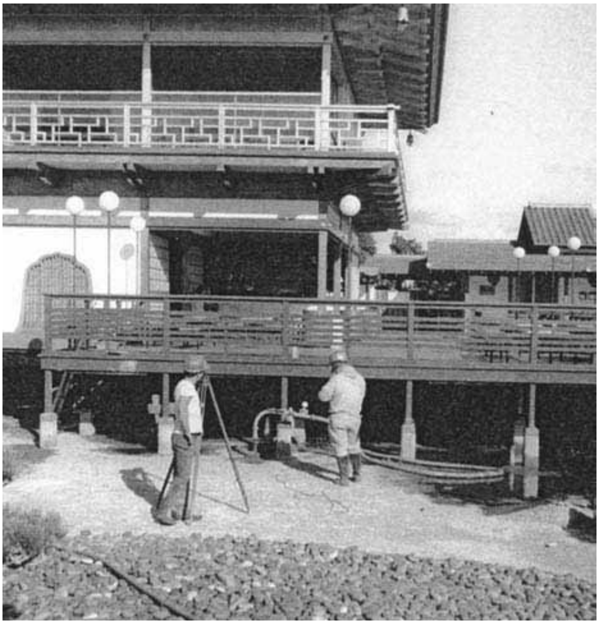

FIGURE 6B.71 Grouting under tourist attraction with little evidence of the ongoing work visible.

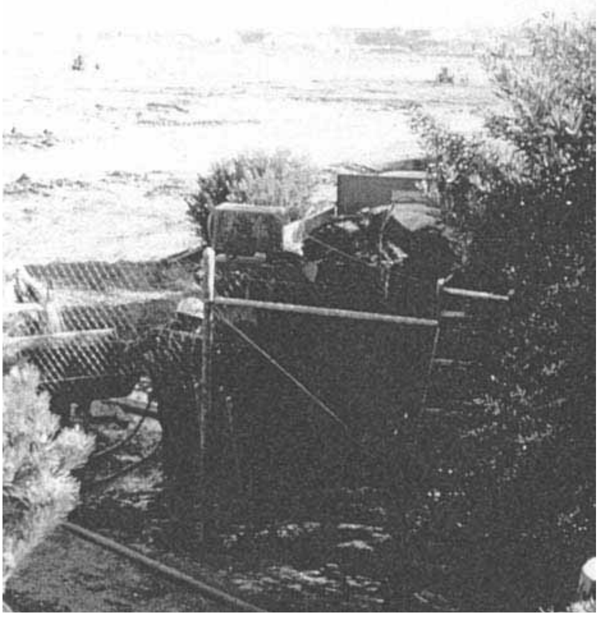

FIGURE 6B.72 Grout pump and big equipment are out of sight.

met. One hundred thirty-four postgrouting SPT tests, made between the grout holes by the same drill rig and operator, produced N values averaging 22.3.

In another case, CPT testing was used to evaluate the appropriateness of compaction grouting to densify loose backfill soil around and underlying several miles of distressed storm drain pipe. The backfill materials were fine to medium sand and silty sand, with silt and clayey silt lenses. The drains were under the pavement of a major freeway, and had resulted in surface settlement so great that lanes had to be closed to traffic. Cone penetrometer testing disclosed cone tip resistance (Qc) of the defective soils, generally less than 30 tons/ft^2 (29 kg/cm^2). Resistance on the order of at least 50 tons/ft^2 (49 kg/cm^2) would be required to prevent settlement. The test section involved injection of 9.5 m^3 (318 ft^3) of grout into 32 injection points spaced at 2.1 m (7 ft) on center, using a maximum grout pressure of 2.8 MPa (400 psi). Details as to the aggregate gradation and pumping rate of the work, which was done on an emergency basis, are not known. The required, after grouting, minimum CPT values, of 50 tons/ft^2 (49 kg/cm^2) were easily met, as illustrated in Figure 6B.73.

As a result of the successful test application, a contract was let to improve the soil around several miles of the storm drains. Bottom plugged, two inch, proprietary, flush wall casing was driven on seven foot centers (Figure 6B.74) in rows on each side of the drains. Prior to injection, the casing was raised about one foot and the plug knocked out. Grout was then pumped at a rate of less than 0.04 m^3 (1.5 ft^3) per minute. As in the previous example, CPT tests were taken both before and after the grout injection. The required degree of soil improvement was easily achieved.

Compaction grouting is often used to raise the density of granular soils, for the purpose of mitigating the risk of liquefaction during earthquakes. In 1983, compaction grouting was used to improve the site for an addition to an existing pile-supported hospital building. Because the work was immediately adjacent to the existing structure, which remained fully operational, excessive noise or

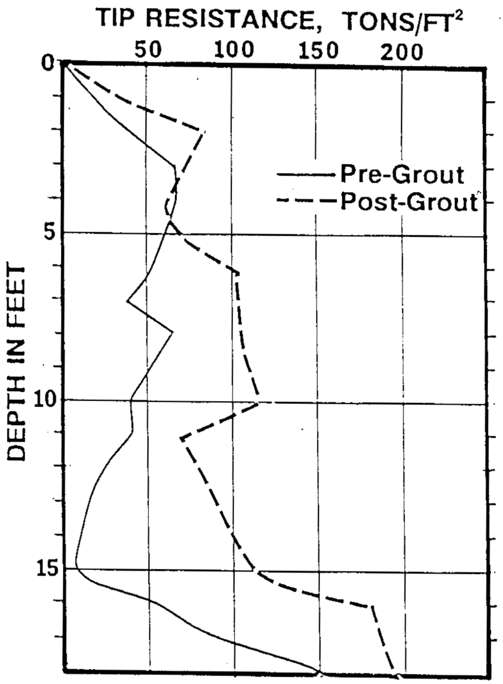

FIGURE 6B.73 CPT values before and after grouting of faulty trench backfill.

FIGURE 6B.74 Driving grout injection casing.

vibration could not be tolerated. Of the several candidate methods evaluated, compaction grouting was the only one that met these requirements. The upper level of the soils from 7 to 17 feet in depth was first grouted. Then the remaining soils were treated from the bottom up, starting at the bottom of the liquefiable sands at a depth of 34 feet.

Prior to construction of the addition, grout holes were drilled on a square grid over the entire site (Figure 6B.75). The holes were spaced eight feet apart, and average grout take was 2.7 cubic feet of grout for each foot of hole. The grout was composed of silty sand, cement, and water mixed to a stiff consistency, with slump between 25 to 50 mm (1 to 2 in). Postinjection testing indicated an increase in the density of the in-place soils of about 20%. The site was in an area that experienced widespread damage from the Loma Prieda earthquake in 1989. The hospital site performed well and no indication of liquefaction was evident.

In 1988, compaction grouting was used to densify foundation soils underlying Chessman Dam, near Helena, Montana. This was the first of many similar projects in which compaction grouting has been employed to upgrade the seismic resistance of dams and other large civil works. In 1999, a massive project was started to improve the soil underlying the Narita airport, which serves Tokyo, Japan, in order to mitigate the risk of liquefaction. That effort will involve one of the largest undertakings to date, in terms of amount of grout used and volume of soil treated.

Another area, in which compaction grouting has been found advantageous, is prevention of surface damage as a result of soft ground tunneling. A phenomenon is well understood wherein a trough of soil overlying such tunnels occurs as the tunnel shield passes. This zone of disturbed soil starts directly over the tunnel and extends up and out until it reaches the surface. It will often extend beyond the foundations of adjacent structures, and can be very damaging, not only to the structures, but also to underground pipes and utilities within the zone of disturbance.

In 1980, a large evaluation program was conducted in the early stages of construction of the Bolton Hills tunnel, part of the Baltimore Metro project. A test application was conducted in a section of the actual tunnel where the adjacent structures were of minimal value. Compaction grouting

FIGURE 6B.75 Small track-mounted rig used for drilling grout holes.

was performed immediately over the tunnel shield as it progressed. This was accomplished through a single row of grout holes, which were drilled from the surface of the street above and extended to within about five feet of the crown of the tunnel (Figure 6B.76). A single stage of grout was injected immediately after the shield passed, in the tunnel below. The trial, which included extensive instrumentation and evaluation of the adjacent soils, was so successful that the method was adopted for protection of the historical and irreplaceable structures on the remainder of the tunnel route.

Since that early experience, compaction grouting has been used in a similar manner on a number of other significant tunnels constructed in both North and South America. It was also specified for use in construction of the Taipei Metro, in Taipei, Taiwan, Republic of China.

Another extensive use of compaction grouting is in connection with distressed buried pipelines. In 1976, a 96 inch drainage conduit, which was badly overstressed and deformed as a result of excessive loading, was repaired with compaction grouting. Because access to the overlying area was not available, all work was done from within. Twelve foot long, radial holes were drilled with hand-held equipment from within the conduit (Figure 6B.77). Grout was then pumped in stages of about one foot, from the bottom out. Excessive deformations of the pipe were removed during the grout injection.

In 1977, the heading of a flexible liner plate tunnel, which was under construction, began to sink uncontrollably. Compaction grout was injected to stabilize a highly organic silt layer found to extend from the invert of the tunnel, a depth of about 12 feet. Once the faulty ground was sufficiently improved, the heading of the tunnel was grout jacked back to its proper elevation. All of the work was done from within the tunnel (Figure 6B.78), as no disturbance of traffic on the major highway above was allowed.

In 1991, the process was used to stabilize defective soil underlying an 84 inch diameter conduit, connecting to a water treatment plant in Des Moines, Iowa. The plant had been underpinned with micro-piles; however, such piles could not be readily tied to the round pipe section. The

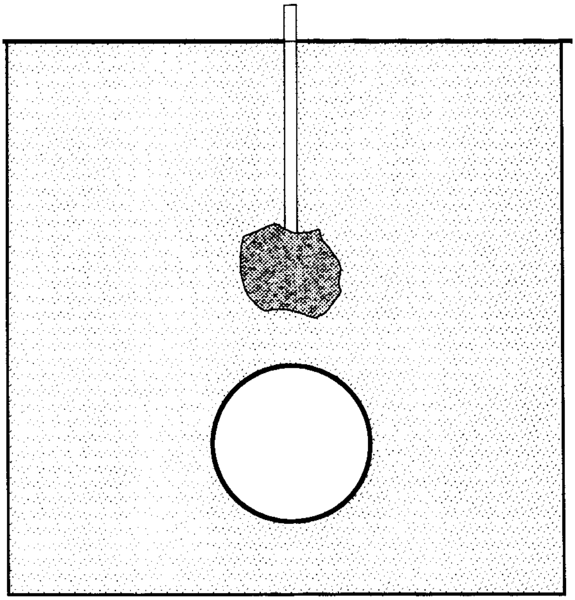

FIGURE 6B.76 Injection of grout to densify soils as they loosen.

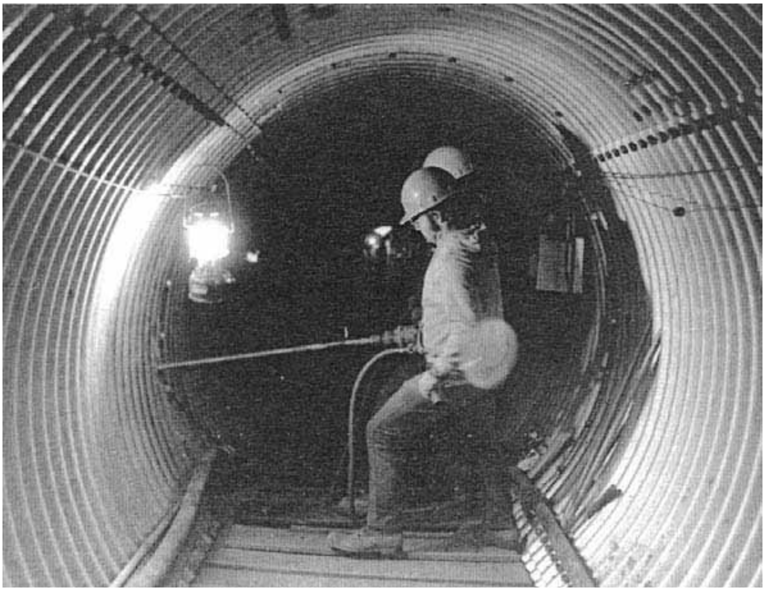

FIGURE 6B.77 Drilling of 12 foot long radial grout holes from inside drainage conduit.

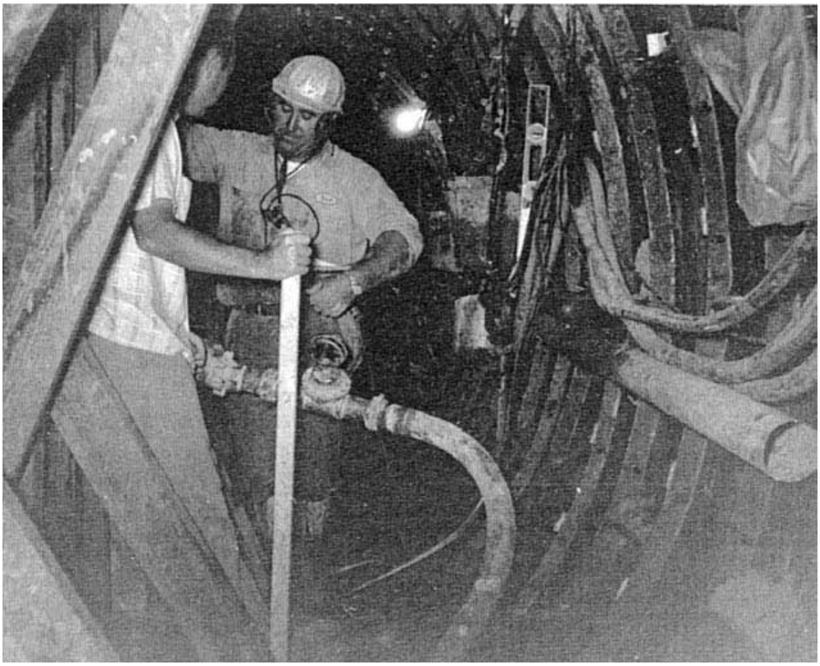

FIGURE 6B.78 Settlement correction to heading of liner plate tunnel.

grouting was used to not only stabilize the underlying soil, but to jack the pipe up to its correct level as well.

Perhaps the most sensitive of any geotechnical remedial applications involves work in water-retaining embankments. Should excessive pore pressures develop, uplift leading to complete failure of the embankment could occur. Obviously, should hydraulic fracturing occur, leakage through the embankment would present an extreme risk, which could also lead to complete failure. Because compaction grouting allows for close control of the grout deposition location, when properly performed, it is one of the safest remedial procedures for correcting defects in such embankments. In order to achieve the maximum assurance against creating hydraulic fracturing, careful design and rigid control of the grout mix to be used is of crucial importance. In this regard, a minimum of 25% of the grout aggregate should be larger than a number 4 sieve. Additionally, the aggregate should be completely free of any clay component.

In 1996, a sinkhole developed in the core of Bennett Dam (Figure 6B.79), which is in the northern part of British Columbia, Canada. Bennett is one of the largest embankment dams in the world. Extensive investigation determined the disturbed zone to be generally less than 20 feet across, but about 400 feet deep. The investigation also uncovered a second sinkhole of somewhat less magnitude. After much deliberation, compaction grouting was selected as the best method for remediation of the defect.

This was a critical problem and warranted the very best design and remedial implementation. Extensive investigation was made to accurately map the sinkholes. To monitor any changes of condition within the embankment or surface profile, an extensive installation of instrumentation was installed. Requirements for continuous computer monitoring (Figure 6B.80) of the parameters of both the drilling and the grout injection were adopted. Continuous analysis of the retrieved data was made, along with appropriate adjustments in the conduct of the work.

FIGURE 6B.79 Bennett dam in British Columbia, Canada, one of the worlds largest earth embankment dams.

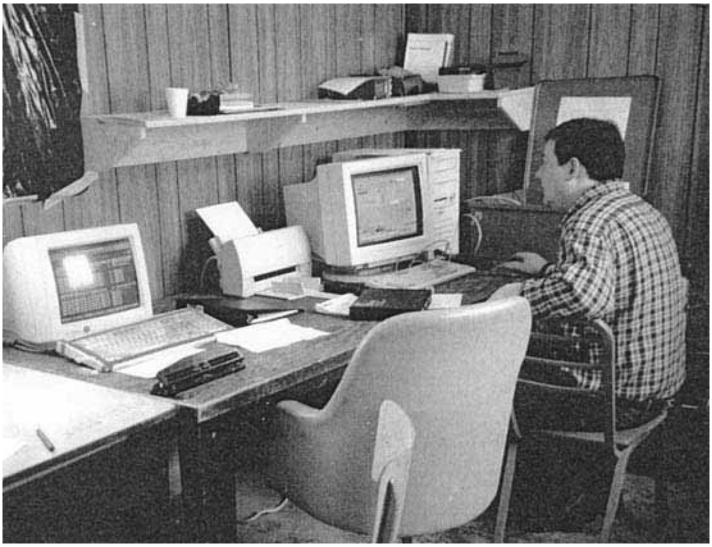

FIGURE 6B.80 Continuous real time computer monitoring.

Compaction grouting had not been previously accomplished to such depths or under such sensitive conditions, and huge losses would result if the embankment were breeched during the repair. Additionally, the work was to be done in severe cold weather, during the winter months. This would require protective, heated enclosures (Figure 6B.81). Thus, a full-scale test hole was drilled and injected, at a site of similar soil materials in Vancouver, prior to any work being accomplished on the dam itself. Among the objectives of the test hole were to assure that the proposed drilling and grouting methods, which were to be continuously electronically monitored, could meet the specification requirements. The hole was required to be within 1% percent of verticality over its entire length. The drilling method also had to prevent any positive liquid head from being imposed within the embankment.

Further objectives of the test hole were to confirm pumpability of the proposed grout mix, which was to contain a minimum of 25% of aggregate retained on a number 4 sieve. The aggregate was also to be essentially free of clay. Creation of a hydraulic fracture signature, by intentionally pumping too fast and/or adding clay to the grout, was also an important objective. This would allow the personnel who would be monitoring the actual work on the dam to become more familiar with areas deserving special alertness. All objectives of the test hole were realized and additional shortcomings of the contractor's operation were identified. Although it represented a considerable cost, the test proved to be extremely valuable to the overall operation, and is credited with enabling the successful completion of the work.

To accurately drill the grout holes, a powerful dual rotary drill rig, such as might be used for water well drilling, was used (Figure 6B.82). It was capable of simultaneously turning both the drill string and the casing in opposing directions. A heavy 6 inch steel casing, was used, and the joints were welded to assure tightness and proper alignment. The completed holes were checked for verti-

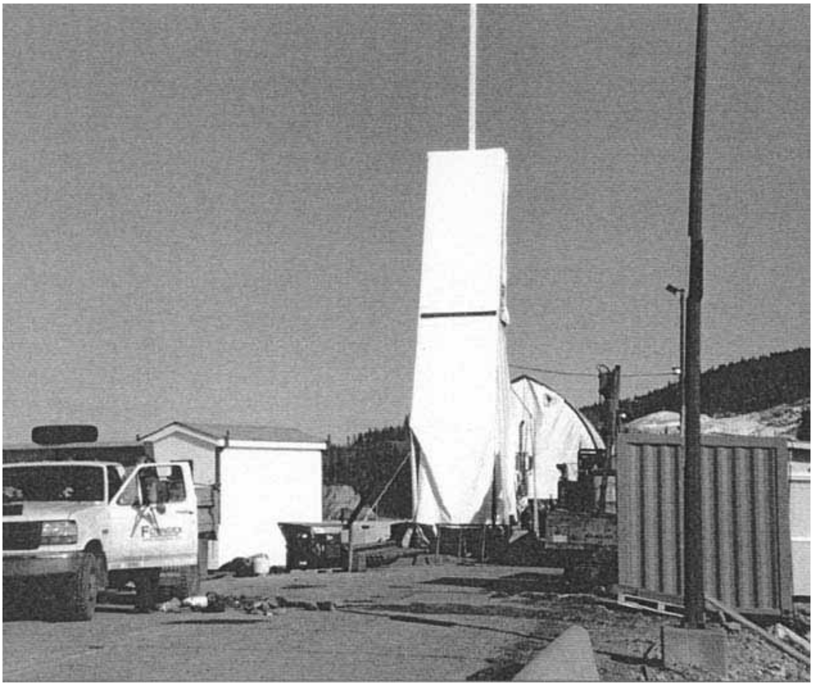

FIGURE 6B.81 Cold weather enclosures.

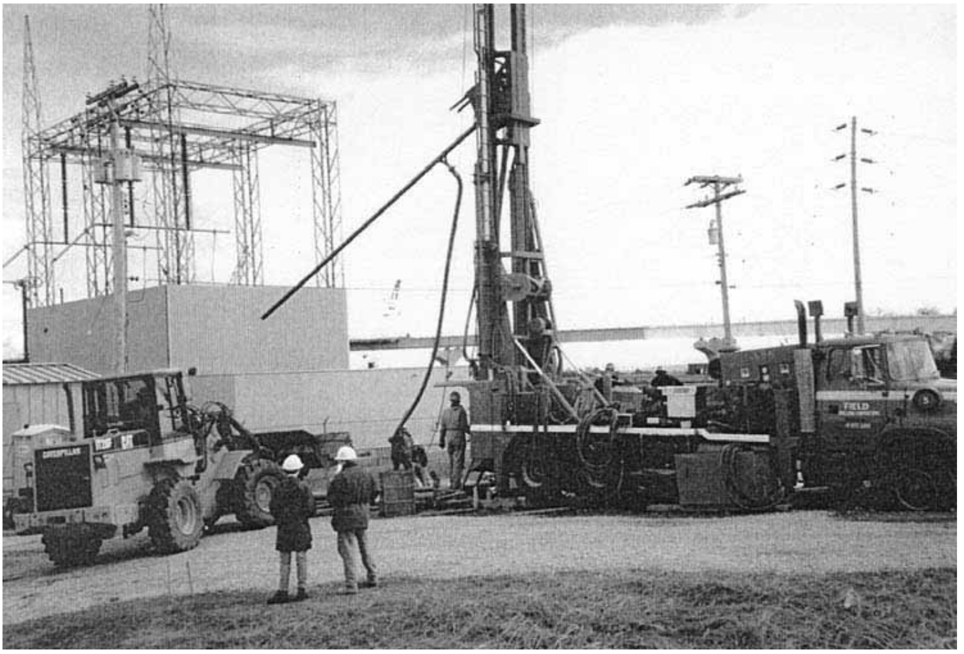

FIGURE 6B.82 Powerful rotary wash drill rig handled casing easily.

cality with a gyro instrument, and found to be well within the required 1% tolerance. The contractor had originally proposed to inject the grout directly through the 6 inch casing, however it was found that the grout would exert a positive pressure of about 0.3 psi for each vertical foot. This could result in unacceptable pressures at the bottom of the hole, so a smaller HWL 4 inch ID, flush wall casing was inserted and sealed off at the bottom of the hole with a packer. With the smaller casing, a negative pressure of about 0.3 psi per foot was experienced.

Analysis indicated that in order to preclude excessively high pore pressures developing in the embankment, as very slow pumping rate of less than 0.5 cubic feet (14 L) of grout per minute, was required. The grout was injected from the bottom up, in stages of one foot (0.3 m). The 6 inch casing and the contained HWL grout tube were pulled by the drill rig for each stage. As each 20 foot increment was withdrawn, the 6 inch casing was cut with a pipe cutter, the threaded HWL casing joint broken, and the header rejoined by welding for the next pull. As expected, the grout take was quite variable, due to the variation of the sinkhole boundaries. Required grout pressure for these very deep holes was on the order of 1400 psi (9660 kPa).

An extensive array of piezometers was installed around the sinkholes prior to grout injection, and pore pressures closely monitored throughout the work. Occasional excessive pressure did require a pause of grout injection in the early stages of the work. This was the result of the contractor exceeding the specified pumping rate. Once he obtained a proper grout pump, and his crew gained experience, the injection proceeded at a rate of about 0.33 cubic feet per minute (8.6 L/min), without further problems. As part of the final remedial work, the soil cap material was excavated down to the top of the injected grout. The grout was found to exist in a perfect columnar mass, as expected (Figure 6B.83). The success of this project proved the engineering validity of a properly designed and controlled compaction grouting program.

Another interesting application of compaction grouting in a water-retaining embankment was

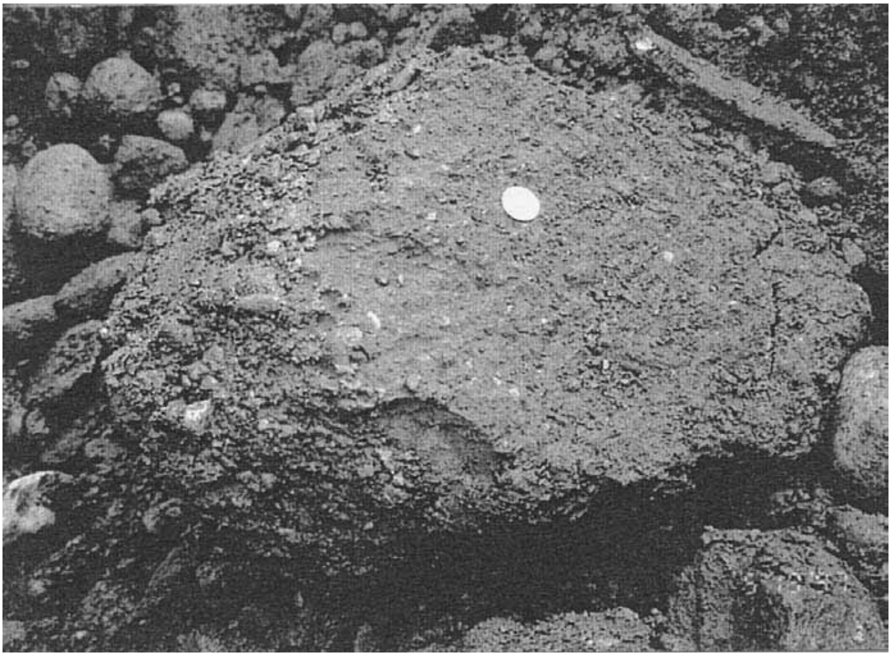

FIGURE 6B.83 Near-perfect columnar mass of grout exposed by excavation of overburden.

the emergency repair at mile 55 of the California Aqueduct (Figure 6B.84). Massive leakage was observed at the base of a substantial embankment (Figure 6B.85), and complete failure appeared imminent. A sinkhole had developed about 300 feet upstream of the leak, and a section of the concrete lining had fallen into it.

The massive leakage was initially controlled by rapid insertion of about 80 burlap-encased, 50 pound paper bags of bentonite, followed by the rapid pumping of 10 cubic yards of ready-mixed concrete into the sinkhole. The concrete had about 30% minus ½ inch gravel and contained 940 pounds of cement per cubic yard. It would have been very advantageous to include an antiwashout admixture; however, this material was not immediately available from the only concrete plant in the area of the work. Although this work was done on a panic basis, and with little control, it was successful in substantially reducing the leakage. Because it was likely that the blockage of flow was in a cavernous void near the sinkhole, and substantial piping voids probably remained in the embankment, further work was required to assure against possible failure of the embankment

A method to "find" any such voids, and fill them, was needed. Compaction grouting was selected as the safest method, because it would provide for the greatest control of the grout deposition area and, properly performed, would not result in hydraulic fracturing of the embankment. This was a particular concern, because of the bagged bentonite, which had been inserted during the emergency. Neat bentonite gel is known to act as a fluid in soil, and would likely initiate a hydraulic fracture if subject to a sufficient pressure, which could result from grout injection. A very carefully controlled compaction grouting program was thus adopted.

Initially, a single row of grout holes, spaced at 16 foot increments, was established adjacent to the top of the channel for the 300 feet segment. Following injection of these primary holes, secondary holes were placed midway between, for a final spacing of 8 feet. Once this first row of holes was completed, four additional parallel rows were established across the embankment. These rows were

FIGURE 6B.84 Mile 55 of the California Aqueduct during the emergency grouting repairs.

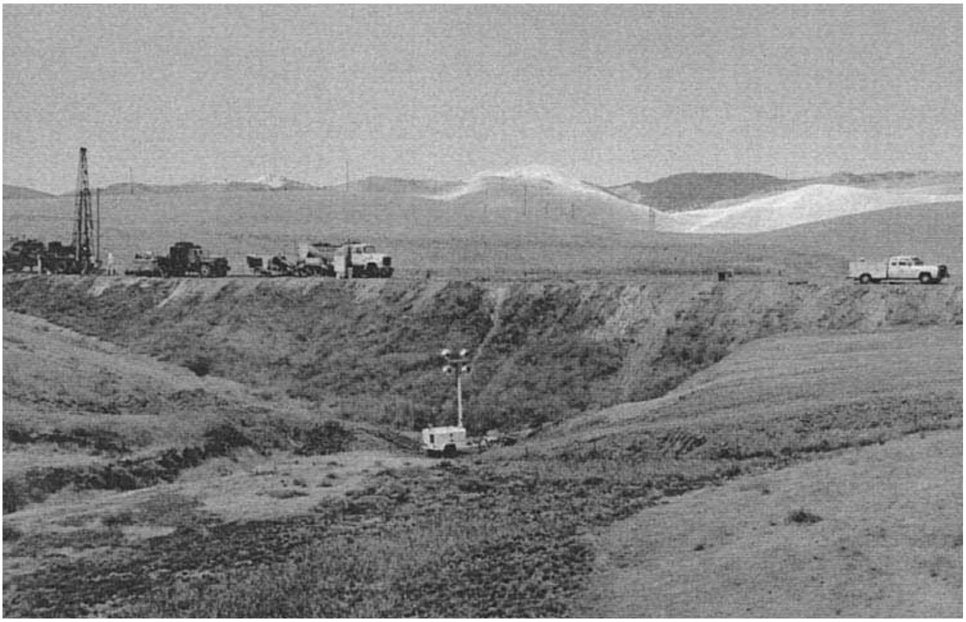

FIGURE 6B.85 Leakage was at the base of the embankment.

about 8 feet apart, with holes located at a spacing of 8 feet. They were grouted using split spacing, with a primary–secondary order of injection.

Grout was prepared using aggregate of the preferable gradation, containing about 25% gravel and about 10% cement. It was injected from the bottom up, in one foot stages, at a conservatively low rate of one-half cubic foot per minute. Because of the sensitivity of the work, which was done with the aqueduct in continuous operation, continuous monitoring of the embankment was carried out. In addition to careful observation and recording of the grout injection parameters, this monitoring included a manometer level control system and continual observation of the ground surface and exposed top portion of the canal lining by optical survey, which is evident in Figure 6B.86, showing the work in progress.

Because of the emergency conditions, the grouting crews arrived within one day of discovery of the problem. They worked 24 hours a day until the work was essentially completed. The rapid mobilization precluded continuous computer monitoring, which would have been preferred. Very carefully made manual records were kept, however, which were entered into a computer database immediately upon completion of injection of each hole. Graphical printouts thereof were assembled on the wall of the trailer office (Figure 6B.87), so that the actual conditions as dictated by the grout injection behavior could be easily observed. The work was a success in that it allowed the aqueduct to function for several months until a scheduled shutdown. A permanent repair consisting of a continuous membrane protected by a shotcrete overlay was then made.

Concurrently with the start of grouting, a subsurface geological investigation, which eventually included 12 carefully logged bore holes and one test pit, was made. It was determined that the embankment had been founded on soil materials underlain by a gypsiferous formation. The piping leakage was determined to most likely be the result of solution of the gypsiferous materials from water permeating through an overlying sand stratum, which was under the embankment fill. This

FIGURE 6B.86 Remedial grouting in progress. Note survey activity to ensure against any surface heave or other movement.

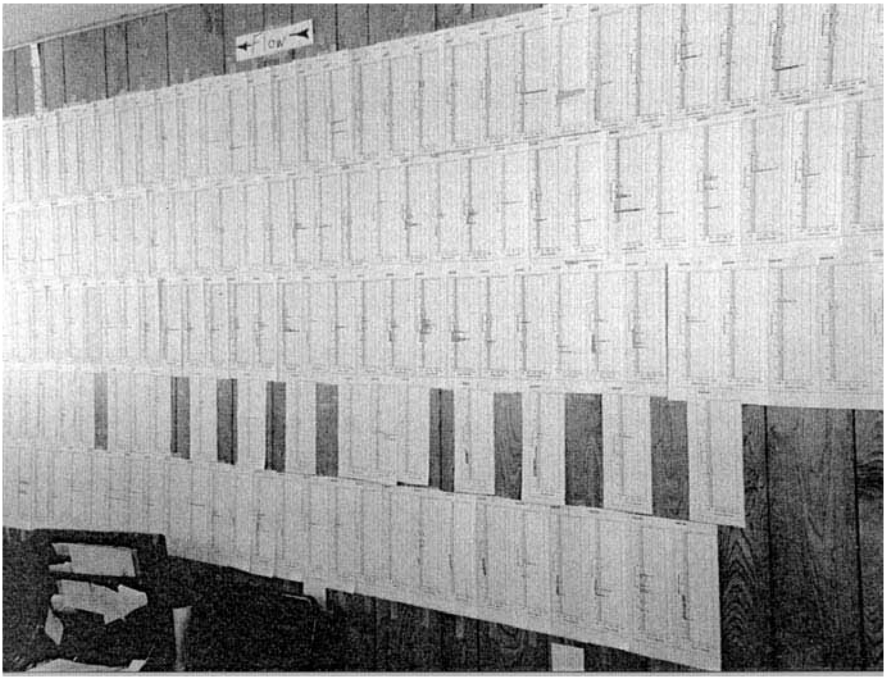

FIGURE 6B.87 Posted records facilitated analysis and evaluation.

conclusion was supported by the results of the grouting program. Therein, the majority of the grout was injected at depths greater than those of the fill and extended throughout a plan area much greater than could be reasonablly represented by typical piping leakage. A total of 4523 cubic feet (125 m^3) of grout was injected into 281 holes extending through the embankment. Careful records of grout take and behavior were kept and promptly reviewed. They confirmed that the voiding was much more extensive than originally thought and that, indeed, it was in the original soils underlying the embankment itself. Figure 6B.87 shows the records for the individual holes, assembled in a collage, on the wall of the office trailer. Thereon, the grout take is indicated by a horizontal bar at each respective depth, such that the actual configuration of the voiding can be readily visualized.

As a final note, *compaction grouting need not be messy*. Figure 6B.88 shows a grout hole casing, within inches of sensitive laboratory equipment in the Spalding Research Laboratory at the California Institute of Technology in Pasadena, California. With reasonable care (note that the equipment is wrapped in plastic sheeting), compaction grouting can be successfully conducted under even the most sensitive environments.

6B.6.3 PERMEATION GROUTING

Permeation grouting is the longest established and most widely used grouting technique. In soils, the procedure involves permeating and filling the soil pore spaces, without any significant disturbance to, or movement of, the individual soil grains. The structure and dimensions of the soil pore spaces dictate the type of grout that can be effectively used. The particles of most fluid suspension type grouts containing ordinary portland cement and are too large to penetrate any but the coarsest

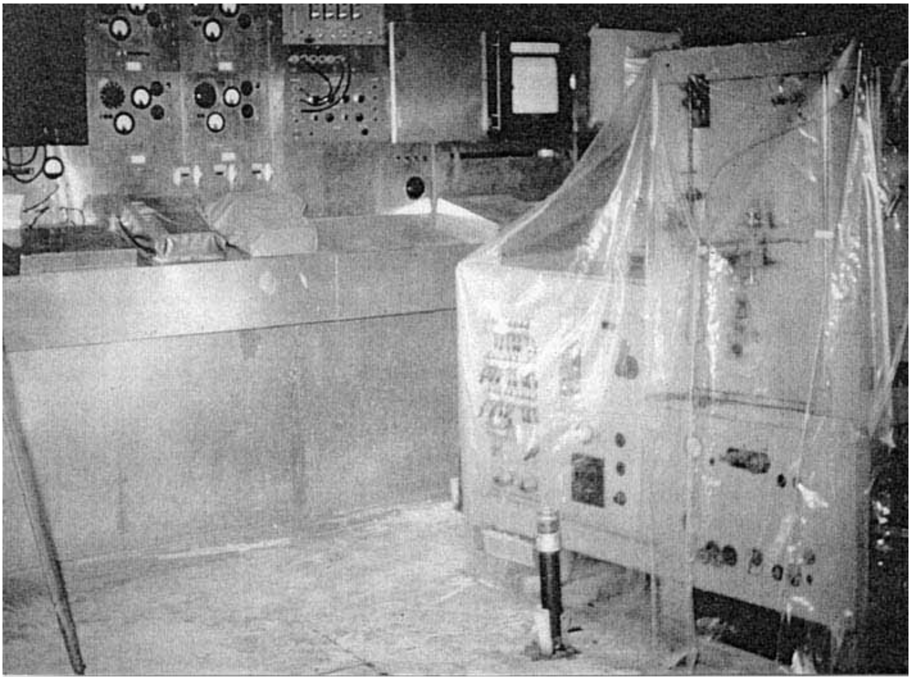

FIGURE 6B.88 Work around sensitive equipment in a research laboratory.

soil materials, limiting their use to reasonably clean coarse sands and gravels. Accordingly, permeation grouting of most soil is performed with either highly penetrable chemical solutions or ultrafine cement grouts. Outside the United States, principally in Europe, clay, combinations of clay and cement, or chemical solution grouts have also been extensively used.

This type of grouting is generally limited to sands and sandy soils containing minor amounts of finer particles. Whereas suitable grouts can be injected into fine sands and silts, slower injection rates are required than in the more permeable coarser-grained materials. The slower injection adds to both the required time for placement and cost of the work, which sometimes preclude its use. Permeation grouting of soil is generally more expensive than is compaction grouting. Its use is thus commonly limited to applications where a considerable increase in the *cohesion*, rather than the density, of the granular material is required, or in clean, coarse materials, which are not appropriate for compaction grouting.

The largest single use of permeation grouting is for temporary solidification of low-cohesion sandy soils as an aid to construction, or to reduce or eliminate soil disturbance or failure resulting from excavation. Figure 6B.89, illustrates such a case; here a self-supporting buttress of solidified soil enabled a nearly shear cut to be made for the basement of a new building. Permeation grouting is also seeing ever-increasing use for mitigation of the liquefaction potential of in situ soils as a result of earthquakes. The procedure is also occasionally used to permanently increase the load-bearing capacity of appropriate soils.

Another significant area of use for permeation grouting is for reduction of permeability or control of the flow of water through a soil formation. Both ultrafine cement and chemical solution grouts are so used. Fine fillers, such as silica fume and ultrafine cement, are sometimes mixed into solution grouts to increase their strength, alter their properties, or lessen their cost. Expanding

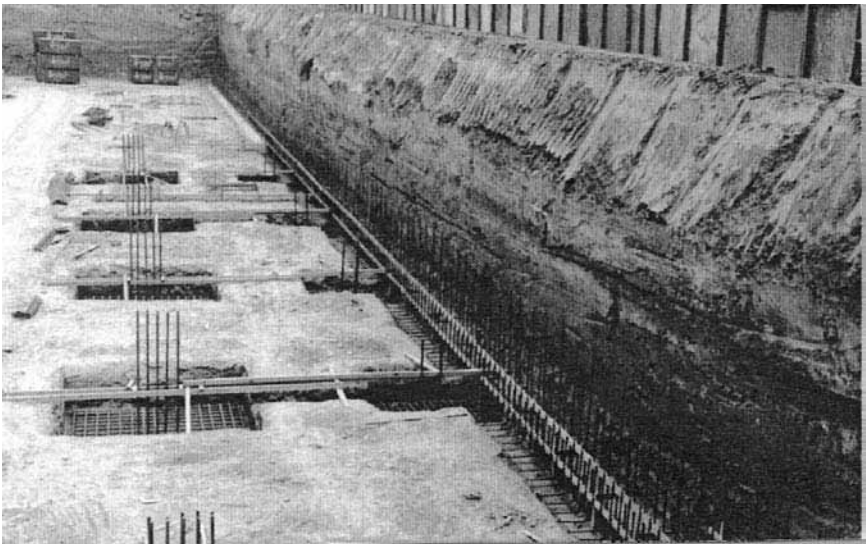

FIGURE 6B.89 Solidification of in situ soils into a self-supporting buttress negated the need for any other support of this excavation in wind-deposited, cohesionless fine sands.

knowledge, more reliable and durable strength, and increasing availability of ultrafine cements have resulted in their increased use for permeation grouting, and they are now preferred over the more traditional chemical solution grouts by many professionals.

6B.6.3.1 Fluid Suspensions for Strengthening

Fluid suspension grouts were the first grouts to ever be used and continue to be widely employed. The largest area of use for these grouts is in the filling of cracks and joints in rock, although their use has been extended greatly and includes controlled injection into soil. In the simplest form, a suspension of ordinary portland cement and water is mixed. High-shear mixing is required to thoroughly disperse the cement particles, and continuous agitation of the mixed grout is required to maintain the individual grains in suspension.

Grout mix design is expressed as the ratio of water to cementitious solids, either by weight or by volume. Volume measurements have been the most often used traditionally, as they were very easy to make when using bagged cement that contains one cubic foot of dry powder. Where bulk cement is used, proportioning by weight is more convenient. In the United States, grout mixtures as thin as 10:1 by volume have been extensively used historically. Many, including the writer, consider such thin grouts to be little more than dirty water, and extensive testing and research by the author and others (Houlsby, 1982; Weaver, 1991) have shown such thin mixtures to provide little durability. Current thinking suggests that thicker mixtures (maximum W/C 3:1 by volume) are far more appropriate for most applications. In a well-documented discussion of appropriate water to cement ratios, Houlsby (1982) concluded that W/C 2:1 by volume has been used on thousands of projects and was optimal for general use.

Although unmodified common cement–water suspensions continue to be widely used, they suffer from settlement of the solids, resulting in accumulation of mix water at the top of unagitated grout. This is known as bleed and is an important factor to consider in any grout formulation. In this regard, the amount of shear subjected during the mixing of the grout has an important bearing upon the resulting bleed. When cement and water come together, the cement grains tend to clump or floc together. To minimize the amount of bleed of a particular grout, high-shear mixing is required so as to break up and separate the individual cement grains. This factor was thoroughly discussed by Kravetz (1959). The essence of his conclusions, as reported by Houlsby, (1990) are:

1. Cement grains, when mixed with water, tend to aggregate and form clumps. This slows the wetting process, as does air attached to the grains. The effect of high-speed shearing or laminating, plus the centrifugal effect, is to thoroughly break up the clumps and to separate air bubbles. As a result, each individual grain is rapidly and thoroughly wetted and put into suspension.
2. During cement hydration, needle-like or spring-like elements of hydrates form on the superficial layer of each wetted grain of cement. In a high-speed mixer, the laminating effect and high-speed rotation keeps breaking these hydrates away from the grain of cement, thus exposing new areas to the water and consequently bringing the formation of new elements. These hydrate elements are of colloidal size, and as the amount of these elements in the mixture increases, the grout becomes colloidal in character.

In reality, virtually no common cementitious suspension grout is truly colloidal, as even well-dispersed cement grains will settle in water unless special admixtures are used, as will be subsequently discussed. High-shear mixing will greatly reduce and can in thick grouts nearly eliminate bleeding. In some places, but generally not in the United States, grout that exhibits little or no bleed is regarded as an “stable.” Stable grouts are generally defined as those that exhibit a total of no more than 5% bleed. European practice commonly calls for “stable” grouts, which typically include a few percent of bentonite, to act as a suspension agent. Whereas bentonite inclusion will tend to lessen the cement grain settlement and resulting bleed, there are significant disadvantages to its use. Modern admixture technology offers a variety of formulations that can minimize bleed without the undesirable aspects of admixed bentonite. This will be subsequently discussed in more detail.

Suspension grouts of ultrafine cement and water are significantly more penetrating than those made with Type 3 cement, which is the finest of the common cements, and some ultrafines can form nearly colloidal mixes. These can be readily injected into sandy soils, as illustrated in Figure 6B.90. Because of the minute grain size of ultrafine cements, they provide very large specific surface areas. Inclusion of a high-range, water-reducing admixture in these grouts is virtually required, and will result in much better dispersal of the individual cement particles. This will substantially reduce the propensity for pressure filtration in the soil. It will also dramatically increase the specific surface area of the cement available for the water to contact, resulting in much increased hydration activity, and higher ultimate strength.

There are two main types of ultrafine cement—those based on slag and those based on portland cement. Although the two are comparable in many respects, there is a fundamental difference in the span of hydration and the resulting setting time. Because of the very high specific surface area of ultrafine cement based on portland cement, hydration activity is very high, resulting in rapid setting and strength gain. For this reason, retarding admixtures are sometimes included in the cement to delay the otherwise rapid setting. Portland cement combined with some common retarders can be extremely sensitive to temperature, so the proportions of the additive must be accurately dosed and appropriately matched to the temperature of both the injection environment and the formation being grouted. Problems with flash set and difficulties with both setting time and rate of strength gain have been reported when using some of the earlier ultrafine cements based on portland cement.

Cements based on slag tend to set and gain strength much slower than their portland cement cousins. Thus, retardation is seldom a problem, but accelerating admixtures may be required in some instances. Well-established accelerators are available and their behavior is quite predictable, so time of setting is not much of a problem with slag-based ultrafines. It is only in recent times that ultrafine cements have become commonly available. With their ever-increasing use and accumulated field experience, continuous improvement in the available products is occurring, and continual monitoring is required to keep up with the latest technology.

Most ultrafine cements are composed of a range of differently sized particles. Providing they are

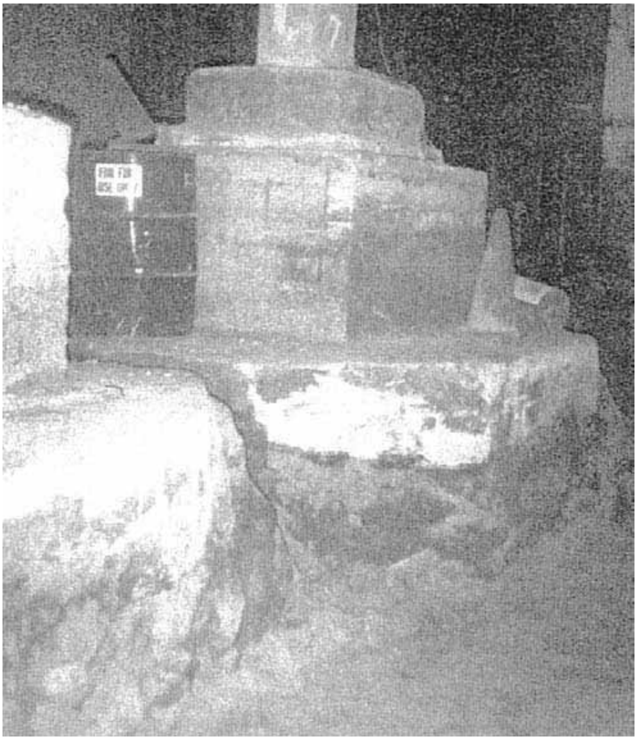

FIGURE 6B.90 Ultrafine cement grout used to solidify cohesionless sand allows excavation under the foundations of a structure.

well dispersed, some larger particles are acceptable for grouts that will be used to fill obvious fractures and voids, as long as the largest particles are smaller than the thickness of the defect. In the permeation of granular soil, however, because the soil itself acts as a filter, the larger-sized particles of the grout can build up as they are pushed into the soil, initiating the formation of a filter cake. The size, and the proportion, of the larger grains of ultrafine cement used for permeation grouting of soil is thus of great importance. Obviously, use of a cement with the smallest mean particle size and a minimal amount of larger particles is desirable for soil grouting. The particle size distribution of several different ultrafine cements as well as Type III common cement are shown in Figure 6B.91.

It should be noted that not all type III cement has the same grain size distribution. ASTM C 150 “Specification for Portland Cement” calls only for Blaine fineness of the specific surface area. A compliant Blaine value can be obtained with a fairly wide range of grain size distribution. The Type III curve provided in Figure 6B.91 is for an actual batch of cement produced at one given plant.

6B.6.3.2 Chemical Solution Grouts for Strengthening

Another form of strengthening is adhesion of the individual soil grains with a chemical solution grout. There are a variety of chemical solution grouts available, but they all work in essentially the same manner. A base material, which usually represents the majority of the final mix and is often diluted with water, is combined with one or more reactant materials. The base part is usually fluid, whereas the reactants can be either liquid or powder depending upon the individual grout formula-

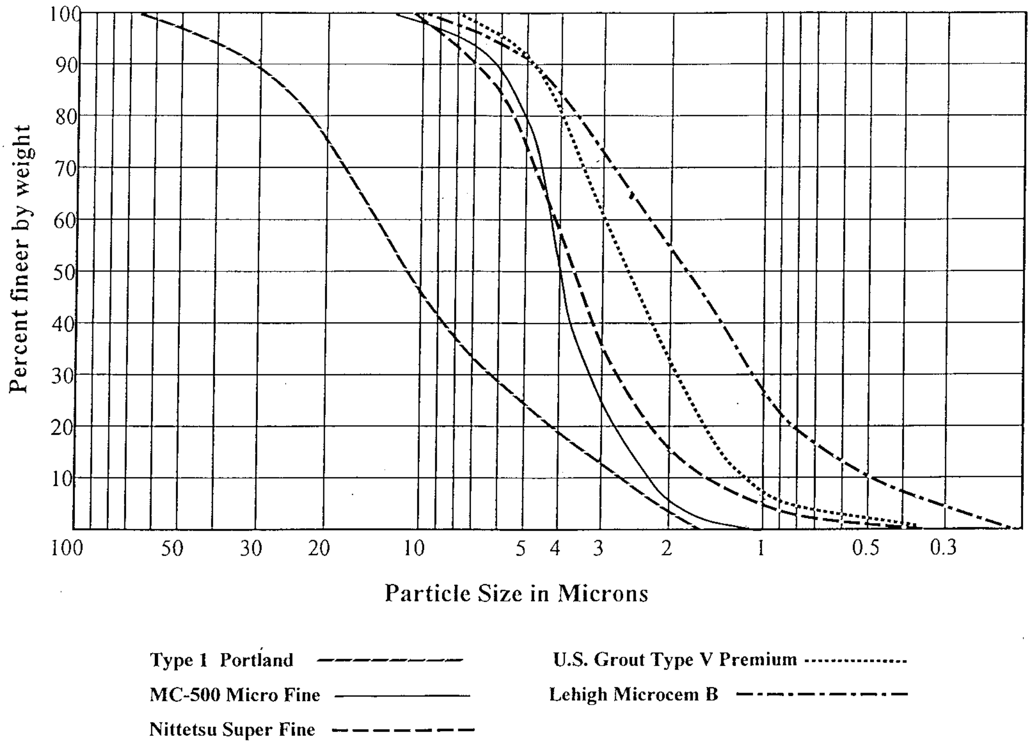

FIGURE 6B.91 Grain size distribution for various cements.

tion. At some period of time, after mixing the base and reactants together, setting occurs and the injected solution turns either to a foam, gel, or solid state. The setting time can be instantaneous to several hours or even days, depending upon the individual system used and the proportion of the individual ingredients thereof.

Many chemical grouts are mixed and pumped as single solutions, whereas the different components of others are individually pumped and stream mixed at the injection point (Figure 6B.92). There are many manufacturers of proprietary chemical grout formulations, especially those used for control of water movement within the soil or into underground substructures. For strengthening applications, especially where large quantities of grout are required, most specialty contractors tend to purchase the individual chemical components separately and mix them on the job site near the point of injection. Although some specialists represent their formulations as something very special and "proprietary" (a word greatly overused, in the writers opinion), the vast majority of grouts used for strengthening of soil throughout the world involve sodium silicate as the base material. There are, however, a wide variety of different reactant systems, most of which are well known and readily available.

For the strengthening of soils, the most important parameters for selection of a chemical solution grout are injectability, strength, safety of use, and cost. Hard, rigid gels are desired, as they provide the greatest stiffness as well as providing good strength. Cost can also be an important factor because this type of work is often performed on a massive basis, where very large quantities of grout are involved. With these factors in mind, grouts based on sodium silicate are particularly suitable and most frequently used for strengthening applications. Appropriately, the technical literature is

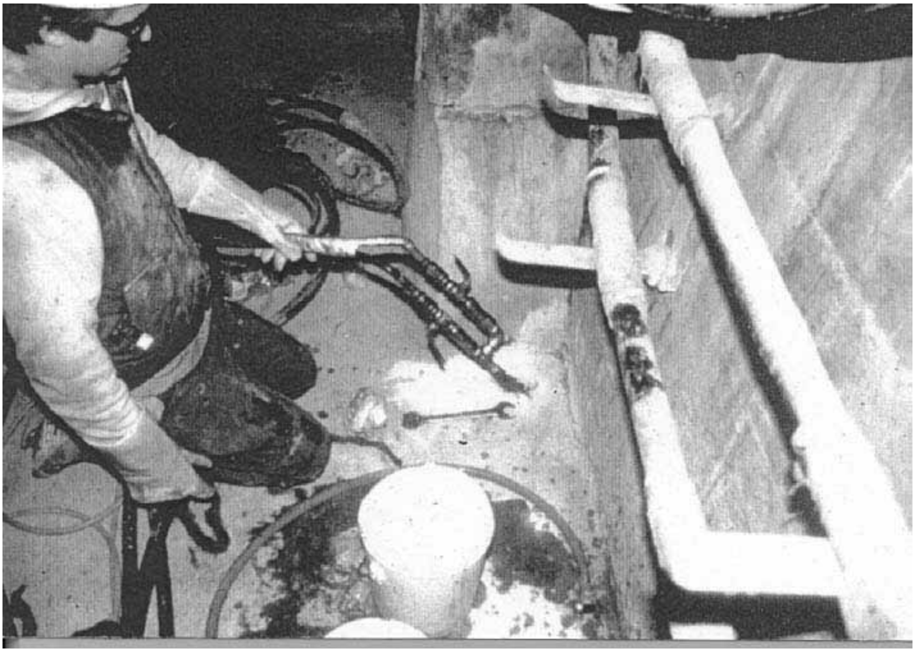

FIGURE 6B.92 Two-component water-control grout being stream mixed at the injection point.

filled with publications relative to the use of sodium silicate base grouts. Unfortunately, much of it is quite academic, and some downright misleading.

As an example, several publications present strength data for "sodium silicate grouts" but make no reference whatever to the reactant system. While sodium silicate may represent the largest component of a chemical solution grout, it requires a reactant component to harden. There are many different reactant systems, which may consist of one or more components, as will be shortly discussed, and the properties of the resulting grouts will vary greatly with the different systems. The combination of sodium silicate with some reactant systems results in a grout that will break down and lose strength with time, whereas other mix combinations have proven to be durable over long periods of time. The only available guide for the selection of chemical solution grouts for strengthening soil is ASTM D 4219, "Standard Test Method for Unconfined Compressive Strength Index of Chemical-Grouted Soils." The preamble of this document states that it applies only to the "short-term" strength of the grouted mass. No time definition of short-term is given, but at best, grouts conforming to the standard should be considered for temporary strengthening only.

Graf, Clough, and Warner (1982) reported on the long-term aging effect of chemically grouted soil specimens that were aged from 9 to 11 years. They concluded that "Except for the very weakest samples, the strengths of the stabilized soils tested with no environmental change, showed no change or a modest increase over those measured after one to two years aging." The specimens on which they based their conclusions were composed of either 50% or 60% sodium silicate and reactant systems that are still available but not frequently used today. The reactant systems most frequently now used with sodium silicate, consist of dibasic or tribasic esters. There are no long-term data available as to the permanence of grout made with these reactants.

The strength of the sodium silicate concentration will also affect the final properties of the grout, including long-term durability. Some publications fail to recognize this crucial factor or in some cases report incorrectly that sufficiently strong solutions either cannot be injected or cannot obtain a satisfactory hardening time with a given reactant system. Bad results have been experienced in several instances where strongly diluted solutions were used. This notwithstanding, such poor results are virtually always the result of faulty mix design, and not indicative of the behavior of properly designed sodium silicate formulations. There are many factors that influence the long-term strength and durability of chemical solution grouted masses and these will be dealt with subsequently.

6B.6.3.3 Grouts for Water Control

Historically, cementitious suspension grouts have been used to stop the flow of water through the cracks and joints of rock. The pore size of most soils, however, is significantly smaller than the width of typical rock joints, and except in the case of coarse sands and gravels, it is insufficient for intrusion of a common cementitious, particulate grout. Thus, water control in soils is almost always accomplished with chemical solution grouts or suspensions based on ultrafine cement.

Whereas hard, rigid chemical gels are preferable for strengthening applications, some flexibility is usually desirable in water control work, and thus different chemical grout systems are in order. Dilution of the grout with groundwater is always a concern, and where rapid flow of the groundwater occurs, the grout must be resistant to being washed away as well. This usually means use of a rapidly setting grout. A variety of materials that can provide instantaneous setting are widely available. These grouts are usually more chemically complex than those used for strengthening of soils, and some formulations present health risks and require special handling. They are thus usually obtained as proprietary systems from specialty grout producers.

With the increasing availability of ultrafine cements, many of which are able to readily penetrate the pores of even the finest sands, suspension grouts of these cements are increasingly being used for soil solidification. They usually result in a stronger solidified mass, and are often less costly than chemical solution grouts. Also, unlike many chemical solution grouts, they do not lose strength over time.

6B.6.3.4 Grout Injection

Permeation grouts can be injected from the top down, bottom up, or selectively at any depths or locations desired, through the use of sleeve port pipes. The proper sequence of injection is dependent upon the relative permeability of the different strata of the particular formation being treated. A common mistake, which has resulted in poor performance on many projects, is to inject upstage in a deposit that is significantly more permeable at depth than at the higher elevations. In such cases, the grout being injected, even at the higher levels, will tend to pipe down into the more permeable zones. For uniform permeation of soil, it is important to inject the proper quantity of grout in each stage of each hole. Except in the case of sleeve port injection, this usually means injection of the zones of lowest permeability first.

6B.6.3.5 Hole Layout

Grout holes are generally placed at regular spacing of about two to six feet. In rare instances, holes might be as close as about 0.5 feet where very low permeability soils are to be penetrated, or where very precise injection is required. Where the thoroughness of the grout saturation is of little importance, greater spacing might be used. When injecting ultrafine cement suspensions, filtration of the grout particles can occur, especially in finer-grained soils. Thus, closer hole spacing might be required. Where several rows of holes are used over a large area, they are usually staggered in alternate rows, as shown in the top of Figure 6B.93. Alternate *primary* holes should be injected first. Once the primary (P) holes have been injected, grout can be placed in the secondary (S) holes. If the optimal spacing has been used, the secondary holes should require slightly less grout than the primary. In those cases where extremely thorough grout saturation is required, a pattern as shown on the bottom of Figure 6B.93 can be used. Here the tertiary (T) holes are grouted after the adjacent primary and secondary holes have been completed.

In water control grouting, a reduction of the water flow is an indication that the grout is being placed in an effective manner. In strengthening applications, however, the only assurance of com-

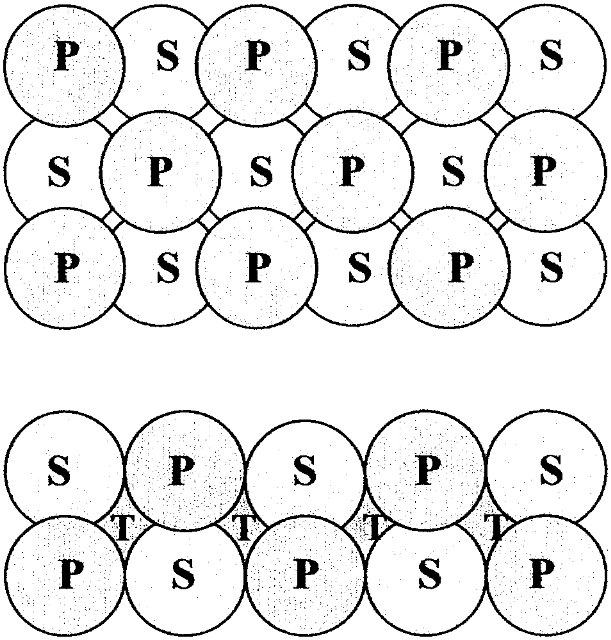

FIGURE 6B.93 Typical grout hole layout. P = Primary, S = Secondary, T = Tertiary.

plete saturation of the intended soil is accurate preinjection appraisal of the soil's porosity, and precise placement of the proper quantity of grout at each intended location. In this regard, continuous observance of the injection pressure behavior is mandatory, as any sudden drops of pressure are indicative of the occurrence of hydraulic fracturing. Obviously, grout that is allowed to follow fractures will not solidify the intended soils. Excessive hydraulic fracturing is usually indicative of an excessively high injection rate.

6B.6.3.6 Soil Strengthening Examples

Permeation grouting of soil has been successfully used on a wide range of different applications. These have included both temporary as well as permanent solidification of a variety of sands and sandy soils, as well as shutoff of water leakage, both into substructure and through the soil. Perhaps the largest single area of permeation grouting in construction is increasing the cohesion of in situ soils prior to the excavation of soft ground tunnels. On a worldwide basis, huge quantities of grout have been used in such work. The most common objective is to prevent soil runs during tunneling, as well as to improve support for any structures lying adjacent to or over the tunnel alignment. The most commonly used materials for this type of work have been sodium silicate based chemical solution grouts. As the ground improvement need last only long enough for the construction to take place, long-term durability of the grouted formation is usually not a concern. This type of work can be performed either from the tunnel heading using fan shaped arrays of grout pipes, as shown in Figure 6B.94, or from the surface, as shown in Figure 6B.95.

Where practical, working from the surface is preferred, as it does not interfere with the tunnel excavation and can be done well ahead of the tunnel advance. Although some cohesion is imparted

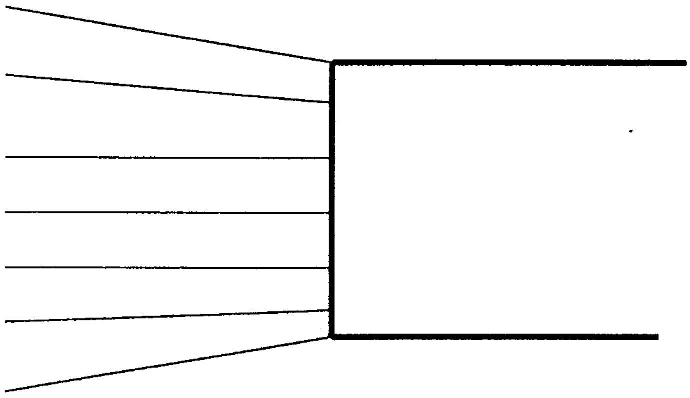

FIGURE 6B.94 Grout holes in fan array.

to the soil within hours of injection, substantial strength does not occur for a minimum of several days. Thus, a time lag is required between grout injection and tunnel excavation. In some instances however, injection from the surface might not be possible, due to excessive tunnel depth or limitations of surface access. In such cases, where work must proceed from the tunnel heading, selection of a grout system that rapidly gains strength is essential.

Permeation grouting for temporary support during construction has also been used on a wide variety of different types of excavation and structures. As part of the expansion of Los Angeles International Airport, it was necessary to underpin an existing concrete retaining wall, which was found-

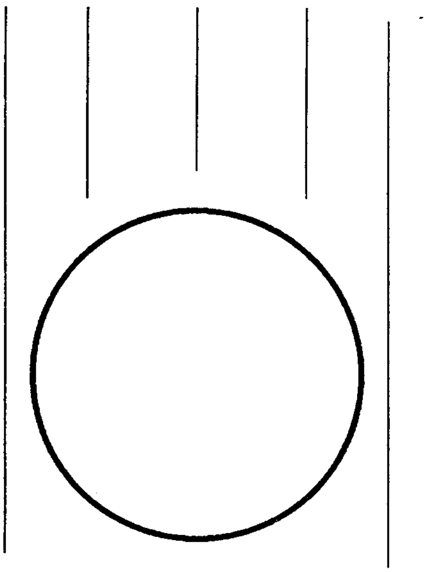

FIGURE 6B.95 Grout holes from surface.

ed on wind-deposited fine dune sand. The wall was to be undermined and extended by a depth of 18 feet. A sodium silicate based grout was injected from the top down using a special injection needle, which was advanced in one foot increments (Figure 6B.96). Two rows of holes, were used, as indicated on Figure 6B.97. The lower portion of the grouted mass was injected from a row of holes inclined about 10 degrees from vertical, and the upper portion from holes inclined at about 20 degrees. The quantities of grout injected at each level was adjusted so as to provide the required lateral saturation needed, considering the inclination of the injection. The existing sands were improved to an unconfined compressive strength of about 150 psi. Excavation, which proceeded about two weeks after the injection, required pneumatic tools, due to the strength of the mass.

The rehabilitation of a side-hill structure supporting a road required the excavation of an immediately adjacent vertical face. Because normal support was not possible, due to the existing downslope, permeation grouting was performed to solidify the existing soil under the structure. In order to make the shear vertical cut, it was necessary to actually remove a portion of the footings of the structures supporting pillars, as can be observed in Figure 6B.98.

A sodium silicate based permeation grout was injected into the wind-deposited fine sand. This was accomplished through a single row of grout holes 15 inches behind the face of the structure. The holes were spaced 30 inches apart. A quantity of grout that would solidify a theoretical 36 inch diameter column based on the soil's porosity was injected into each hole. Holes were injected on an alternate primary/secondary split spacing order. Because the sand became less permeable with depth, the grout injection pipes were driven to the final depth (Figure 6B.99) prior to injection,

FIGURE 6B.96 Grout injection from the top down with special drive needles.

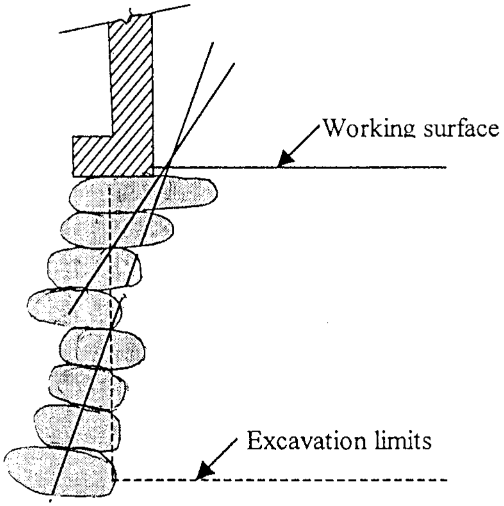

FIGURE 6B.97 Two rows of inclined injection probes were used.

FIGURE 6B.98 Solidified mass allowed excavation of shear vertical cut, including removal of part of the supporting concrete footings.

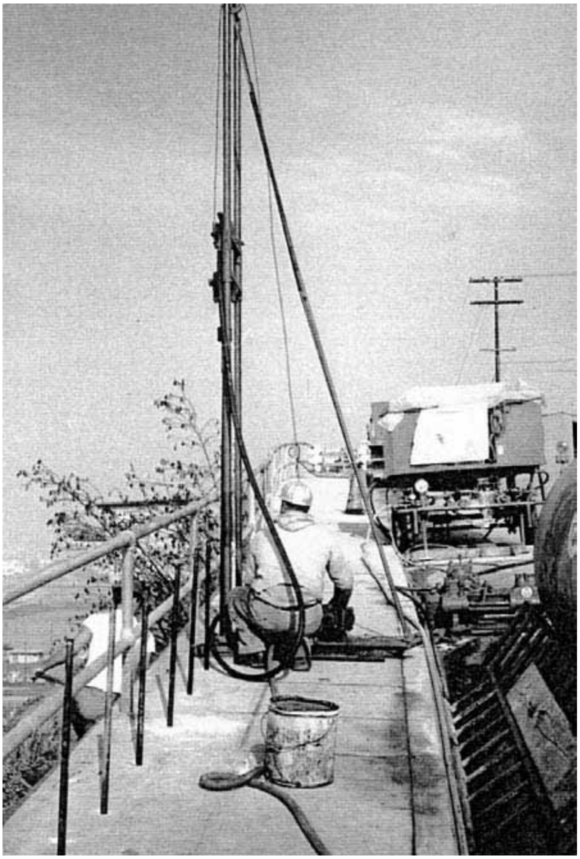

FIGURE 6B.99 Special needles were driven to the depth of injection, which proceeded from the bottom, up.

which was then made in one foot stages, from the bottom, up. As can be seen in Figure 6B.98, pneumatic tools were required to trim back the solidified mass to the specified alignment. The flush cut was 240 feet long and varied in height from 6 to 24 feet, as shown in Figure 6B.100. For those sections which exceeded a cut height of 12 feet, a second row of holes was injected 27 inches behind the cut face, so as to provide a stabilized wall approximately five feet thick.

The foundation for new equipment in a steel mill required excavating to within four feet of the shallow concrete pad foundation of a column, which supported a large traveling crane. In spite of the dynamic forces that would be transferred into the column during operation, the crane was required to be in service 24 hours a day, seven days a week. The foundation was underlain with cohesionless sand and gravel, into which the required excavation would extend some 11 feet.

The entire area under the footing was solidified with a chemical solution grout, which was injected from the top down. This was done with hand-jetted, open-end ¾ inch pipe injectors (Figure 6B.101). Injection probes were on a grid of 24 inches, and a quantity of grout sufficient to create an equivalent 30 inch column was placed. The nearly vertical excavation was then made (Figure 6B.102) with continual operation of the crane. Due to the dynamic forces exerted into the footing by the passing crane, it was important to provide maximum possible stiffness, as well as

FIGURE 6B.100 Vertical cut face varied from 6 feet to 24 feet high and extended for 240 feet.

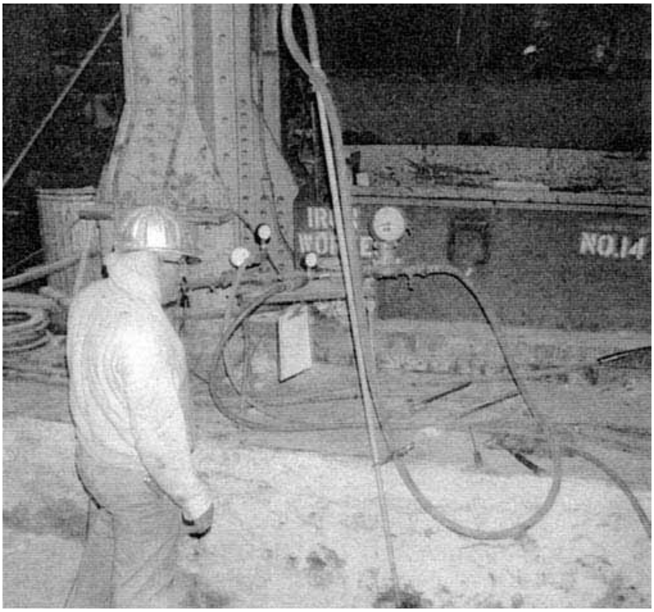

FIGURE 6B.101 Grout injection in progress through standard ¾ inch schedule 40 pipe.

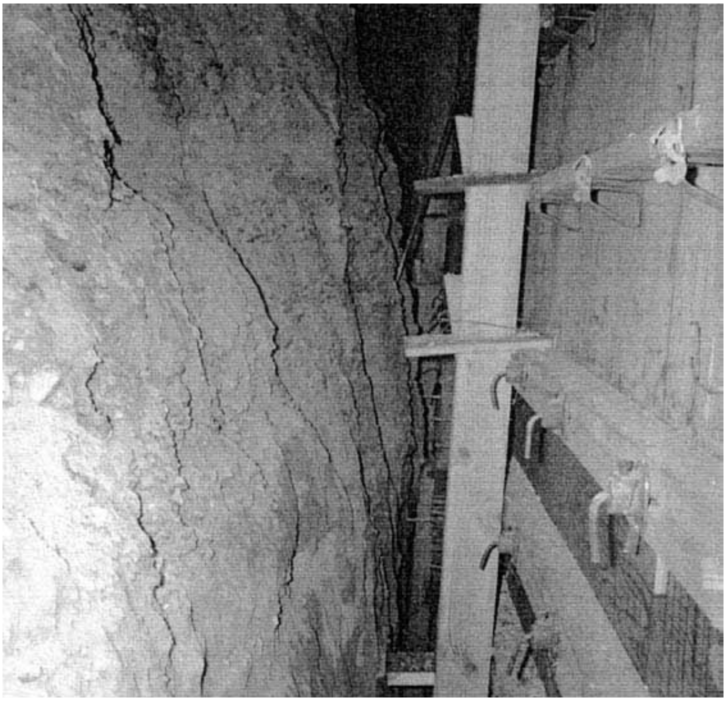

FIGURE 6B.102 Formwork for the new foundation, adjacent to near vertical cut of cohesionless soil.

strength to the underlying soil. This would have been an excellent candidate to be grouted with ultra-fine cement grout; however, that material was not readily available at the time the work was performed.

Seismic retrofit of the San Francisco Civic Center required undermining of existing foundations by as much as nine feet. The foundation soils are basically round-grained beach sands, with virtually no cohesion. The ground, which was to remain under the foundation elements, was solidified with an ultrafine cement grout. As can be seen in Figure 6B.103, this allowed for otherwise unsupported excavation, as required for the new extended footings. In order to maximize the penetrability of the grout, it was mixed in a high shear colloidal mixer (Figure 6B.104).

When the terminal building at the Kansas City International Airport was originally built, the excavated space behind the basement retaining walls was backfilled with a granular fill sand. A contractor building four access tunnels connecting to the building in 1997 encountered this sand fill. It caved into his excavation, exposing utilities and undermining adjacent pavement. To mitigate the problem, ultrafine cement grout was used to solidify the cut face of the required excavations. The grout was injected from the top down, through jet pipes that were marked at one foot intervals. The pipes were positioned at a 15 degree angle toward the proposed excavation. They were jetted with grout into the formation, with a pause at each foot interval, as required to place the predetermined amount of grout (Figure 6B.105).

For most of the work, the holes were placed on three foot centers. The calculated amount of grout to form a four foot column was injected. Where the excavation was greater than 10 feet deep,

FIGURE 6B.103 Nine foot vertical cut under foundation in cohesionless beach sand.

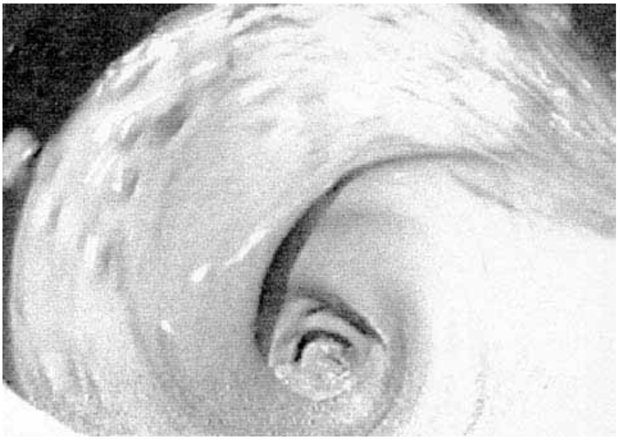

FIGURE 6B.104 Ultrafine cement grout was mixed in high-shear mixer. Note vortex in grout.

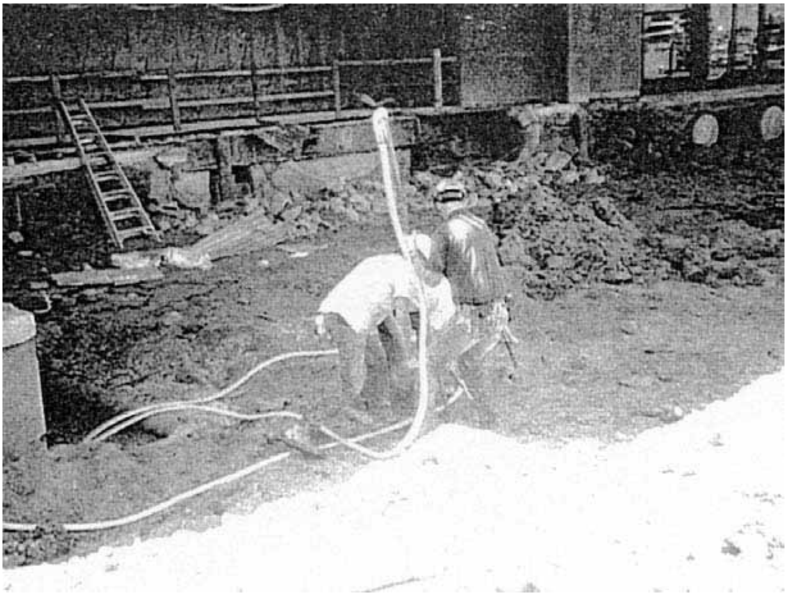

FIGURE 6B.105 Jetting grout into the sand backfill.

a second row of holes was provided, split-spaced and three feet behind the initial row. The grout was composed of:

One bag Nittetsu Superfine cement

3.46 cubic feet (26 gallons) water

0.44 pounds high-range water reducer

It was mixed in a high-shear colloidal mixer and transferred into an agitator tank, which supplied a 2L8 Moyno pump. The pump was set for an injection rate of 2.6 cubic feet (20 gallons) of grout per minute.

Upon excavation, the sand fell freely from the injected columnar shaped masses (Figure 6B.106). They withstood the elements, including several cycles of freeze and thaw, for a period of several months. Both vibration and impact forces were created during demolition of the adjacent concrete wall but had no apparent effect on the solidified mass.

To save time and money, NASA used permeation grouted piles rather than conventional piling for the foundation of a 350 ton missile launch gantry at Vandenberg Air Force Base in California. A tight schedule for launch of a Physics Interplanetary Monitoring Platform satellite into orbit required very rapid construction of the launch facility. There simply was not sufficient time to construct conventional piling. Studies found the underlying soil to consist of about 20 feet (6.09 m) of fine to medium dune sand, underlain by shale bedrock. Groundwater was present, at a depth of about 15 feet 4.57 m).

The gantry traveled on sets of rails mounted to two parallel pile caps. The heavily reinforced concrete caps were supported on two rows of piles, as shown in Figures 6B.107 and 6B.108. The pile design was based on a permanent, chemically solidified mass, with an unconfined compressive strength of at least 100 psi (689.5 kPa). This was considered conservative, as strengths nearly twice that value were found to exist after seven years in a nearby installation, which used the same grout formulation. Accordingly, the 188 piles were designed, with a minimum diameter of 30 inches (0.76

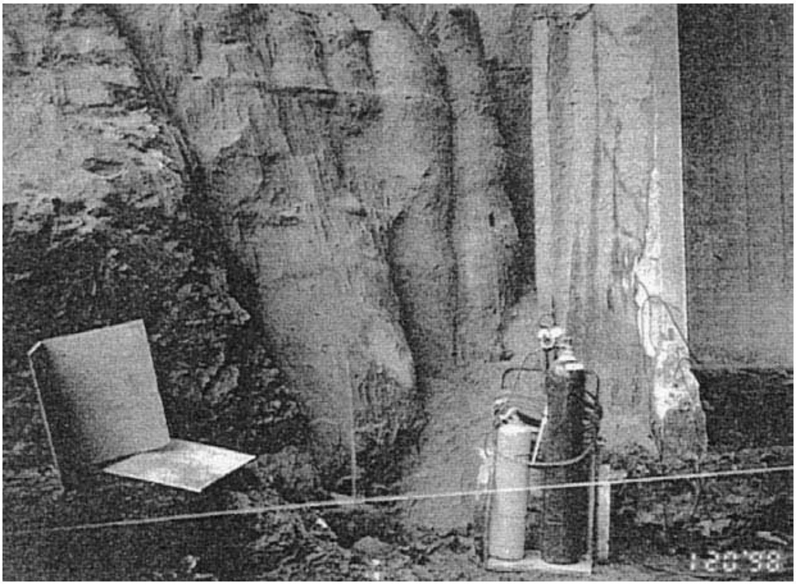

FIGURE 6B.106 Self-supporting solidified sands.

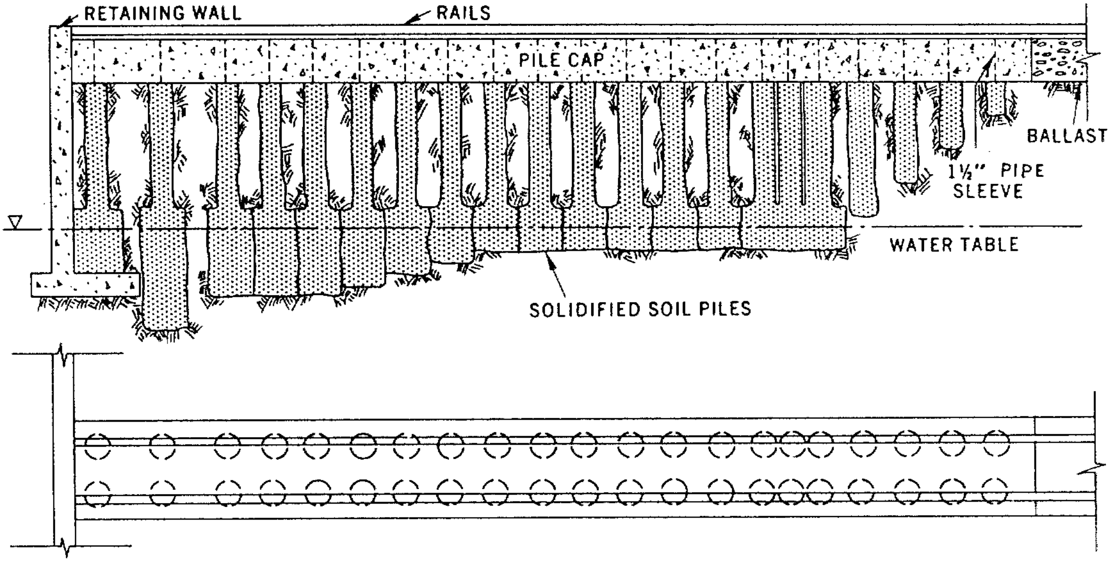

FIGURE 6B.107 Two rows of piles supported each parallel cap.

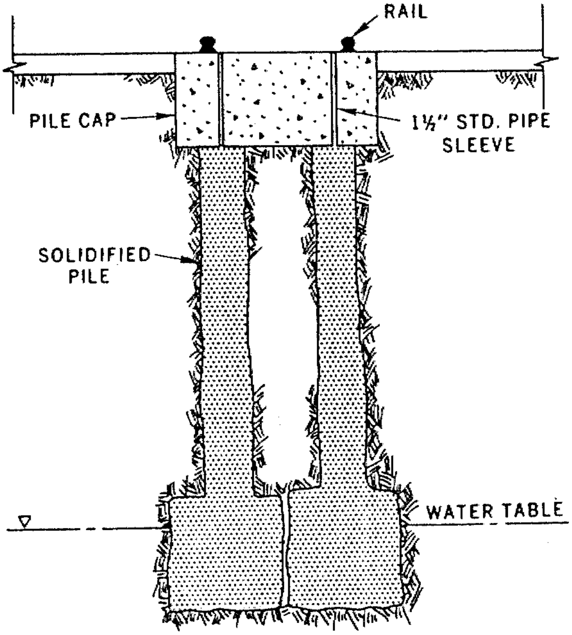

FIGURE 6B.108 Cross section of pile cap and solidified piles.

m). The diameter was increased to 60 inches (1.52 m) for the portion below the water table, as chemically grouted masses are known to obtain lower strengths under water. The piles extended full depth to the underlying shale formational deposit.

One distinct advantage of injected piles for the project was the ability to form them either before or after the pile cap had been cast. This greatly reduced the total time required for the work. Initially, piles were injected prior to casting of one of the parallel concrete pile caps, while the other cap was being simultaneously constructed. Sleeves of 1½ inch (3.8 cm) pipe, as shown in Figure 6B.108, were cast into the second cap at each pile location. The remaining piles were then injected under the completed pile cap, using the pipe sleeves for access.

A three-component, sodium silicate based chemical solution grout was prepared in batch mixing tanks (Figure 6B.109). The base tank combined sodium silicate, water, and a surfactant. The reactant tank combined ethyl acetate and calcium chloride reactants, with water. These were accurately proportioned by way of properly calibrated metering pumps, which fed a "flash mixer" located immediately ahead of the main grout pump. The flash mixer was simply a three inch squirrel cage blower enclosed in a small tank, turning at about 1800 rpm, so as to provide very high shear mixing. The final grout mixture contained 55% sodium silicate.

Injection was from the top down, through special drive needles (Figure 6B.110). The needles were advanced in one foot increments, followed by injection of the grout. A calculated amount of grout, sufficient to completely saturate a 36 inch column, was injected for the 30 inch sections, and 66 inches (1.67 m) for the 60 inch (1.52 m) sections. Tests carved from the solidified masses, both during the work and after completion, attained fundamental strengths somewhat greater than 200 psi.

A large, horizontal piston-type air compressor was installed on a massive reinforced concrete foundation. After several years of operation, excessive vibrations resulted, which not only affected the foundation block, but the surrounding building elements as well. Investigation revealed that the

FIGURE 6B.109 Grout mixing and pumping plant.

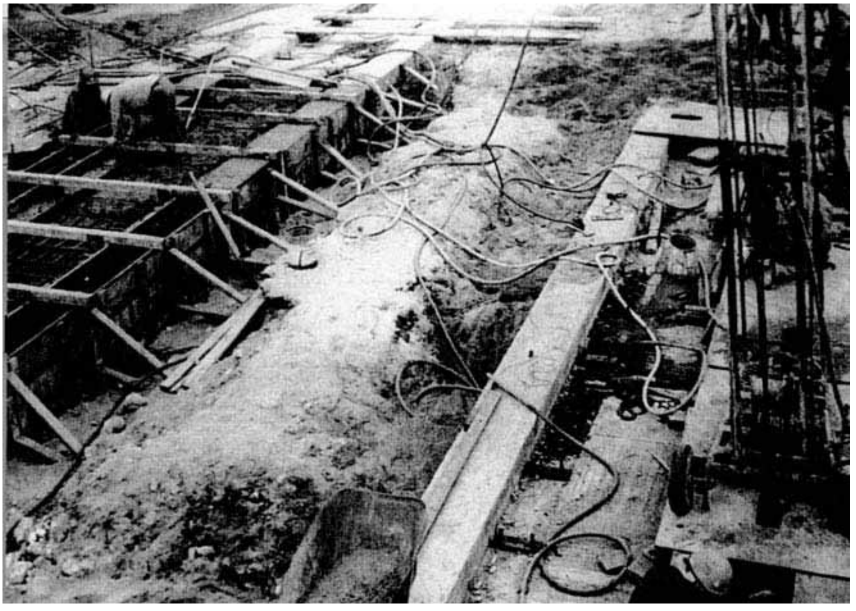

FIGURE 6B.110 Special rig drives injection needles through pipe sleeves in the completed pile cap.

foundation block was founded within a deposit of silty fine sand. The sand had a natural frequency, which was in harmony with that of the compressor. It was concluded that an increase of the stiffness of the sandy soil was required to eliminate the excessive vibrations.

Accordingly, a grouting program was formulated with the intent of stiffening the sand through solidification with a chemical solution grout. The chosen formulation contained a 70% solution of sodium silicate. The penetrability of a grout containing such a high concentration of the base component is significantly less than that of more conventional formulations; however, it was used for this application, in order to provide the greatest degree of additional stiffness practicable. With a grout of such low penetrability, very slow pumping rates are necessary, and it is desirable to space the grout holes closely. The machine resulted in severe restrictions of space for the layout of grout holes. A single row was thus placed on 1.5 foot (.46 m) centers, immediately adjacent to and around the periphery of the foundation (Figure 6B.111). Additional holes were placed within this boundary containment, as allowed by the existing conditions. The grout was injected at an unusually slow rate of less than one gallon per minute, as shown in Figure 6B.112.

Although the cost of injection at such low injection rates can be high, there are situations such as this where they are warranted, and the resulting benefits are very cost effective. The grouting was completely successful, in that the excessive movements of the machine were completely eliminated.

6B.6.3.7 Permeation Grouting for Water Control

The procedures used to create an impermeable barrier in soil are essentially the same as those used to inject grout for strengthening applications. Whereas the grouted mass cannot be seen as in those cases where the adjacent soil has been excavated, the degree to which the objectives have been achieved will usually be obvious by way of reduction of the leakage. This is usually observed in the

FIGURE 6B.111 The machine presented a severe restriction to hole location and drilling.

FIGURE 6B.112 Grout injection rate was very slow due to low grout penetrability.

drainage system, if existing, or from pump drawdown tests conducted in drainage holes placed downstream of the curtain.

It is extremely difficult to effect an impermeable barrier with a single row of grout holes. Accordingly, two or more rows of holes should be placed with split spacing, as illustrated in the top of Figure 6B.93. Where an especially tight barrier is required, hole layout with tertiary injections, as shown on the bottom of Figure 6B.93, can be used. In such cases, a quantity of grout that exceeds the theoretical quantity required should be placed, unless significantly high injection pressure exists, which would indicate that all available voids have been filled.

Stoppage of water leakage adjacent to a solid object such as the wall of a structure is somewhat easier in that one boundary of the work is established. This can usually be accomplished with a single row of grout holes placed adjacent to the wall. An alternative would be to drill holes through the wall, on a regular spacing, followed by grout injection. Such work is best done during periods of active leakage, such that the effectiveness can be observed by way of a reduction of the leakage.

Water was penetrating the concrete masonry unit wall of a subsurface equipment room (Figure 6B.113). A hydrophobic urethane grout was injected in a single line of holes, at 24 inch (0.6 m) intervals, and about six inches (15 cm) outside the wall. Injection was started in those areas that had the greatest leakage. As can be seen in Figure 6B.114, some of the grout penetrated the areas of leakage, reacting on the wall surface. This gave confirmation that the culprit leakage paths were being plugged, although it also created a nasty cleanup job.

In another instance, a seal around a pipe penetrating a concrete wall had failed, allowing water to leak into subsurface space (Figure 6B.115). A hydrophilic urethane grout was injected into a hole adjacent to the penetration, and extending to the back of the wall. As can be seen in Figure 6B.116, the grout, which was formulated to cure as a somewhat flexible foam in order to allow differential

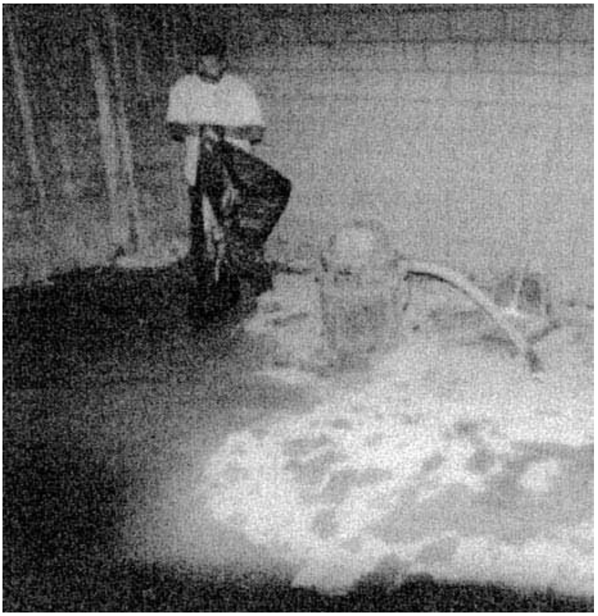

FIGURE 6B.113 Water leakage into equipment room.

FIGURE 6B.114 Reacted grout on wall.

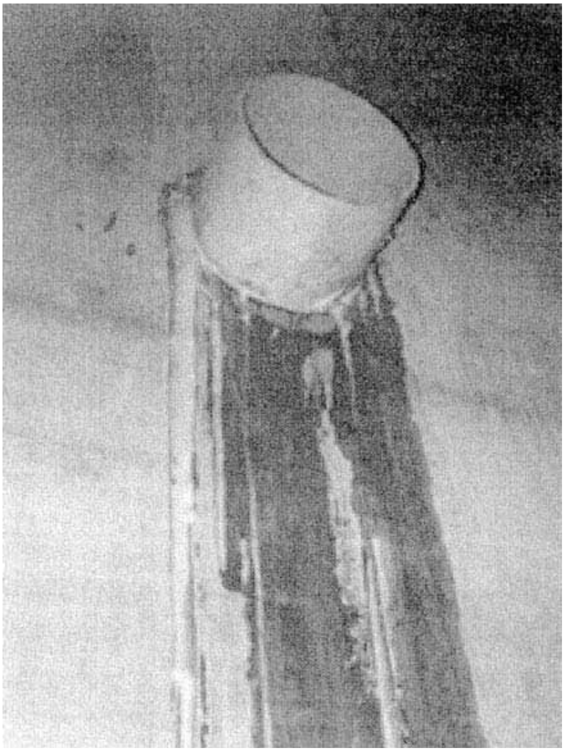

FIGURE 6B.115 Water leakage through defective pipe seal.

movement, completely filled the annulus around the pipe. A small amount of grout also ran down the wall, reacting thereon. Whereas such deposits of grout can be very hard to clean up, they do give assurance that the leakage has been completely stopped.

Excessive water was flowing into a gallery of an embankment dam. The water was under a positive head of about 300 feet (91.4 m) at the gallery level. Both hydrophobic and hydrophilic urethane grouts were injected into the zone behind the leaks. Holes were drilled through the concrete section, packers installed, and the grout injected. Because the exact source of the leakage was uncertain, grout holes were angled in several different directions (Figure 6B.117). The injection was carefully monitored, and grout returns, or diluted returns, carefully recorded. The leakage was reduced by more than 95%, although some minor seepage continued. One advantage of grouting for water control is that more holes can always be drilled and further grout injected until a satisfactory amount of leakage reduction is achieved. As this work site was located far into the gallery, it is fortunate that neither large equipment nor great quantities of material were required. Figure 6B.118 shows the small pneumatically powered proportioning pumps that were used.

6B.6.4 JET GROUTING

Jet grouting involves erosion of the soil with a high-pressure jet of grout, water, or air-enshrouded water and the simultaneous injection of grout into the disturbed soil by means of a rotating drill

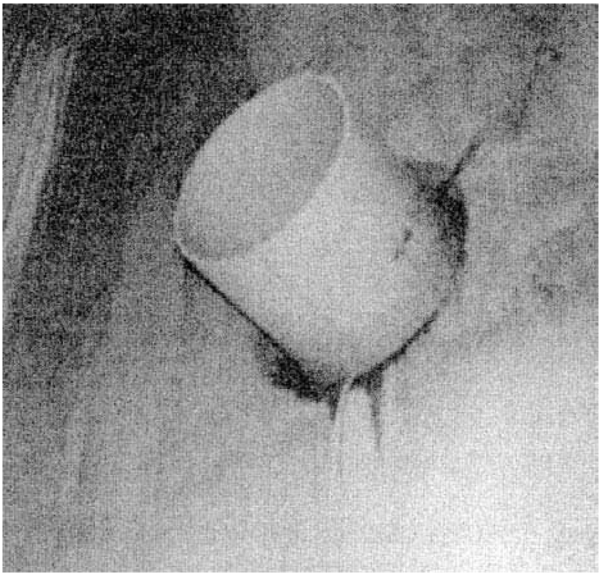

FIGURE 6B.116 Leakage completely stopped.

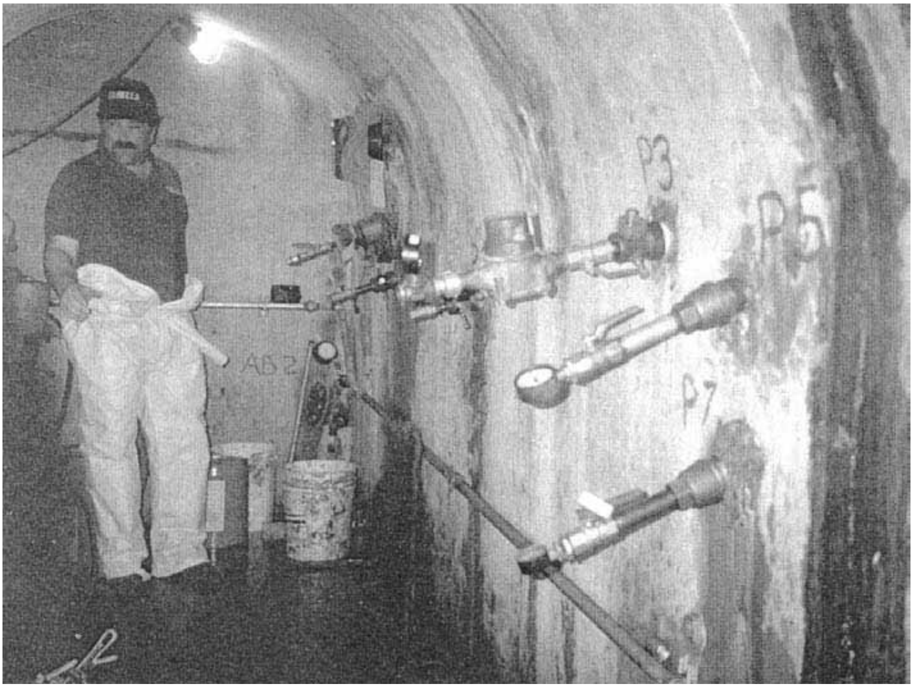

FIGURE 6B.117 Grout holes at various angles so as to affect a large area of leakage potential.

FIGURE 6B.118 Small pneumatically powered grout pump could be carried to injection location.

stem and special jet *monitor* (Figure 6B.119). The drill stem and monitor are simultaneously raised and rotated so as to mix the grout with a portion of the original soil to form *soilcrete* or, if required, it is possible to replace almost all of the soil with grout. By raising the drill stem absent any rotation it is possible to form soilcrete panels. The procedure has the advantage of effectiveness in virtually any type of soil, although the efficiency and thus strength of the resulting mass is soil-dependent. It is lowest in clays and cohesive soils, and increases as the soil becomes more granular. Not surprisingly, the greatest strength is obtained in clean sands and gravels, which result in a concrete-like mass. Jet grouting can be used in the presence of large rocks or other obstructions, although a shadow effect can interrupt the continuity of the grout solidification.

In application, a hole, typically about 4 inches in diameter, must be accurately bored to the desired depth of improvement. The initial hole can be bored separately or by a drilling or jetting tool mounted onto the jet grouting monitor, which contains the nozzles. There are three major variations of the jet grouting operation. In the simplest form, known as the *one-fluid system*, a special hollow drill rod equipped with a monitor that contains horizontal jet nozzles at the tip (Figure 6B.120) is lowered into the hole. A cement suspension grout is pumped down the drill rod at very high pressure (up to 9,000 psi) (52 kPa) while the drill rod and monitor are simultaneously rotated and withdrawn (Figure 6B.121). The grout, which exits the jet nozzles at high velocity, disintegrates the soil and

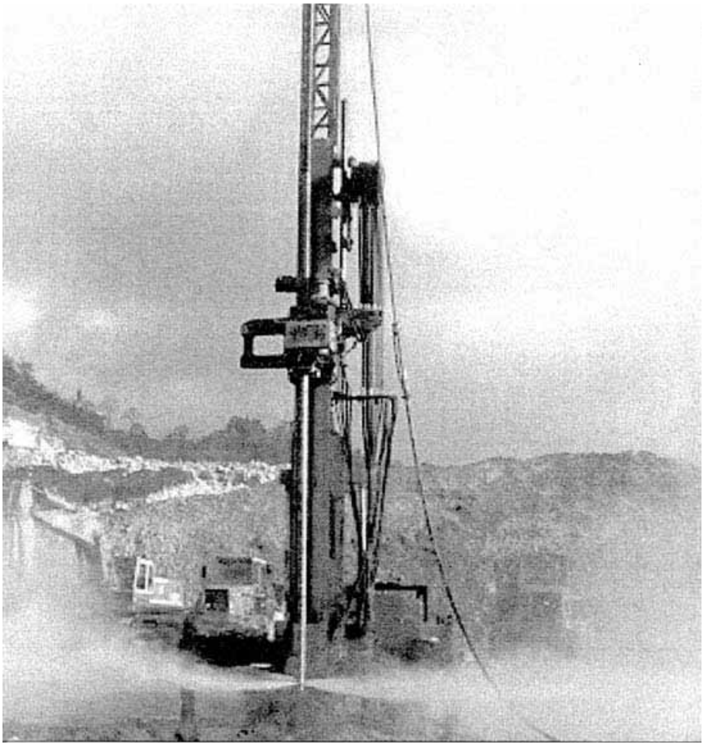

FIGURE 6B.119 Jet grouting monitor ejects water and/or grout at high pressure.

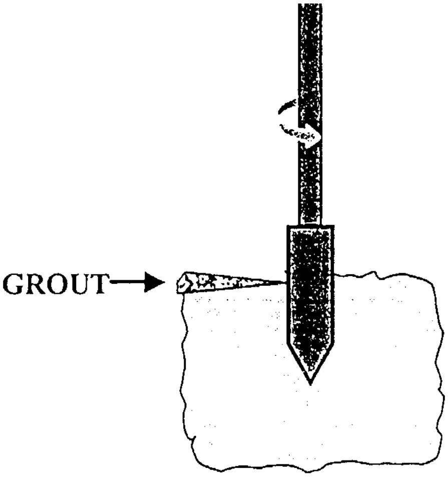

FIGURE 6B.120 One-fluid jet grouting system.

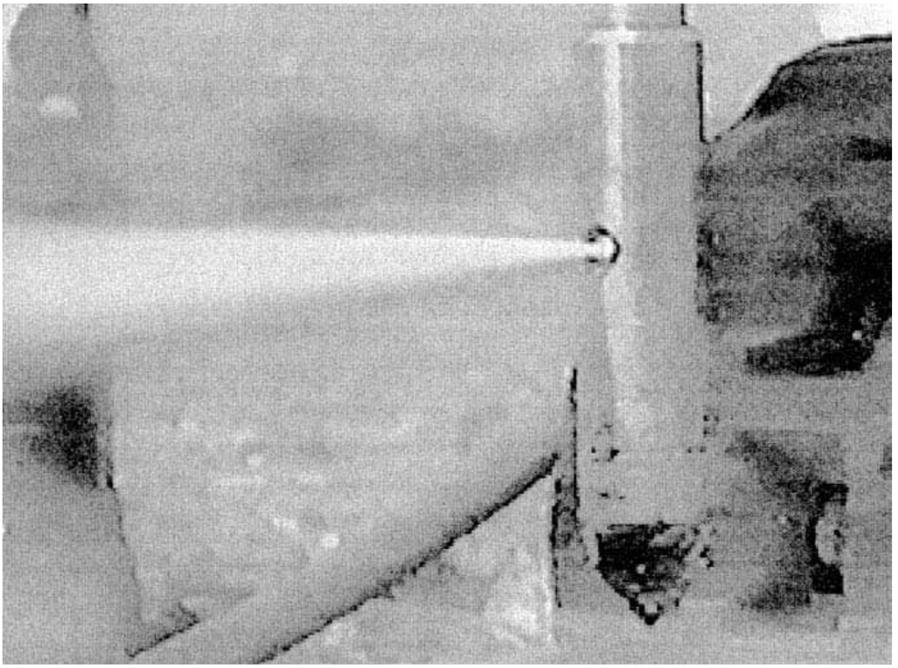

FIGURE 6B.121 Jet grouting monitor for one-fluid system.

mixes with it to form soilcrete columns. The effective radius of the grout jet is dependent upon the properties of the particular soil which is to be treated. Column diameters on the order of 15 to 30 (0.38 to 0.76 m) inches are typically obtained in cohesive soils, and diameters as great as four feet can be achieved in granular soils (Figure 6B.122).

In the *two fluid system* (Figure 6B.123), the grout is encased within a shroud of compressed air. This is facilitated by use of a special coaxial drill string and monitor. The air shroud acts as a buffer between groundwater and the grout, which greatly increases its cutting radius. The air also creates turbulence in the drill cuttings as they rise to the surface, greatly increasing the spoil removal efficiency. Diameter of the columns resulting from the two-fluid system are on the order of 1.5 to 3 feet (0.45 to 0.9 m) in cohesive soils and 3 to 6 feet(0.9 to 1.8 m) in granular soils.

The *three-fluid system* is the most complicated in that it requires a triaxial drill stem and monitor, with appropriate nozzles. In this system, an air-enshrouded jet of water erodes the soil as grout is simultaneously injected through separate nozzles (Figure 6B.124). The cutting jets are always located above the grout supply, allowing for a nearly complete replacement of the soil with grout as the monitor is withdrawn. The triple-fluid system enables formation of the largest diameter columns. Effective diameters of two to five feet in clays, and up to 10 feet (m) or more in sands (Figure 6B.125), have been experienced. It is, however, the most complicated system to use, and requires a very costly and complex drill string and monitor.

Jet grouting can be employed to depths of 200 feet or more. Its application is fast, and it can be performed in a wide range of soil types. Strength of the resulting soilcrete is dependent upon the original soil, and the jetting parameters (rotation and withdrawal rates) used, but can be of structural strength [2,000 psi (13.8 kPa) or more]. By overlapping probes, a nearly continuous curtain can be formed. Generally, large and sophisticated equipment, such as the rig shown in Figure 6B.119, is required which severely limits the availability and competitiveness of qualified contractors. The effec-

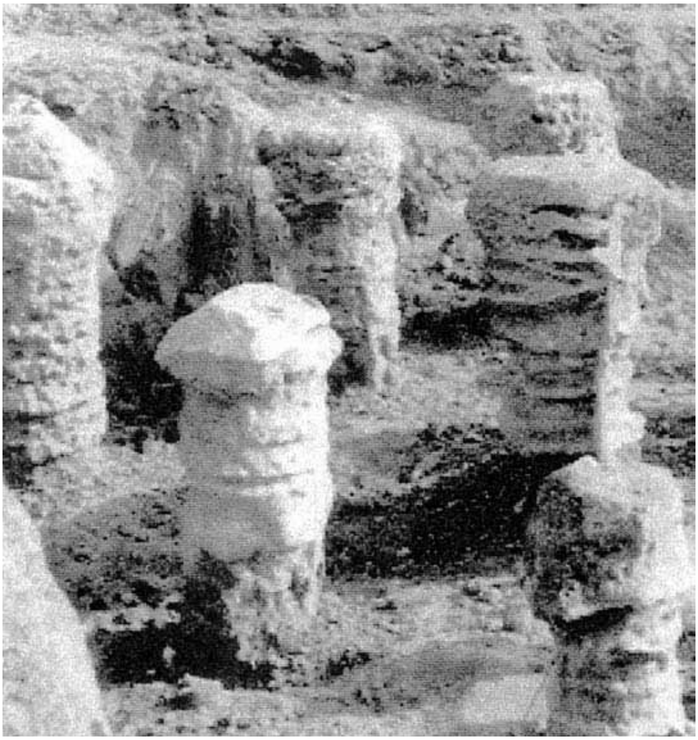

FIGURE 6B.122 Solidified columns resulting from one-fluid system.

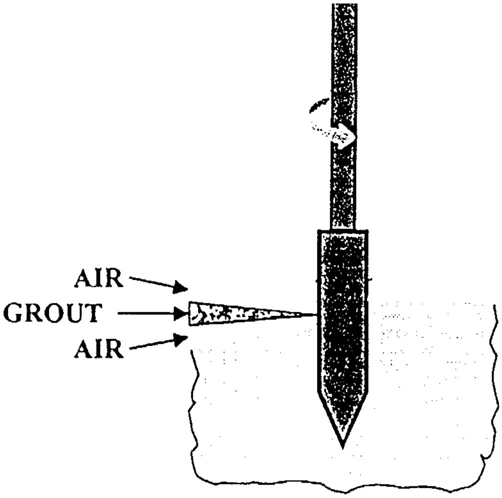

FIGURE 6B.123 Two-fluid system where the grout is enshrouded in air.

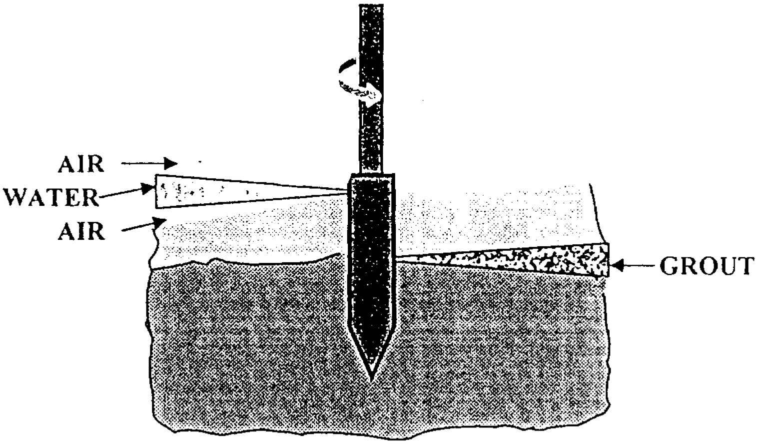

FIGURE 6B.124 Three-fluid system.

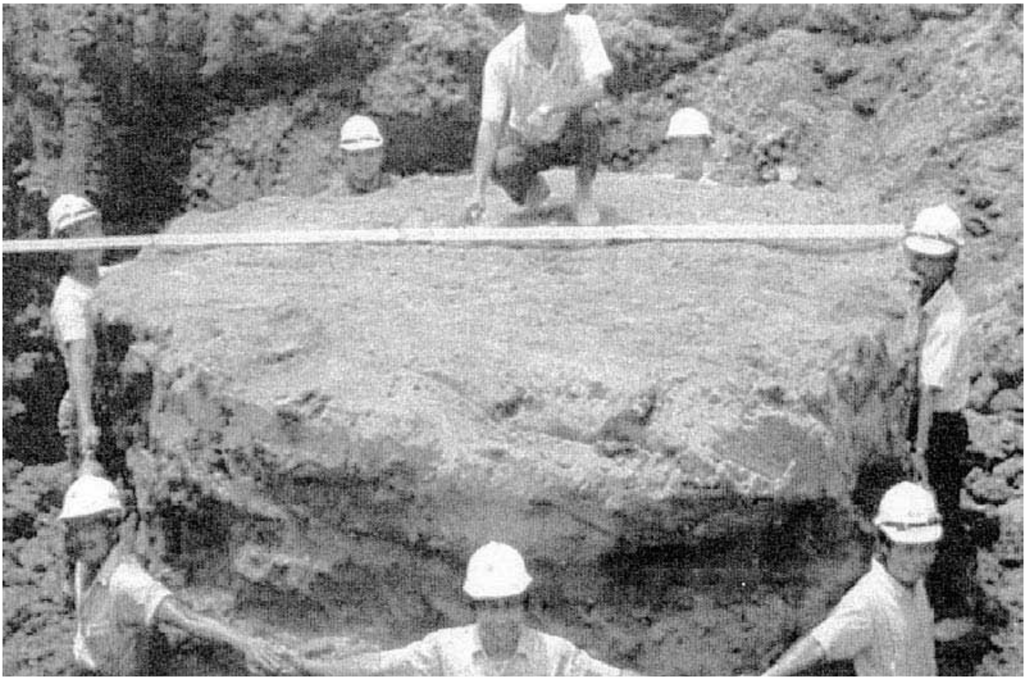

FIGURE 6B.125 Very large columns of soilcrete are possible with the three-fluid system.

tive diameter of the resulting mass is dependent on the individual system used, and the jetting and injection parameters, such as grout pressure, rate of rotation, and withdrawal speed of the jet. Considering these, and the influence of the particular site soil, trial injections are required to establish the optimal hole spacing and operating parameters.

Perhaps the most significant limiting factor, however, is the production of large amounts of spoil, which must be contained, handled, and disposed of. Also, the very high pressures used at the jet can be imparted into the soil at the nozzle depth if passage for the spoil should become blocked. This is a particular risk where the holes penetrate soft clays. Such high pressures could severely damage the adjacent soil and/or structures, and there are situations where the results could be catastrophic. All things considered, the practicability of the procedure is virtually limited to use in relatively large applications. Details of the optimal procedures to be used will vary with individual project requirements and the site soils. Consultation with available specialist contractors is thus advisable when the procedure is contemplated.

6B.6.5 Replacement/Compensation/Fill Grouting

Grouting is often used to fill voids within the soil. These might be large subsurface voids in the ground resulting from erosion, sinkhole activity, or solution cavities, or the result of old mine workings. Conversely, they might be small or thin planar voids resulting from leaking pipelines or similar events, or voids resulting from subsurface construction or tunneling. Filling of thin planar voids adjacent to a hard surface such as concrete (Figure 6B.126), usually referred to as *contact grouting* or sometimes as *backpack grouting* if in connection with a tunnel (Figure 6B.127), is usually performed with a cementitious fluid suspension or slurry grout. Any grout used to fill obvious voids should have a low water to cement ratio or be formulated to have little bleed or shrinkage. In this regard, defects thicker than about an inch are best filled with very thick slurry or plastic consistency mortar grout.

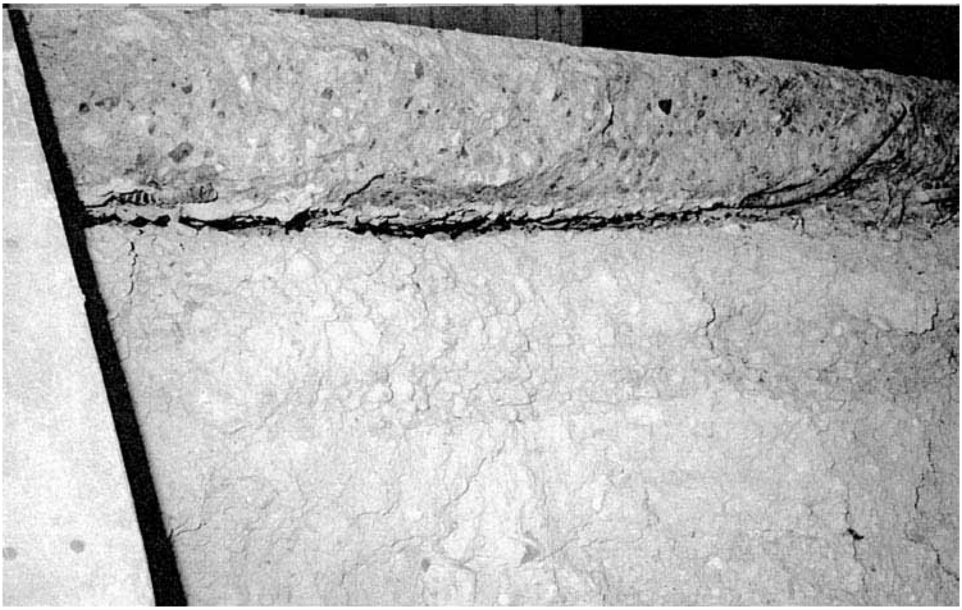

FIGURE 6B.126 Void under concrete foundation resulting from adjacent excavation.

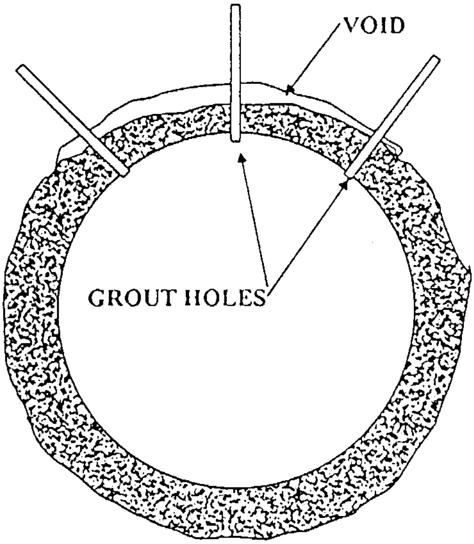

FIGURE 6B.127 Typical void remaining after concreting of a tunnel lining.

Very large voids, those with a volume of more than about a cubic yard, are usually best filled with commonly available concrete or mortar, utilizing ready-mixed material and standard concrete pumps for placement (Figure 6B.128). Unlike in earlier times, modern cement and concrete technology allow design of mixes with virtually any desired properties. As an example, otherwise common mixes can be made to be very cohesive by inclusion of *antiwashout* admixtures. The setting time can be accelerated to less than an hour, or delayed for up to several days.

For those applications where added weight is a concern, foaming agents can be introduced into the mixer so as to provide low-density, hardened compositions. As the density of a cementitious composition lowers, the strength will also be reduced. This is seldom a problem in geotechnical grouting, however, as strengths greater than that of the adjacent soil are seldom needed or justified. Typical density and strengths for readily available cellular concrete are on the order of 50 pounds per cubic foot (800 kg per m^3) for an unconfined compressive strength on the order of 300 psi (2 MPa), and 100 pounds per cubic foot (1600 kg per m^3) for a strength of about 1500 psi (10.3 MPa). An area where low-density materials have been found particularly advantageous is grout filling of the annular space around pipes and liners in tunnels (Figure 6B.129). Not only is the cost of the rather large amounts of material often used in such applications reduced, but also the risk of *floating* of the liner is greatly reduced due to the lower density of the fill.

For the filling large voids, rapid injection rates are usually quite acceptable, and relatively little pressure is normally required. Care must be taken however, to not develop too great a pressure when the defect is nearing complete filling. A rapidly increasing pressure will nearly always indicate this, as closure occurs. For this reason, pressure gages are required and must be monitored, as in all other grouting.

Although it is possible to design ready-mixed compositions with low water to cement rations, and thus minimal shrinkage, more often than not, such mixes will be subject to shrinkage. This can become significant where the depth of such masses is great. Therefore, where the filling of such large voids is to be complete and tight, injection of a fluid suspension or slurry contact grout can be made after the main body has been filled. Where done, this should be delayed as long as possible,

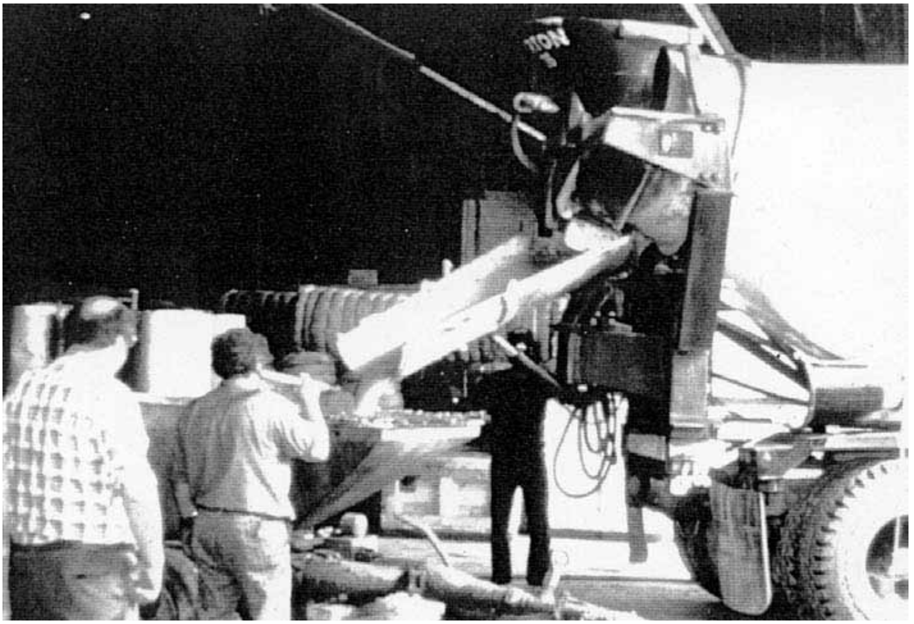

FIGURE 6B.128 Fill grouting of large voids with ready-mixed mortar and a standard concrete pump.

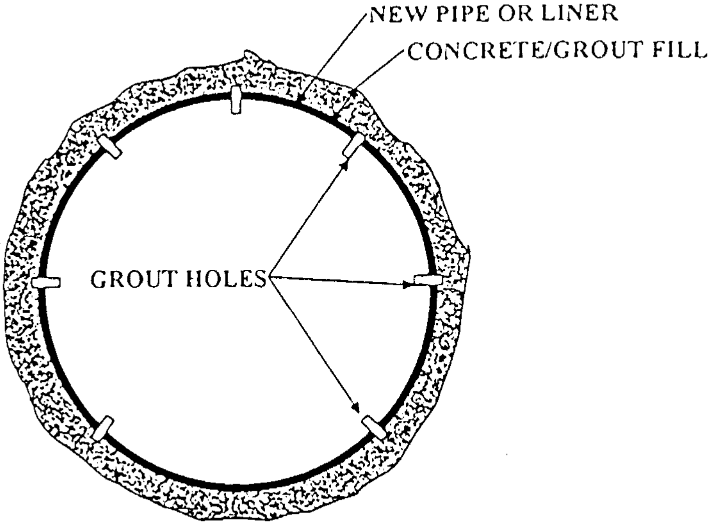

FIGURE 6B.129 Low-density fill for the annular space around linings provides economy as well as lessening the risk of floating the lining.

and at least several days, to allow primary shrinkage of the mass placement. In those situations where a strong grout is not wanted, it is possible to inject a low-mobility compaction-type grout with a very low cement content or even completely absent of cement; however, this will be much more expensive than simply filling with ready-mixed material.

6B.6.6 Groutjacking

It has long been recognized that settled pavements could be accurately raised by slab jacking, which is simply the controlled injection of grout under them. (Refer also to Section 7B.2.) If a void exists under the slab, which allows the grout to immediately spread, no further preparation is required. If such is not the case, a void can be created by jetting through the grout holes with water or compressed air (Committee on Grouting, 1977). The same principal is involved when groutjacking a structure, except that the void must extend over a larger area, due to the higher loads involved. The area of void required is a function of the weight of the structure to be jacked times the available grout pressure, and must include provision for any frictional resistance between the structure and the adjacent soil. Thus, for heavy structures, considerable pressure and area is needed. Such lifting has sometimes been facilitated by horizontal boring or hand mining under the structure. Whereas light pavements can be raised with a wide variety of grout materials and only nominal pressure [generally less than 100 psi (690 kPa)], once lift has been initiated, very low mobility grouts such as those used for compaction grouting and generally much higher pressures (500 to 2,000 psi) (3.4 to 13.8 MPa) are required under heavy structures.

If compaction grouting is being used to improve the soils under a structure, it is possible to lift it during the grout injection. If sufficient lift has not been obtained during the compaction, however, or perhaps the underlying soils are not in need of improvement, it is possible to jack a structure by creating a *controlled*, horizontal hydraulic fracture in the soil. The fracture is then expanded or *jacked* until the desired lift occurs. Uncontrolled hydraulic fractures that occur during grouting, usually follow the orientation of the minor principal stress of the soil, which is normally a vertical plane. A horizontal fracture can usually be created, however, by injecting a small amount (3 to 5 gallons) of a highly mobile grout into a casing that extends to within about one inch of the bottom of the hole. A low-mobility grout, such as that used for compaction grouting, can then be injected so as to open the fracture and raise the ground and overlying structure. Raising of both pavements and structures to very close tolerances is possible. Although many grouting contractors lack the knowledge and ability for this type of work, those who are qualified will routinely raise a structure to within about 0.1 of an inch of the specified elevation.

One side of an aluminum smelter, which was in full operation and filled with molten metal, had settled several inches. The differential settlement resulted in fracture of some of the steel framing element connections, which restrained the firebrick lining (Figure 6B.130). The very high heat in the furnace area made investigation of the problem or remedial work extremely difficult. However, because the owner had a commitment for delivery of a highly specialized alloy, several months of further operation were required, and something had to be done to temporarily relieve the distress of the steel containment structure.

Grout holes inclined about 60 degrees off of vertical were drilled under the settled side. They were fully cased to the edge of the shallow foundation elements, and extended about another eight feet under the base. About one cubic foot of slurry-consistency grout was injected into the holes, followed by low-mobility grout, such as would be used for compaction grouting. Although, the objectives of the work were to raise the settled portion of the structure only, and no compaction of the underlying soils was intended, an unexpectedly high grout take revealed that the silty soils immediately under the foundation were quite loose. Although the intent of the groutjacking was limited to raising the settled portion of the foundation about one inch, in order to temporarily relieve the structural distress, the foundations were raised nearly four inches (20.1 cm) to near their original level.

Based on the grout takes and the history of the site, it was suspected that localized soil distur-

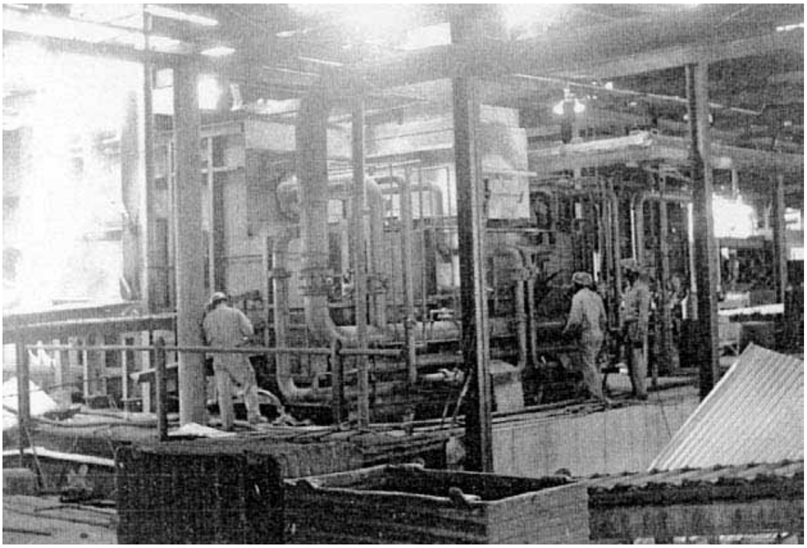

FIGURE 6B.130 Metal furnace full of molten aluminum during groutjacking.

bance resulted when a drainage sump was removed from the area prior to construction of the furnace. As no further settlement occurred over the next several months, the originally contemplated investigation and permanent remedial program were cancelled.

Work on a new parking structure in Texas had to be stopped due to differential settlement of the pile-supported foundation. Groutjacking was used to precisely raise the individual settled columns and their supporting piles (Figure 6B.131). A two inch (5 cm) ID casing, such as that commonly used for compaction grouting, was placed to slightly past the pile tip elevation. A five gallon bucket of cementitious slurry was dumped into the casing, immediately followed by a compaction grout mix that incorporated the preferred aggregate gradation shown in Figure 6B.57. The columns were all raised to within about 1/16 th of an inch (1.5 mm) of their original elevation.

6B.7 SPECIAL CONSTRUCTION TECHNIQUES INVOLVING GROUTING

There are a number of specialized foundation construction methods that involve grouting of one sort or another. The development of such systems has advanced rapidly within the last several years. This has been greatly facilitated by the ready availability of both high-capacity cranes and modern concrete and mortar pumps, as well as significant advances in cement and concrete technology.

6B.7.1 Auger-Cast Piles

In the late 1940s, the Intrusion-Prepakt Company began experimenting with novel pile installation methods. An early result was the auger-cast pile. Therein, a hollow-stem, continuous-flight auger of the pile design diameter (Figure 6B.132), is drilled into the soil to the intended pile tip elevation. A

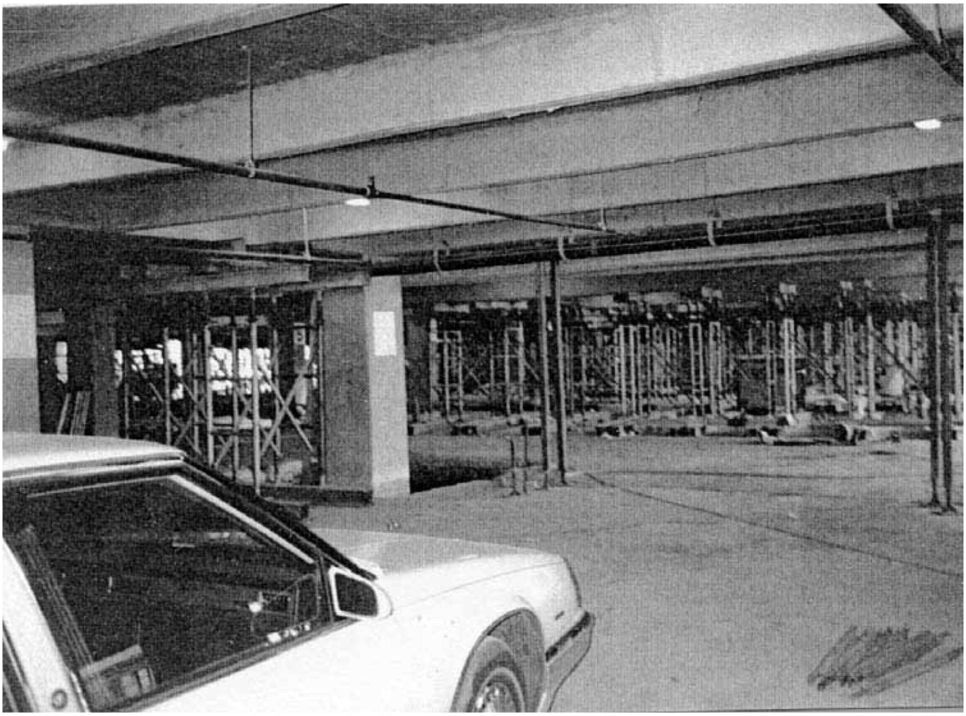

FIGURE 6B.131 Groutjacking was used to precisely level pile-supported columns of parking structure.

cementitious grout or mortar is then injected as the auger is withdrawn. The injection pressure of the grout can actually aid in the withdrawal of the soil-laden auger, and contributes significantly to the realized skin friction. As originally conceived, equipment limitations controlled the size of piles that could be practically constructed, and thus their capacity.

With the advent of the modern concrete pump in the 1960s, it became practicable to rapidly inject high-strength mortar or small-aggregate concrete, rather than the original particulate grout. This allowed virtually complete filling of the hole with the higher strength replacement material, which greatly widened the obtainable pile capacities and the range of soil for which the technique is suitable. This period also saw significant advancements in hydraulic power transmission technology, which made possible very high torque drilling machines and high-capacity cranes and carriages on which to mount them.

With presently available equipment and technology, auger-cast piles can be installed in virtually any granular soil that is relatively free of large rocks or boulders. Piles with diameters up to 36 inches and lengths on the order of 130 feet (39 m) have been constructed. They are routinely placed to withstand a design load of 200 tons, (182 metric ton) or more. The auger-cast method can be used to construct individual piles, including batter piles, or any combination or groups. In fact, the method has been used extensively to install secant piles so as to form a continuous wall for earth retention (Figure 6B.133). The technology for auger-cast piles is now well developed and several specialty contractors regularly install them. The Deep Foundations Institute (1993) has available a reference, *Augered Cast-in Place Piles Manual*, which discusses the process in detail and includes a guideline specification.

FIGURE 6B.132 Casting of an auger-cast pile.

6B.7.2 Mixed-in-Place Piles

Mixed-in-place piles involve rotating a hollow drill stem into the ground to the proposed pile tip elevation. The stem is equipped with suitable outlet nozzles and mixing blades of the desired pile diameter. A cementitious suspension grout is pumped down the hollow stem while it is slowly inserted and simultaneously rotated so as to initially blend the grout into the in situ soil. Rotation continues as the apparatus is slowly withdrawn from the hole, further mixing so as to enhance the initial blending. As originally conceived in the early 1950s, the finished pile would be composed of a mixture of the native material with a cement suspension grout. The piles can be formed in a range of soil materials, as long as they are not very stiff or very dense and are free of large rocks or boulders. Strength of the resulting piles is dependent on the amount of grout blended and the gradation of the in situ soil.

In essentially clean sands that are reasonably free of silt or clay fines, strengths similar to those of conventional mortars are routinely obtained. As the amount of fines increases, however, the resulting strength levels decrease. Auger-cast piles have many advantages. They can be installed quickly, and the installation is relatively free of noise and vibration. Because the hole is always filled with either soil or the finished pile, neither groundwater nor caving soil is a problem. Mixed-in-place-piles were developed by the Intrusion-Prepakt Company in the early 1950s and were the forerunner of the much larger-scale process now known as DMS (deep soil mixing). Traditional mixed-in-place-piles were commonly 16 inches in diameter with depths up to about 50 feet (15 m). Because of the method of formation, they provide a very tight and irregular interface with the surrounding soil, and capacities are thus often greater than equivalent driven piles.

FIGURE 6B.133 Shoring composed of cantilevered auger-cast secant piles.

6B.7.3 Deep Soil Mixing

Deep soil mixing involves the simultaneous injection and mechanical mixing of a cementitious suspension grout with in situ soil. It is applicable to any soil that is not very stiff or dense and is free of boulders. The process requires very large and sophisticated equipment and overhead clearance that must be greater than the depth of the mixed soils. This limits its application to generally large projects. The process, as now practiced, was developed in Japan in the early 1970s and has been used in the United States since 1986. The grout typically used is a cement suspension, which might contain a small amount of bentonite (less than 5%).

Deep soil mixing can be used for the construction of individual elements such as columns or short walls, although its most common application is to construct continuous walls. These may be either structural, such as for earth retention, or hydraulic for cutting off the underground flow of water or other liquid. The size of the element produced varies with the equipment used, but thickness is usually between three and five feet (0.9 and 1.5 m), with the width of a single probe on the order of three times the thickness. Common depth limits are on the order of 100 to 150 feet (30 to 45 m), although more than 200 feet (60 m) have been produced. The process is highly mechanized and employment of high-capacity, fully automated grout plants is common. It is thus performed only by a limited number of specialty contractors, most of which have their own proprietary equipment and methodology.

6B.7.4 Micropiles

Micropiles are formed by drilling of the pile hole, usually with simultaneous placement of a stay-in-place casing. Structurally, they are thus a composite of the casing and a grout fill. They are typically designed as friction piles, and their intimate contact with the surrounding soil is thus critical. Micropiles are of relative small diameter, usually less than 10 inches (25 cm), and have been placed to depths of about 200 feet (30 m), although more typical depths are on the order of 100 feet (60 m) or less. The casing is usually composed of appropriate lengths of threaded flush wall sections. In order to accommodate the flush threaded joint and the structural requirements of the particular design, fairly thick casing walls are typical. Additional reinforcement can be embedded into the grout fill before it hardens, if required.

Once the pile is drilled to depth, a cementitious grout is injected, oftentimes through the drill rig mud swivel. Although cement suspension grouts are the most often used, ready-mixed mortars have also been employed. Typically, the drill bit is of greater diameter than the outside diameter of the casing, resulting in an annular space, which must be filled with grout, so as to provide the intimate contact with the soil required of a friction pile. The grout is usually injected into the casing, with injection continuing until it has returned up the annulus to the surface. The casing is nearly always custom fabricated, and can be of any length, which allows the piles to be placed in areas of restricted headroom. Figure 6B.134 shows an installation of closely spaced, 200 foot deep micropiles in an area of restricted access. Because of the requirement of special equipment and skills, micropiles are generally constructed by a limited number of specialty contractors, many of whom have their own proprietary systems. The work is often performed on a design–build basis, with performance load testing of the design pile capacity the primary acceptance requirement.

6B.7.5 Posttensioned Soil Anchors

The pullout capacity of soil anchors can be greatly enhanced by injection of grout in the anchorage zone. This is the result of both the increased intimacy at the anchor–soil interface and, often, an improved soil in the anchorage zone. Although cement suspension grout is most commonly used, chemical permeation grouting has been employed in sandy soils. The grout is usually injected to the tip of the anchor, either through a separate injection tube or the anchor itself, in the case of hollow stem anchors. Construction of ground anchors is somewhat specialized, and grouting in connection therewith is usually performed by the anchor contractor.

6B.8 INJECTION FUNDAMENTALS

Regardless of the type of grout being used or the purpose of its injection, there are several important parameters that will always be present and must be considered. A good understanding of these factors and their relationships are thus fundamental to proper design or field performance of any grouting program. The pore pressure of the soil into which grout is being injected comes under increased pressure and, oftentimes, the soil is subjected to mechanical disruption as the grout is introduced. It is important to understand these factors, and to make prompt adjustments in the injection parameters as required, if excessive disruption or damage is to be precluded.

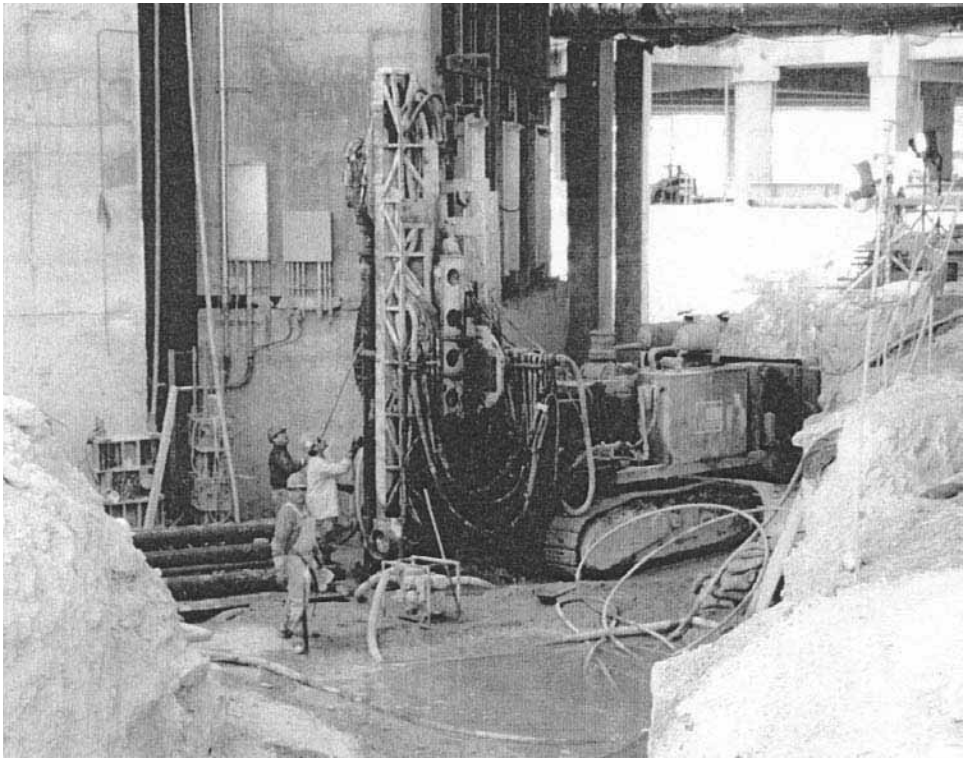

FIGURE 6B.134 Construction of micropiles in an area of restricted access.

Except in those cases where continuous monitoring of previously installed geotechnical instrumentation occurs, the only parameter that provides timely information is the injection pressure, and more particularly, its behavior during injection. Although a given pressure level in itself is of little value, knowledge of the pressure variations at a constant pumping rate usually reveal much valuable information as to the constantly changing conditions in the soil at the point of injection. The pressure is directly effected by both the grout rheology and the pumping rate. Whereas changing the grout rheology during injection usually requires a time delay, adjusting the injection rate can be done instantly, providing proper equipment is being used. Continuous observation of the grout pressure behavior and its control through appropriate variation of the pumping rate is necessary if optimal effectiveness of the grout injection is to be realized.

6B.8.1 Injection Pressure

Setting of the maximum injection pressure is perhaps one of the most contentious issues in geotechnical grouting, especially in the United States. A widely recognized rule of thumb says that the maximum pressure in psi should be limited to twice the injection depth in feet. Well-documented experience, in the United States as well as abroad, however, has proven this idea to be faulty. The developed pressure is directly proportional to the rate of injection. For reasons of economics, use of the highest *safe* injection rate, and thus highest pressure, is desirable. This pressure level will be variable and is dependent upon many factors, including the type and rheology of the grout being used, the purpose of the injection, particulars of the site soil, hole geometry, etc.

The pressure used in grouting can be divided into two distinct modes: that which results from friction and any restriction within the delivery system (pump output to lower end of casing), and that which results from resistance to permeation or displacement of the formation or other medium being filled. Where injection is through a void of uniform cross section, such as a pipe or injection hose, the total pumping pressure will be dependent upon the length thereof, and will be the product of the unit length resistance times the number of units of length. Unless a void is filled with water or isolated such that air cannot escape, pressure at the leading edge of the grout will always be zero. Where injection is made into a porous mass such as soil, or voids of varying size and configuration, the pressure distribution within the grout mass will be ever variable. It will, however, always be greatest at the point of injection, which is typically the lower end of the injection casing, and will decrease with distance therefrom.

Because the effective pressure on the formation will invariably start at the casing outlet, it is important to consider the pressure differential between that point and the top of the casing, where pressure readings are typically observed. This value may be either positive or negative, depending upon the particulars of both the grout and the delivery system. As an example, on a recent compaction grouting project, the contractor proposed use of a six inch (150 mm) diameter injection casing for some very deep holes. Because of concern that the static head pressure might be excessive, a trial injection using the six inch casing was made. A positive grout pressure of 0.45 psi (0.3 kPa) per vertical foot (0.33 m) of casing resulted. Because the total gravity pressure that would result at the bottom of the hole would be excessive, a smaller three inch (75 mm) casing was required for the production work. This resulted in a head *loss* of about 0.45 psi (0.3 kPa) per vertical foot.

With the above in mind, it is possible, through experimentation and calculation, to determine the *approximate* pressure differential between the pressure gauge at the point of injection and the casing outlet, where the grout makes first contact with the formation. The term approximate is used because some variation will occur within a delivery line, depending upon the smoothness of its interior, number and sharpness of bends, and any restrictions at couplings or other fittings. Because it is the pressure at the bottom of the injection casing that is pertinent, the gauges for pressure evaluation should always be located as close as possible to the point of grout entrance into the formation, and the injection pressure corrected for any loss or gain.

Many factors contribute to the total pump pressure required for injection. Major contributors to that pressure are frictional resistance within the delivery system, rheology of the grout being pumped, rate of pumping, size of the lines and resulting grout velocity, and frictional resistance at the grout conduit wall interface, which is a function of the wall smoothness and its affinity for the particular grout being used. Once the grout is in the medium to be filled, both the size of the individual fractures or pores and their surface condition are major factors. Obviously, the rheology of the grout and its affinity for the formation surfaces are also significant contributors.

6B.8.2 Pumping Rate

As aforementioned, the pumping rate and developed pressure are directly related. An increase in pumping rate will *always* result in an increase in pressure, as will a reduction result in lower pressure. This is well illustrated in Figure 6B.135, which is an actual printout of a computer-generated record of pressure behavior on a recent project where the grout pump malfunctioned. The piston speed varied such that one piston was traveling nearly twice as rapidly as the other. The actual pump strokes can be observed by their pressure differentials, and it will be noted that the pressure on the short stroke (higher pumping rate) was more than 100 psi greater than that on the slower long stroke. In this figure, it is also of note that the pressure raised slightly during each of the short strokes, which is indicative of a generally optimal pumping rate, whereas there was a slight pressure decay during the long strokes, which indicates a slower than optimal rate.

Because the resistance developed within the formation to be treated is beyond our control, the in-

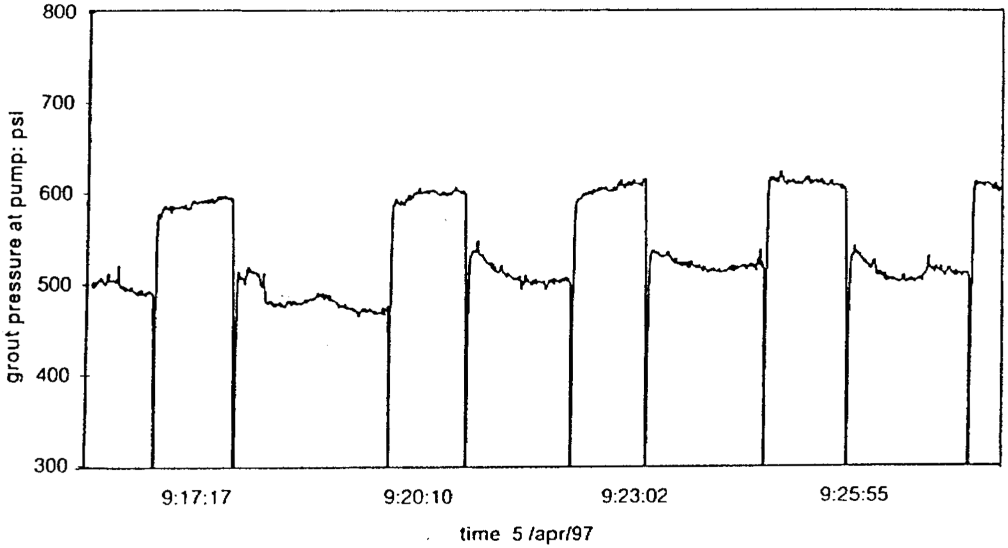

FIGURE 6B.135 Computer-generated printout showing variation of pressure concurrent with changes in pumping rate.

jection rate selected must be limited, such that excessive pressure will not be developed. Because wide variation of formation resistance is common, flexibility of the output capacity of the pump is usually required. This is no doubt the reason for the past widespread use of circulating grout injection systems. In such systems, the grout is continuously circulated in a route from the pump, past the grout hole, and back to the pump, as shown on Figure 6B.136. A connection to the grout hole is made with a "T" fitting in the line. A valve is located on the return side of the "T" fitting and a valve and pressure gauge are located between the "T" fitting and the injection hole casing, as illustrated in Figure 6B.137. The amount of grout that is allowed to go into the hole, and thus the injection rate, can be varied by "throttling" the valves. Because economy dictates use of the highest practical injection rate, the valves are adjusted so as to allow the maximum amount of flow possible, without exceeding the maximum allowable pressure.

Single-line direct delivery systems, wherein the grout flow is directly from the pump to the grout hole (Figure 6B.138), should be used only with variable output pumps. Therein, the speed of the pump, and thus the rate of the grout output, is regulated so as to maintain injection rates that will not exceed the allowable pressure. Varying the rate in discrete increments provides the advantage of observation of even slight changes of pressure, facilitating optimum evaluation of the grout movement within the formation.

Grout pumps and delivery systems should be sized for reasonable injection rates, so as to preclude the development of excessive pressures during injection. All other things being equal, system pressures will usually diminish as the pipe or hose size increases. The size should not be so large, however, as to preclude complete emptying of the system within a reasonable time. In this regard, the set time of the grout must be considered so that it is not allowed to react within the system. Many grout materials, and especially cementitious mixes, will tend to build up on the wall of the delivery system if sufficient grout velocity is not maintained. Such build-up results in an ever decreasing opening for the grout to travel, and will thus increase both the grout velocity and resulting pressure. Figure 6B.139 delineates the velocity of grout through various size delivery lines.

The importance of the relationship of injection pressure to pumping rate cannot be overempha-

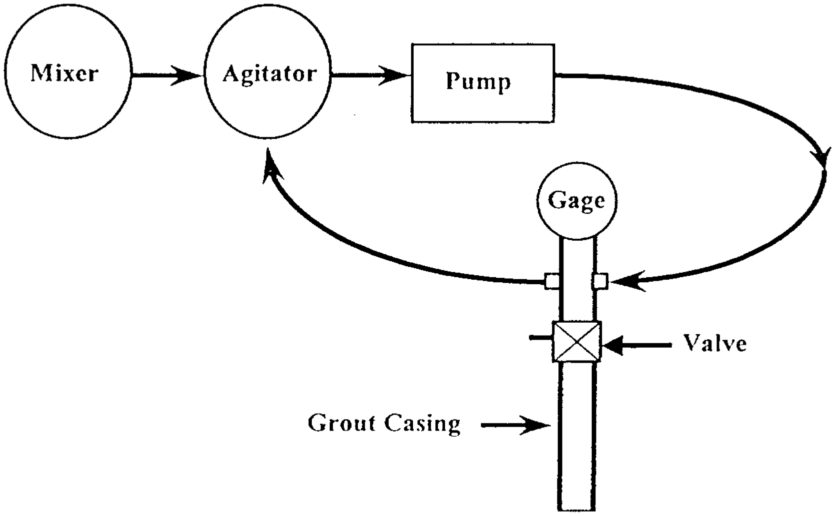

FIGURE 6B.136 Circulating injection system with grout return to the agitator.

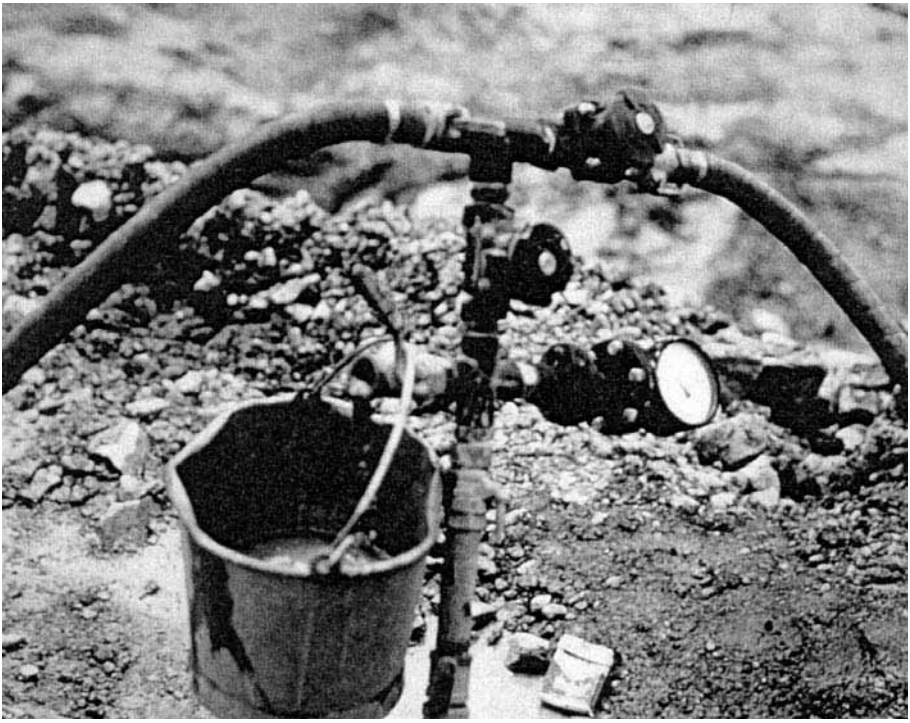

FIGURE 6B.137 Valves are manipulated to maintain proper pressure.

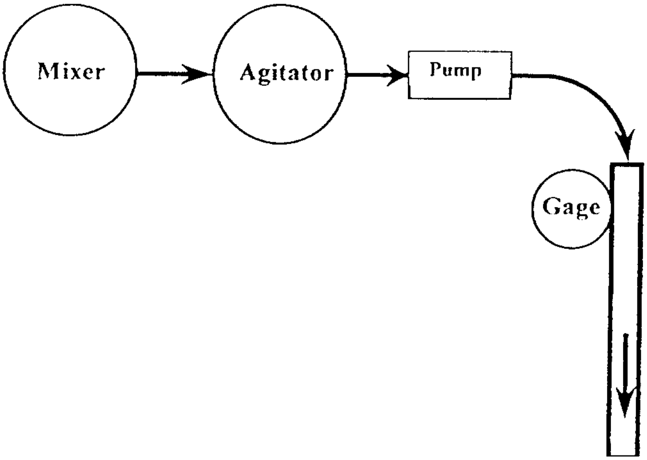

FIGURE 6B.138 Direct injection system where grout flows from the pump directly to the grout hole collar.

GROUT VELOCITY IN VARIOUS SIZE LINES

FLOW Cubic Feet/Minute	FLOW Gallons/Minute	VELOCITY Feet/Second	FLOW Cubic Feet/Minute	FLOW Gallons/Minute	VELOCITY Feet/Second
1/2 Inch Line			**1 1/4 Inch Line**		
	0.5	0.8	0.5	3.8	0.7
0.1	1	1.6	1	7.5	1.4
0.3	2	3.3	1.5	11.3	2.0
0.4	3	4.9	2	15	2.7
0.5	4	6.5	3	22.5	4.1
			4	30	5.4
3/4 Inch Line			**1 1/2 Inch Line**		
0.1	1	0.7	.50	3.8	0.6
0.3	2	1.4	1	7.5	1.3
0.4	3	2.2	1.5	11.3	2.0
0.5	4	2.9	2	15	2.7
0.7	5	3.6	3	22.5	3.5
0.8	6	4.4	4	30	5.3
0.9	7	5.1			
1.1	8	5.8			
1 Inch Line			**2 Inch Line**		
0.3	2	0.8	1	7.5	0.7
0.5	4	1.6	1.5	11.3	1
0.8	6	2.5	2	15	1.4
1.1	8	3.3	3	22.5	2.2
1.3	10	4.1	4	30	2.9
1.6	12	4.9	5	38	3.6
1.9	14	5.7	6	45	4.4

FIGURE 6B.139 Velocity of grout through various sized delivery lines.

sized. Many practitioners seem to fall under the *erroneous* impression that the maximum grout pressure level alone indicates the degree of soil improvement obtained. In many cases of investigating the cause of grouting program failures, the writer has found records that indicated only the hole number, grouting depth, and *maximum* pressure reached during injection. Such records are usually short of useless for obtaining an understanding of what actually happened in the ground, and thus the cause of the failure. The pumping rate/pressure relationship is so important that it warrants special attention. The specifying or recording of injection pressure is of absolutely *no* use unless the injection rate at which the pressure level occurred is also specified and/or recorded.

6B.8.3 Pressure Behavior

A basic law of physics tells us that an increase in pressure *always* indicates greater resistance to flow, whereas a reduction indicates less resistance. Although somewhat subjective, knowing the value and nature of the pressure movement allows one with experience to make an informed prediction as to the cause. As an example, if we were to assume that a pressure of 100 psi (700 kPa) was required to pump a grout material through a given length of hose, should a connection located in the middle of the length become broken, the pressure would sharply drop to approximately half of its original value. Similarly, were a restriction placed on its outlet end, a sudden increase in pressure would result. Applying this theory to the real world of grout injection, it is widely recognized that a sudden reduction in pressure indicates a likely disruption, in the interior of the soil or rock mass being grouted (Houlsby, 1990), (Warner and Brown, 1974).

Within the writer's experience, sudden changes in pressure, or increases of injection rate at a constant pressure, *always* indicate the occurrence of a significant event. In more cases than not, such events lead to negative results, which can be greatly minimized, and in many cases completely averted, through quick response, usually the immediate lowering of the pumping rate or complete cessation of injection.

Typical events that are accompanied by a loss of pressure include hydraulic fracturing of the soil; displacement or heaving of the formation; grout loss into a subsurface pipe or other substructure; outward displacement of a downslope or retaining wall; grout entering a much larger fracture or void, or encountering a much softer or more permeable formation; thinning of the grout or other change in the grout rheology, which increases its mobility; leakage of the grout; or pump malfunction.

An interesting example was observed during the injection of a very deep compaction grouting hole. A sudden drop in pressure was noted. Consideration of the value of the pressure drop, combined with the depth of injection and the injection rate, ruled out hydraulic fracturing of the formation as the cause. The particular circumstances of the injection led to the conclusion that some sort of break in the injection line was the most plausible source of the sudden pressure loss. The writer assumed that a casing joint at a fairly shallow depth had failed. After withdrawal of forty feet of casing, however, a vertical split, as shown in Figure 6B.140, was found. Whereas there were no underground pipes in the area of the injection, if such had existed, the indicated pressure behavior could have indicated leakage of the grout therein. The importance of meticulous monitoring of the injection behavior cannot be overemphasized.

Events that are preceded by an increase in pressure include plugging or restriction in the injection system or formation, thickening or lowering the mobility of the grout, completion of the filling of a fracture or void, or binding or wedging of a structure or slab that is being raised, (Committee on Grouting, 1977)

The compaction grouting procedure employs relatively high pressures, even at shallow depths. This results in relatively large pressure variations that are especially instructive. Significant pressure movements develop during injection, as conditions within the zone of influence of the growing grout mass change. Where the defective soil is of a uniform though inadequate condition, such as in the case of very loose wind-deposited granular materials, the pressure buildup will generally be constant, as illustrated in Figure 6B.141.

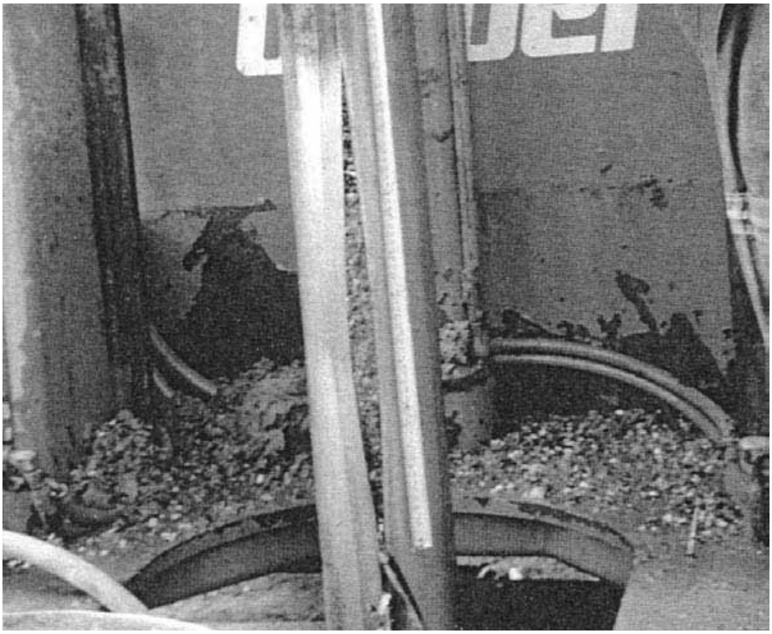

FIGURE 6B.140 Split of casing that resulted in sudden drop of injection pressure.

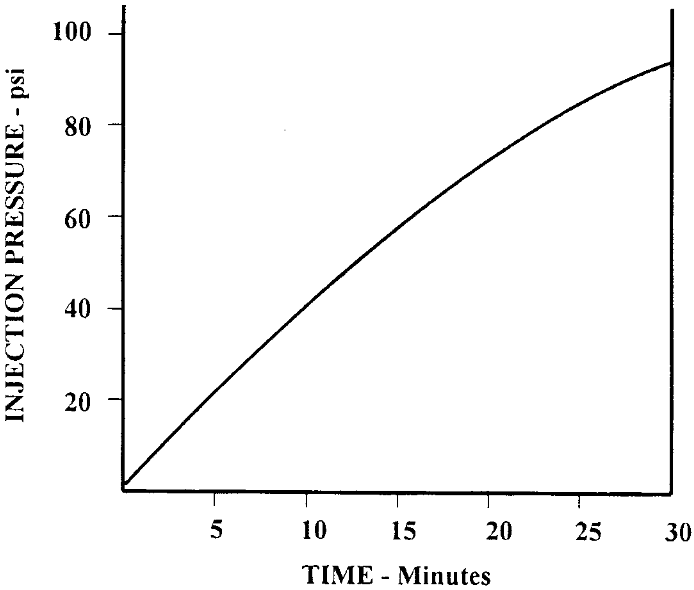

FIGURE 6B.141 Steady pressure buildup indicates generally uniform soil.

Often times, however, settlement problems develop as a result of defects such as masses of buried brush or large boulders in or under a fill, uncompacted soil resulting from initial clearing or development of haul roads, or insufficient removal of slopewash or other weak surface materials prior to fill placement. Both the existence and approximate volume of such subsurface defects can be determined through continuous pressure behavior monitoring. In such cases, a nonuniform pressure buildup will be experienced (Figure 5.B.142). In the figure, the approximate volume of the apparent defect can be ascertained by determining the amount of grout that was injected from the time of the initial pressure drop until the previous pressure level is regained.

As soil grouting is frequently performed under or in close proximity to existing structures, underground piping and other substructure are often nearby. There is thus inherent with the procedure always a risk of grout leakage into such utilities. The occurrence of grout entering substructures will nearly always be preceded by a significant drop of the injection pressure. Whereas such drops commonly indicate other events, in cases where grout has entered an underground pipe a slow but steady increase of the initial entrance pressure will typically occur. In such cases, the level and behavior of the pressure will usually be markedly different from that which was experienced prior to the occurrence of leakage, signaling the need for corrective action.

Whereas major events can be detected by substantial pressure movements or changes of injection rate, more subtle events often result in only minor changes. Such minor changes cannot easily be observed in circulating injection systems or those systems that are subject to pulsations from pump stroking. Thus, where detection of minor events is important, use of a constant output pump, free of any significant pulsation, combined with careful and continuous pressure monitoring is recommended. Piston pumps operating at rates greater than about one hundred strokes per minute, combined with a minimum hose length of about 100 feet (33 m), have been found satisfactory, as at high rates the stroking results only in a flutter of the gauge needle, and dampening occurs in the flexible hose. Pressure pulsations resulting from stroking of piston or diaphragm type pumps can

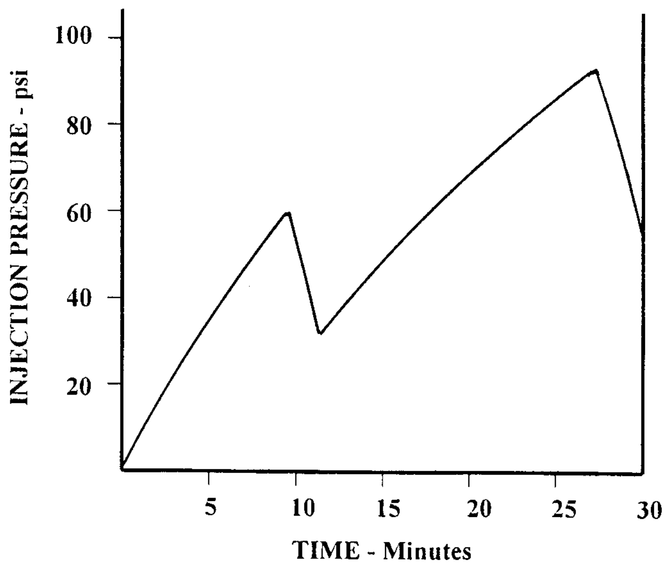

FIGURE 6B.142 Erratic pressure buildup indicates subsurface voids.

also be lessened through use of a hydraulic accumulator near the pump outlet and/or an increase of at least 50% in the diameter of the injection hose for a distance of about 25 feet (7 m), starting at the pump outlet. Conversely, as illustrated in Figure 6B.135, the actual pressure of individual pump strokes can be observed in the case of piston pumps operating at very slow stroke rates.

Pressure behavior can be obtained through continuous readings of a pressure gauge and manual entries onto an appropriate form, or with fully automated equipment employing either a continuous disk or strip chart recorder, or with computer processing. Regardless of the recording method, the actual time of each entry should be included, and when feasible, continuous real time should be included in the record. When manual monitoring is performed, it is good practice to record entries at predetermined uniform increments of pressure. Alternatively, entries can be made at regular time intervals. The magnitude of pressure increments or time intervals employed will vary according to the individual application and type of grout being used, but should be of sufficient frequency to allow plotting of a curve that accurately displays all significant pressure movements.

Although disk recorders are commonly used with automated systems, especially in Europe, and are satisfactory for monitoring pressure behavior, continuous strip chart recorders can include other injection parameters that are more easily compared and interpreted and are thus favored by the writer. Regardless of the method used, the records produced must include real time and facilitate immediate interpretation on the job site, as well as provide a permanent record for later reference.

Computer monitoring systems are now readily available that provide for instantaneous readout of all injection parameters. They have the advantage of providing a permanent record on disk and the ability to print hard copies as desired. When using traditional practice employing circulating systems, the constant variation of injection rate virtually precludes accurate manual recording. The use of real time, continuous computer monitoring is thus especially beneficial in such cases.

6B.8.4 Pressure–Volume Relationship

The quantity of grout pumped at any one location and the shape that it forms in the soil have a significant effect upon the soil reaction. A given pressure on a very large mass of grout will affect a greater area, and thus exert a much larger *total* force within the soil, than will a small mass (Figure 6B.143). Depending upon the particulars of the individual injection, this might affect the pressure level that can be safely used. Where large quantities of grout are placed, the pressure used for initial injection may be too high once a significant mass of grout has been placed. Therefore, reduction of the pumping rate, and thus pressure, is often in order during injection as the injected quantity of grout grows.

The shape of the injected grout mass is also important. As an example, if the grout were to form an essentially horizontal lens, a large surface area would be affected, and the likelihood of displacing the surface upward would be great. If a vertical fracture were created, any effect of the grout would likely be adjacent to the plane of the fracture, rather than in the planned zone adjacent to the hole. As previously discussed, initiation of *controlled* groutjacking usually requires a relatively low-mobility grout. Jacking can occur with any grout, however, if the injection rate exceeds the rate at which the soil is able to accept it. Much damage has been done as a result of *uncontrolled* jacking resulting from too great a pumping rate and resulting excessive pressure, especially when combined with a large or adversely shaped grout mass.

6B.9 PLANNING A GROUTING PROGRAM

All too often, grouting is considered only after a problem has developed, with the work proceeding hastily and lacking rational planning. Whereas grouting can often solve soil problems effectively, it is an established technology, and should be planned just as any other geotechnical construction. The purpose of the grout injection, and clear objectives, should precede any injection. Pumping of grout

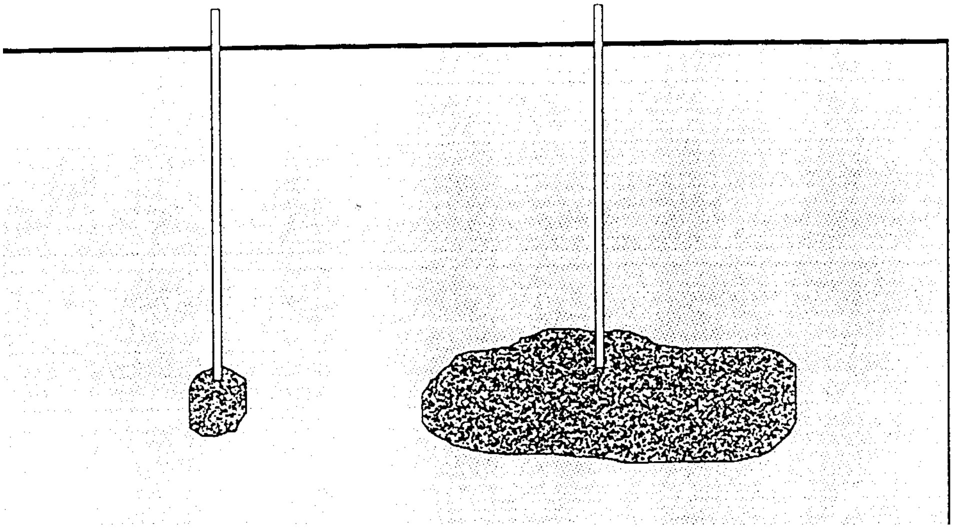

FIGURE 6B.143 A large grout mass exerts a much greater total vertical uplift force than does a small mass.

without a good understanding of the particulars of the site is risky at best. This is particularly pertinent on improved sites and especially where extensive subsurface improvements exist.

6B.9.1 What are the Objectives?

Clear objectives should be established as part of any grouting program. Why is the grouting being done, and what properties are expected of the improved soil? Is the required improvement to be permanent, or is it temporary, such as solidification of a cohesionless soil to enable tunneling or other construction? If the grouting is being done to increase the existing soil density, what density is required? How will it be confirmed? Should the grouting be done to reduce the flow of water through the soil, how much reduction is required, and under what conditions? The precise purpose and objectives of the proposed grouting program should be thoroughly considered and expressly stated prior to commencement of the work.

Where used under emergency conditions, such as reducing or correcting piping of water through an earth embankment or settlement around an active sinkhole, both the time available for consideration and the selection of methods and materials is often limited. The objectives and associated risks should nonetheless be considered. Blindly pumping any grout into the ground can have adverse results, and such risks and ramifications should be recognized and considered prior to the start of the work.

Unlike most construction, the planning of a grouting program should not end upon start of the work. A great deal of information, relative to the existing soil conditions, can be obtained during both the drilling and injection. In a sense, each grout hole can act as an exploratory hole, complimenting the original exploratory program. This will often suggest beneficial changes to the originally conceived program and will provide a better finished product or possible cost saving. In cases of remedial grouting, the writer has in many instances determined the actual cause of the problem for which the grouting was being done solely as a result of close observation of the drilling and grout injection behavior. In this regard, grouting contract provisions should always allow for and facilitate possible changes in the work, as dictated by the information gained during progress of the work.

6B.9.2 Access/Surface Improvements

Of all the available soil modification methods, virtually none can be performed with less disturbance than grouting. Although many, including a number of grouting contractors, opine that the procedure is inherently messy, such need not be the case. The writer has been involved in more than 100 projects where grouting was done inside structures that remained fully occupied and operational throughout the work. In many instances, normal operations were ongoing in areas only a few feet from the grout injection location. An extreme case is shown in Figure 6B.88, where compaction grouting is being done immediately adjacent to sensitive equipment in the Spalding Laboratory at the California Institute of Technology. Unlike most other remedial methods, large equipment is not required at the injection site, and that which is can be free of excessive noise, vibration, dust, and other annoying elements. Access or other restrictions will, however, affect the manner in which the work progresses.

It is important to consider the layout, construction, and condition of any existing structures when contemplating a grouting program. Obviously, any requirements for occupancy or continued use of the affected areas, must be considered. Drilling of grout holes in soil need not require large drill rigs, and in fact, some contractors routinely use hand-held drills for holes to depths up to 100 feet (33 m) or more. Most grout mixtures can be pumped a distance of at least several hundred feet, so the grout mixer and pump need not be located very near the area of injection. Provision must be made for routing of the grout delivery lines; however, they can be run over or under any obstructions or through areas that must be maintained in operation. Where walls or other solid elements block access to the injection site, small holes can be cut through them for access of the grout hose and other utilities.

Work inside structures or where cleanness is of particular importance does result in increased costs, but the increase should usually not be great. When grouting is being considered under such conditions or in other sensitive locations, consultation with experienced grouting contractors with a proven reputation for orderly work is advisable. The condition and especially any defects within the structure should obviously be observed and noted prior to commencement of the work.

6B.9.3 Subsurface Structures and Utilities

Good field practice dictates that a careful inspection of the area, and the proposed grout hole locations in particular, be made prior to the start of drilling. Grout hole layout might disclose conflicts with existing improvements or underground substructure. In this regard, it is especially important to locate any existing conduits, pipes, or drains. Electrical and communication conduits usually run in fairly straight lines between switches, outlets, or other exposed features. Placing of grout holes in such locations is not advisable unless the exact line locations have been positively identified.

The location and proper functioning of sewers and drain lines should also be established. Even where the location is known, it is a good idea to run water into the system, from upstream of the area to be grouted, and observe its flow at a downstream location, such as a clean out or manhole. In some cases where downstream access is not readily available, excavation to the top of the pipe and selective opening of a hole therein might be advisable. Where grout holes are located near such lines, continuous running of the water during grout injection, combined with occasional observation of the downstream opening, is prudent. Should grout be entering the line, a change in the color of the water will be observed prior to any extensive filling with the grout. This will allow relatively easy clearing of the grout by immediate flushing with water.

Drilling should also be carefully monitored as it can disclose unanticipated objects, especially if the work is in or near a structure. Gravel is often used to provide drainage or as bedding for pipes. Penetration of such gravel could indicate existence of a subdrain or pipeline. Encountering unantic-

FIGURE 6B.144 Much can be learned from observation of the drill cuttings.

ipated concrete is always of concern. It could be an underground structure or encasement for conduit or piping. Encountering organic matter at unanticipated depth is usually significant and can often indicate the cause of settlement, leakage, etc. By careful observation of the project site and structures and retrieved drill cuttings, as illustrated in Figure 6B.144, and continuous vigilant observation of the injection behavior, a great deal can be learned about the foundation soil. In cases where faulty conditions are being remediated, such diligence often provides a greater understanding of the cause of the problem that the grouting is intended to correct.

6B.10 CONDITION ASSESSMENT/GEOLOGICAL CONSIDERATIONS

Injection of grout into soil, absent a good understanding of the existing geology or soil conditions, is risky at best, and in some cases can have serious negative consequences. Grout injection will often affect movement of water through the zone of treated soil, and can in some cases, especially where chemical solution grouts are used, change the groundwater levels. This can result in changes of the state stresses within the soil, which might affect their future performance. Additionally, all grouting adds weight to the treated soils. The magnitude of the increased body load can be significant with some grouting methods such as compaction grouting. It is thus crucially important to assure that any soils underlying the proposed grouted mass are capable of supporting the additional weight of the grout.

Where grouting is being considered in regard to structural settlement, a level survey should be provided, with sufficient entries to enable a good understanding of the settlement patterns and differentials. Where the structure has a slab on grade floor, it is especially useful to provide relative elevation contours thereof, as illustrated in Figure 6B.145. This can be readily accomplished with use of a manometer system (water level). In such an operation, a reservoir of water or other fluid is placed at a generally central location and appropriate elevation (Figure 6B.146). One end of a small flexible tube, usually ⅜ or ½ inch (10 or 13 mm) in diameter is connected to the reservoir, with the other end attached to a rigid rod that is provided with a ruler, as illustrated in Figure 6B.147. Fitting of a hardened sharpened point to the bottom end of the rod, as shown in Figure 6B.148, greatly reduces the time required for such level surveys, in that it can be poked through carpet or between sections of floor covering. This obviously negates the requirement for elevation correction calculations.

Information relative to cracking or other signs of structural distress should also be provided. Where significant cracking exists, it should be mapped prior to the grouting. Obviously, original construction documents, including any geotechnical reports, plans, photographs, and such, if available, should be reviewed and also be made available to the grouting contractor. Record or "as built" drawings can be useful; however, their accuracy should not be relied on. These documents are often produced only after construction has been completed, as a requirement for receiving final payment. The writer has witnessed a large number of instances where information provided on as built drawings proved to be grossly wrong. As a general rule, it is reasonable to hold the grouting contractor responsible for any damage to underground lines for which the accurate location has been provided. Conversely, it is not reasonable to for the contractor to accept responsibility for

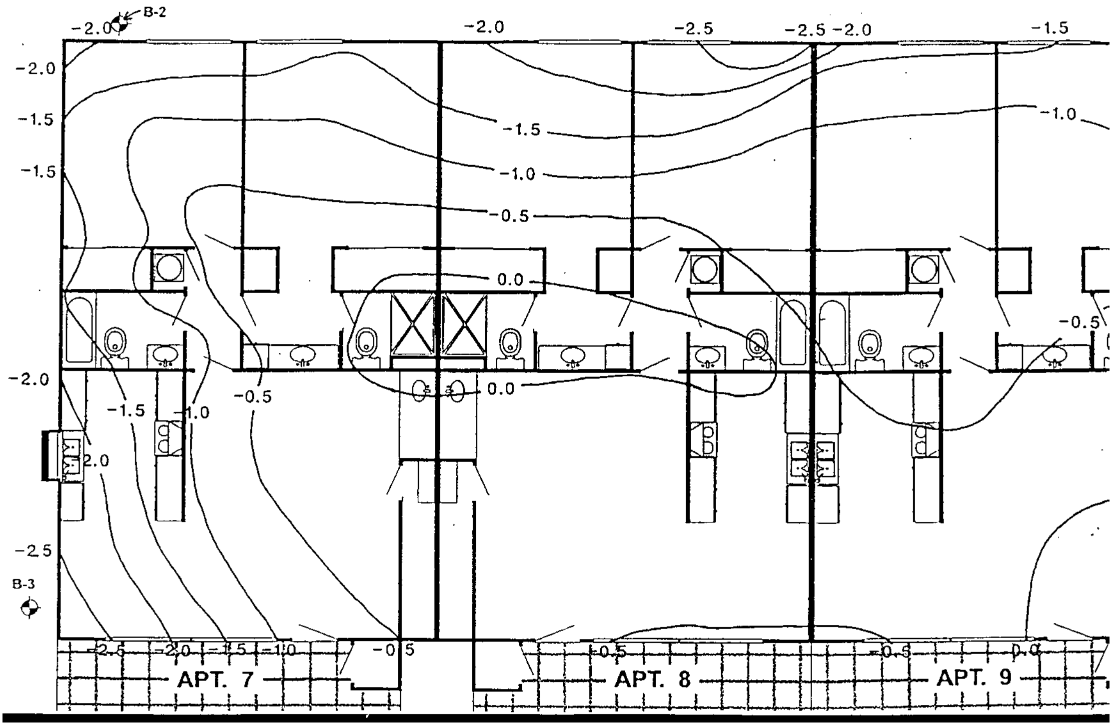

FIGURE 6B.145 Contour map of the mode and magnitude of settlement of a typical structure.

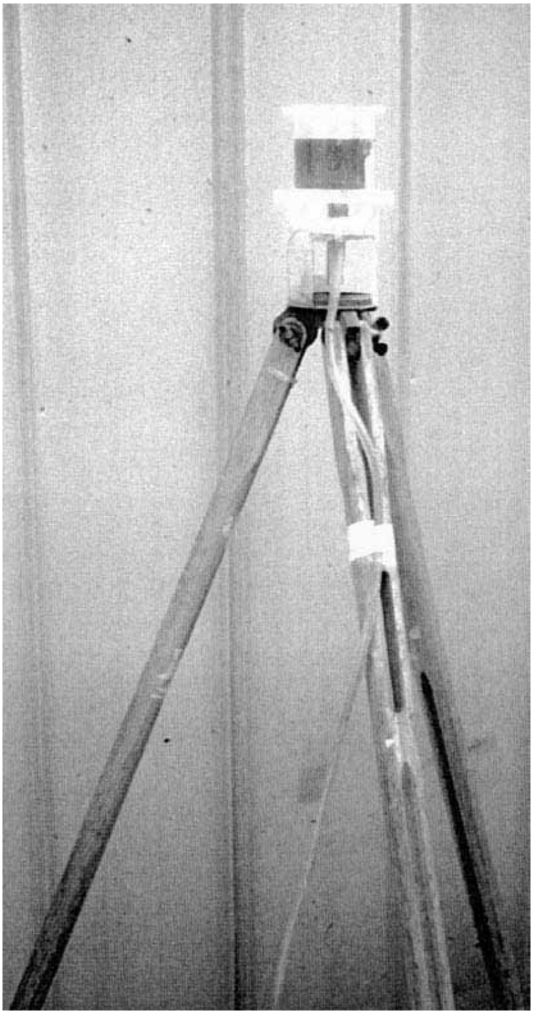

FIGURE 6B.146 Manometer reservoir is centrally located in the survey area.

damage to such improvements if their location is unknown or incorrectly indicated. Refer also to sections 7B.7, 7B.D, and 9A.

6B.10.1 Soil Type and Properties

Grouting is performed in soil for one of two reasons—either to repair some fault of the soil, or to change its performance. In either case, a rational grouting program cannot be formulated without a good knowledge of the existing soil properties, boundaries of the zones requiring improvement, and any faults therein or adjacent thereto. Selection of the grouting method, the particular grout formu-

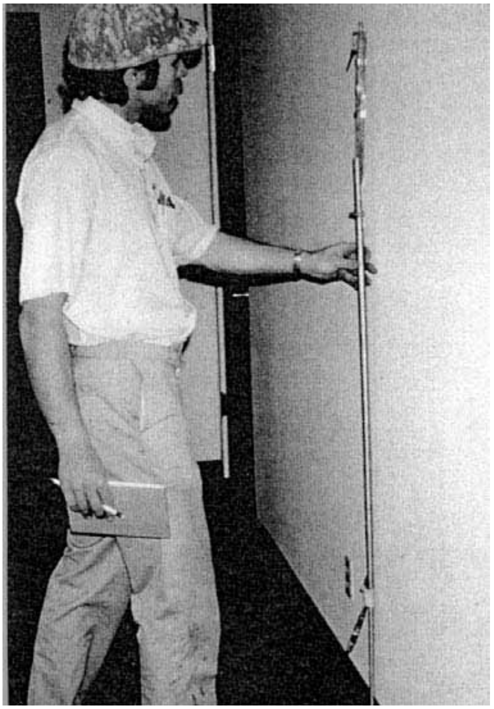

FIGURE 6B.147 Elevation readout is from ruler attached to rod.

lation to be used, and the parameters of injection are dependent upon the soil structure and condition. In some instances, more than one grouting method and/or material, will perform equally satisfactorily for a given condition, whereas in other cases only one will be clearly superior or appropriate. Effect of the grouting on the underlying and adjacent formation can only be determined with knowledge of the particulars thereof. Thus, unless the properties of the soils are well understood by prior experience, exploratory borings or excavations, penetrating not only the areas to be improved, but those adjacent as well, and most importantly, those underlying the zone to be grouted, should always be assessed. A thorough investigation for a grouting program is an activity that is almost always beneficial to proper planning and engineering. Such performance nearly always *pays*, in lower ultimate cost and greater effectiveness of the work. In the long term, *it does not cost*.

A sufficient number of borings or excavations should be made to enable a reasonable, basic understanding of the site soils. Reported data should include the field soil classification, consistency, moisture content, groundwater elevation, sampling method, and Standard Penetration Test (SPT) or Shelby tube blow count. The frequency of sampling will be dictated by the amount of soil variation within the borings, the objectives of the proposed grouting, and the degree of advanced certainty as to grouting costs required by the client. As previously discussed, once injection commences, every grout hole provides further information about the in situ conditions. Sufficient sampling must be provided before grout injection begins, however, to enable determination of the most appropriate grouting program and the general method and materials to be used. Obviously, the location and dimensions of the borings and the drilling and sampling methods used should also be provided.

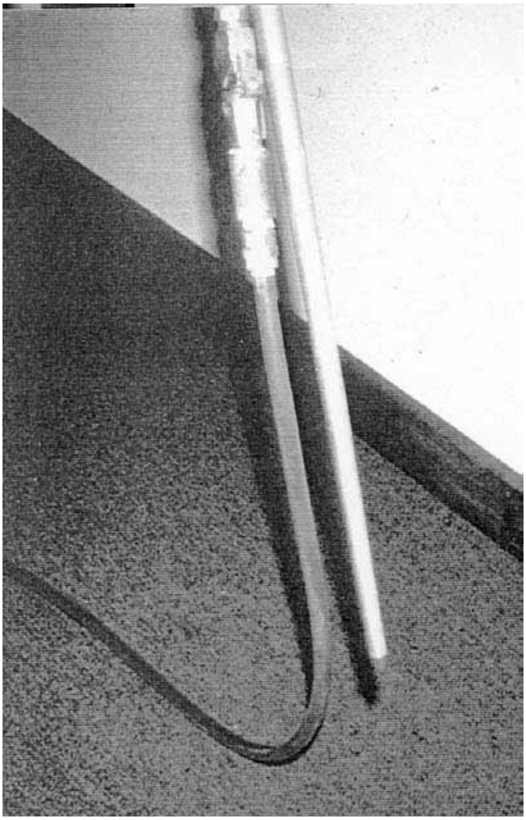

FIGURE 6B.148 Sharpened steel point eliminates correction calculations necessary for carpet and many other floor coverings.

Where the purpose of the grouting is to strengthen the soil or increase its bearing capacity, Cone Penetration Tests (CPTs) are often faster and less costly than SPTs in conventional borings. They can provide good data and are very useful. On most sites however, at least one conventional boring should be provided to allow for visual inspection and classification of the soil and provide specimens for laboratory evaluation. CPTs do provide an excellent means of evaluating the comparative condition of the soil, before as well as after the grout injection, and are especially useful for verifying the increase in density achieved by compaction grouting.

The purpose of the grouting will dictate the necessary laboratory tests. Where permeation grouting is anticipated, both dry density and grain size analysis should be provided. Where the quantity of minus 200 mesh fraction is significant, hydrometer tests should be performed so as to further classify that material. Permeability determinations are useful but it has been the writers experience that laboratory measured permeability is seldom reflected by actual field injection results. He attributes this to the inability to reasonably duplicate the in-place soil structure in remolded laboratory specimens and thus does not rely on them much. In this regard, full-scale field permeability tests should be performed in those instances where the penetrability of the soil must be known. When densification of fine-grained soil is expected, both Atterberg limits and consolidation test data are useful. In those instances where significant variation of the soil properties are found, frequent sampling and abundant laboratory testing are in order.

6B.10.2 Formation Stratigraphy

When permeation grouting is contemplated, it is important to understand the detailed stratigraphy of stratified soils, as this factor can have a dramatic impact on the grout distribution and penetration. The effect of stratigraphy is vividly shown in Figure 6B.149, which shows an extricated solidified mass resulting from the test injection of ultrafine cement grout into a sandy soil. The clear boundaries of the various strata and decreasing radial penetration of the grout with depth can be clearly observed in the lower portion of the grouted mass.

Grout will tend to flow to and through the more permeable soils, often completely missing those of lower permeability, unless special injection efforts are made. Also, grout penetration can be retarded and even completely stopped by layers of clay or other very low permeability soil. Even very thin seams of clay can block movement of the most penetrable of grouts. When injecting grout below the water table, the groundwater must be able to escape at a rate at least equal to that of the grout placement, and at the driving pressure of the grout.

It is not unusual for faulty fine or silty sands in need of grouting to exist over deposits of much more permeable clean and/or coarse-grained materials, which although quite adequate and not in need of improvement are significantly more permeable. There have been many instances where a chemical solution grout intended for permeation of the upper layer actually traveled to the underlying more permeable material and failed to solidify the soil zone needing improvement. Well-recognized injection techniques are available that can prevent the misapplication of grout in such cases, but the stratigraphy must be understood prior to the start of injection.

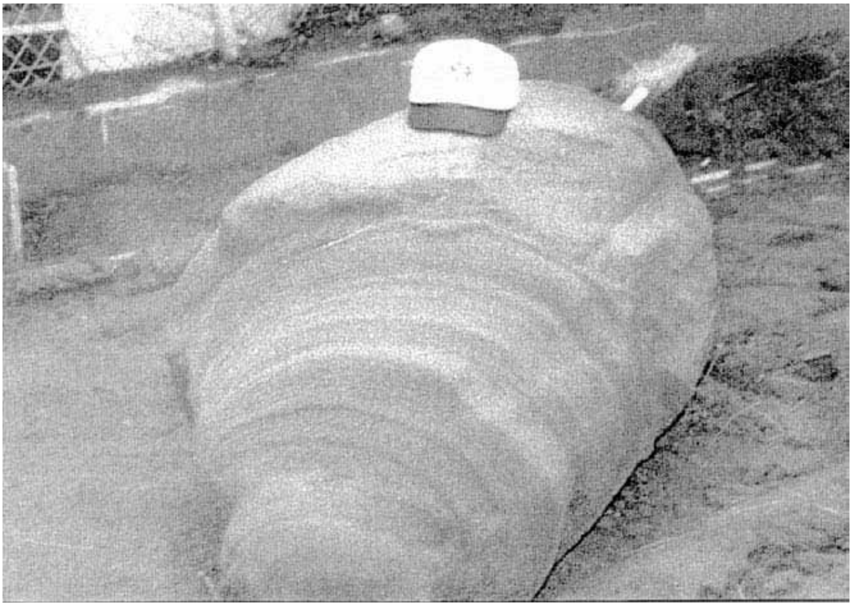

FIGURE 6B.149 Extracted solidified mass from test injection showing effect of soil stratigraphy on the effective radius of grout injection.

6B.10.3 Groundwater

As previously discussed, the level of the groundwater can be important in the design of a grouting program. Chemical solution and fluid suspension grouts can be diluted by existing water in the formation. If that water is moving, the grout can be washed away from its intended zone of deposition. The setting time of most grouts is affected by temperature, and differences of temperature can exist above and below the groundwater elevation. In rare instances, unusual chemistry of the groundwater can affect the setting or hardening of the grout.

Depending upon the grout injection rate and pressure, the pore pressure in the vicinity of the injected mass might increase, which could raise the groundwater level, at least temporarily. This is why the ability of the water to drain from the zone of injection often dictates the acceptable injection rate. When grouting in sensitive situations, it is sometimes useful to monitor the groundwater level during and following the grout injection. If the water level is permanently raised as a result of the grouting, water movement at depth can be affected. As an example, a substructure that was leak free at the pressure head exerted from the original groundwater elevation before grouting began might leak at the increased head resulting from the higher water table elevation.

Compaction grouting is relatively unaffected by the presence of groundwater, as long as the soil is able to drain at least as fast as the grout is being placed. There could be an increase in the localized pore water pressure, which is entirely satisfactory as long as the soil continues to exhibit drained behavior. Should the pore pressures become so excessive that undrained behavior of the soil occurs, loss of control of the injection and hydraulic fracturing of the soil, as shown in Figures 6B.42 and 6B.46, would likely occur.

6B.11 INJECTION MONITORING AND EVALUATION

The effectiveness of the grouting is often made obvious by the stoppage of visible leakage of water or jacking of a settled structure to proper grade. Verification of the results of much grouting, however, is not so obvious. As an example, where otherwise running soils have been solidified by permeation grouting to facilitate tunneling, it is desirable to know the completeness of the solidification prior to the tunnel heading reaching the grouted zones, which may not have been satisfactorily treated. Likewise, where the purpose of the grouting is strengthening of the soil at depth, where it cannot be directly observed, verification of the grouting effort is needed.

The completeness and quality of a grouting program begins with appropriate preinjection planning and a clear knowledge of the soil to be improved. It continues with careful and accurate monitoring of the work at all times during injection and indicated modification of the work as dictated by the observed behavior of the grout and the site of the work during grout injection. If these matters have been conscientiously attended to, the likelihood of a high-quality finished product are virtually assured. If, on the other hand, as is all too often the case, injection begins without a good knowledge of the soil, and little or no monitoring of the work occurs, less than quality performance is virtually certain.

In earlier times, monitoring of grout injection behavior was often made with methods and systems that lacked accuracy, and the reported results were usually subject to unavoidable delay. This made quick corrective action difficult at best and virtually impossible in many instances. Further, in the United States, designers and owner representatives have often relied on the grouting contractor to monitor and report on the work. This results in a delay of reporting and, sadly, experience has produced often questionable data. With modern computer technology, the earlier limitations have been eliminated. With present technology, there is no excuse for not properly monitoring the work and making timely adjustments so as to obtain a predictable quality product.

It must be realized that subsurface conditions do vary, and practicality and economics will virtually limit our understanding of the details of the beginning soil. Unknown or changed conditions, however, will be indicated by the grout behavior. If timely and appropriate monitoring and reporting procedures are employed, required changes can be made in an effective manner, without extraordinary increased costs. These can assure that the intended purpose of the grouting is being achieved and increase the knowledge of the starting soil conditions. It is recognized that there are many smaller projects where computer monitoring may not be practicable. In such cases, careful manual records should be kept and evaluated in a timely manner.

Continual observation of the ground surface and both overlying structures and substructures is imperative when grouting methods are used that might cause soil deformation or upheaval. With, compaction grouting in particular, a slight heave of the surface is often a criterion for limiting grout injection at a particular location. It is important in such instances that the upward movement in any one stage be very small, as the accumulation of many grouting stages can be considerable. In all grouting that is performed in urban areas, or where substructure might be present, careful observation is imperative to avoid leakage into or displacement of the substructure.

6B.11.1 Injection Behavior

Much has been said previously, relative to the importance of continuous monitoring, of the grout pressure behavior during injection. As the cost of the work is directly linked to the rate of grout injection, it is advantageous to all parties involved with a project to use as high a pumping rate as practicable. With the knowledge that too high a rate will result in hydraulic fracturing of the soil, however, it is quite acceptable to slowly increase the pumping rate at the beginning of injection, until a sudden pressure drop occurs, which indicates the initiation of a fracture, and thus the practical maximum pumping rate that can be used safely. The rate should then be reduced slightly, to a point that no further sudden pressure drops occur. Ideally, the pressure at each grouting stage should increase slowly and steadily, as shown on Figure 6B.150, until injection of that stage is complete. Should the rate of pressure rise be too fast, or should it increase abruptly, the pumping rate should be immediately reduced and adjusted so as to maintain a slow uniform rise, as illustrated in Figure 6B.150.

Conversely, should the pressure level remain static or slowly deteriorate during injection, it is likely that the pumping rate is too low. Of course, if the pressure drop is sudden, fracturing of the soil or leakage of the grout to an unwanted location is indicated, and immediate reduction of the injection rate or complete cessation of injection is in order. Although occasional rapid pressure reductions should be expected on most grouting jobs, their occurrence should be infrequent, and immediate corrective actions should be the norm. When very sensitive work is being performed, such as injection into a water-retaining embankment where even minor hydraulic fracturing of the soil cannot be tolerated, abnormally low pumping rates are in order. On such sensitive applications, ample instrumentation and monitoring of the grouting site should also occur. Of special importance in this regard is monitoring of the pore water pressure. Unacceptable increases thereof can be controlled by slowing the pumping rate.

Figure 6B.151 is an actual record of the injection behavior of a continuously computer monitored compaction grouting operation. The pressure spike occurring at the beginning of the record is the elevated pressure required to start the flow of grout, following a short time at rest for removal of a section of casing. This is normal and the result of thixotropy (cohesion) of the grout, as illustrated on Figure 6B.17. The nearly complete pressure losses indicated are the result of pulling of the casing to the next higher injection stage, as is clearly indicated by the rise in total depth of injection shown on the bottom curve. The smaller dips of pressure during individual grout stages are indicative of the pump strokes. Note that these line up with the increases in grout quantity shown on the grout volume curve.

Real time records that show all the injection parameters, as in Figure 6B.151, provide a good knowledge of the grout deposition and potential effectiveness. Hydraulic fracturing, loss of control of the grout, or other undesirable events would likewise be indicated by a sudden pressure change, assuming the pumping rate was uniform.

In permeation grouting, it is usually prudent to place a limitation on grout to be placed at any

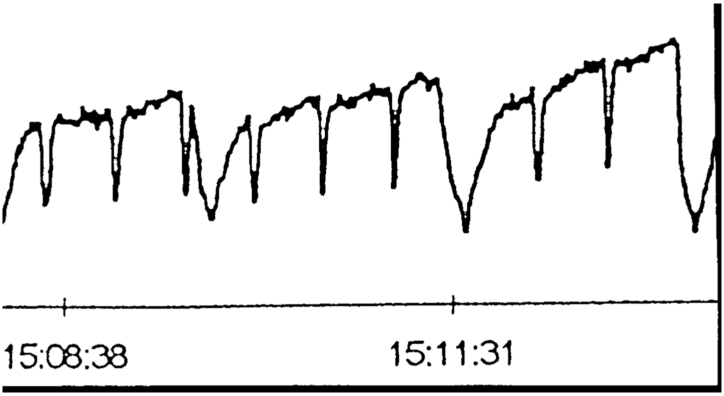

FIGURE 6B.150 Proper slow, steady increase of injection pressure from actual project record.

one stage. In those rare instances where unreasonably large quantities of grout are taken at a number of holes, or stages of holes, the spacing is likely too close. For economic reasons, the greatest spacing of grout holes that will allow the intended results should be used. Thus, firm establishment of the grout hole spacing is best delayed at least until initial injection takes place. Even on the same site, it is not unlikely that more than one hole spacing will be found to be optimal.

Whereas the pressure behavior at a constant pumping rate discloses a huge amount of invaluable information relative to the formation being treated, knowledge of the pressure, absent that of the injection rate or variations therein, is of absolutely no value. The widespread practice of reporting only the highest injection pressure reached on a given grout hole, without any information relative to the injection rate or pressure variation, is unfortunate, although many professionals who should know better have blindly accepted such data.

Experienced grouters, and especially grout pump operators, know that if it is desired to reach a given high pressure, the only action that is required is a sudden increase in the injection rate. This being so, high pressures are often inappropriately created and are reported far too often, usually to the economic benefit of the contractor. Relative pressures and injection rates are very instructive however, and should be routinely monitored and reported in real time. This can be effectively done either manually or with automated recorders or computer processing.

6B.11.2 Site Surveillance

Virtually all grouting into soil imparts elevated pressures into the formation. This results in a continual risk of possible hydraulic fracturing, upheaval, and lateral spreading. Such deformations can continue into surface improvements or substructure, resulting in their damage. It is thus imperative that the ground surface and all improvements thereon be continuously monitored while grout is being injected. Likewise, underground structures, and especially sewer and drainage piping, must be monitored to assure that they have not been displaced or entered by the grout. Most damaging movements of the soil or associated structures are immediately proceeded by a significant loss of pressure, as has previously been discussed. It is thus imperative that the grouting personnel are particularly observant of the work area upon any such sudden losses.

There are many methods available to monitor the elevation of the surface or any overlying structure. To be effective in grouting, however, the selected methods must be capable of surveying very

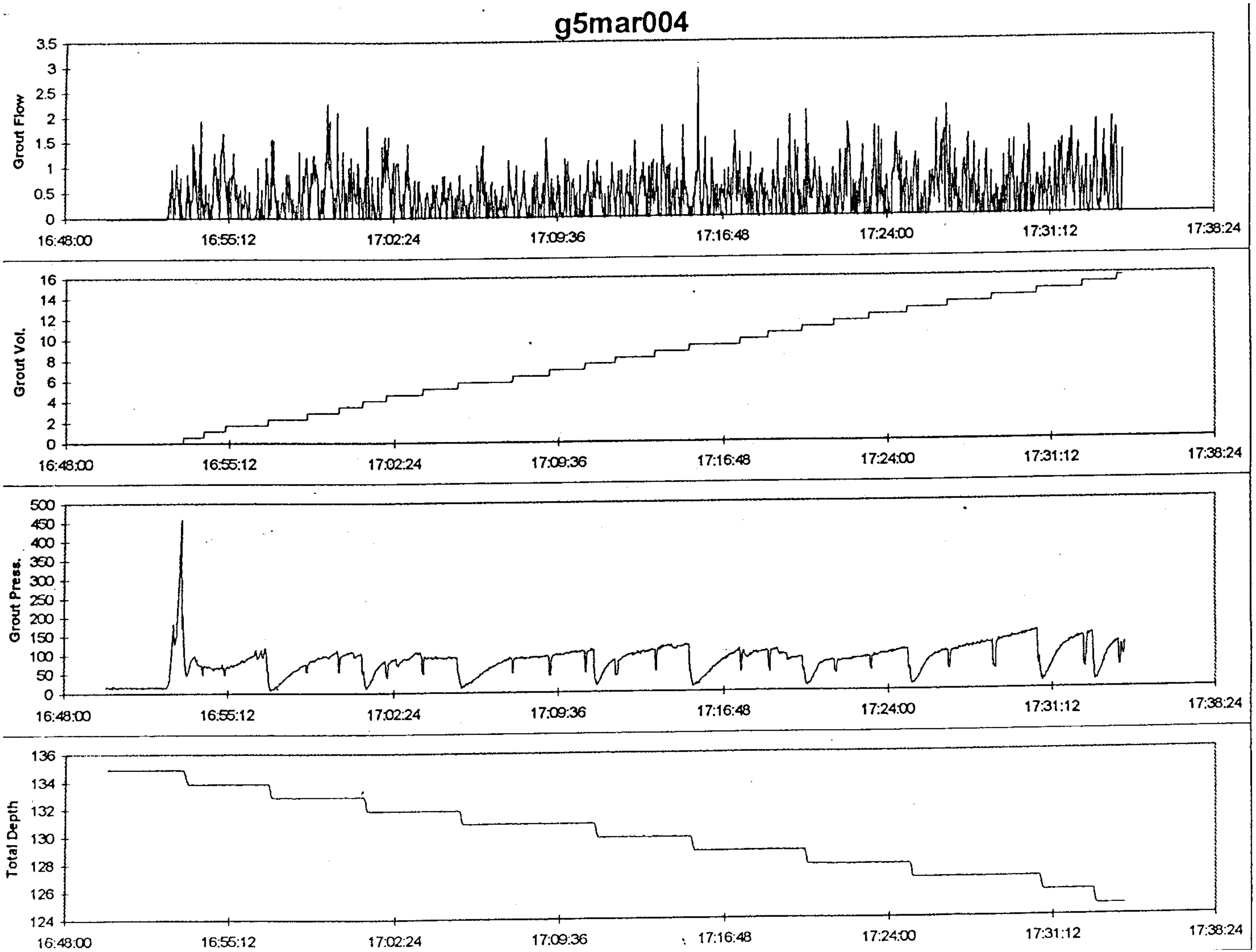

FIGURE 6B.151 Computer-generated record of grout injection parameters. Note generally constant but slowly rising grout pressure.

large areas and providing immediate notice of any movements. Thus, whereas conventional surveying can be useful, it usually is not satisfactory for the primary surveillance. Rotating laser instruments (Figure 6B.152), combined with multiple targets (Figure 6B.153) mounted on stands that are distributed throughout the work area are quite effective.

Another method, which has been used on literally thousands of projects, is the multistation manometer. Therein, one or more reservoirs, such as illustrated in Figure 6B.146, are used. Any number of tubes can be connected to a single reservoir, with the terminal end fixed to the ground surface or on structures as desired. The precise water level is marked on the tube or adjacent mount prior to grout injection (Figure 6B.154). Any vertical movement that occurs will cause a change in the water level, which is readily observed. In order to enhance visibility, the water can be colored, which is easily accomplished with a few drops of food color in the reservoir.

Multistation manometers are extremely flexible, as is the manner in which they are used. Figure 6B.155 shows one of a number of tube terminals that have simply been attached to posts driven into the ground so as to be easily observable at normal eye level. A similar terminal, attached to small stands that can be distributed over the ground surface, is illustrated in Figure 6B.156. Note that the tube is fixed at an angle to the vertical, so as to facilitate reading from a standing position. Figure 6B.157 illustrates tubes from several different reservoir locations that are attached to a common terminal board, enabling surveillance of an extended area from one location. In Figure 6B.158 the terminal end of a tube is simply fixed to the wall of a structure with duct tape.

FIGURE 6B.152 The signal from rotating laser instruments can cover a large planar area.

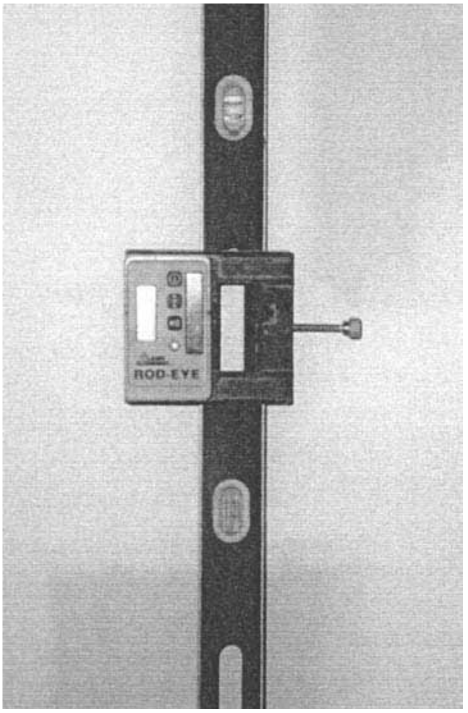

FIGURE 6B.153 Any number of laser targets can be mounted on stands distributed throughout grouting area.

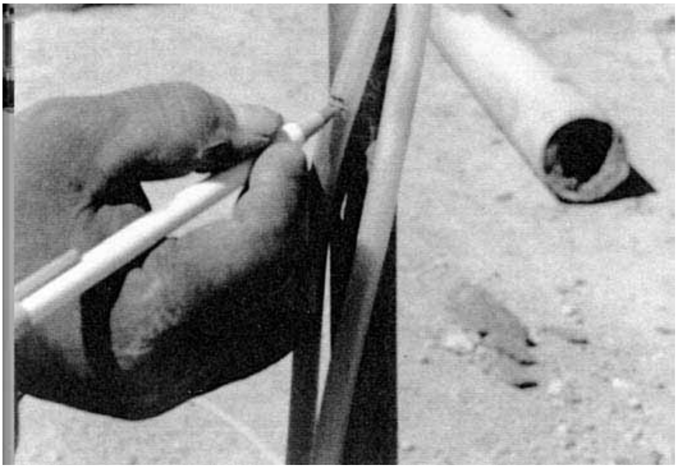

FIGURE 6B.154 The precise water level is noted when the instruments are set up.

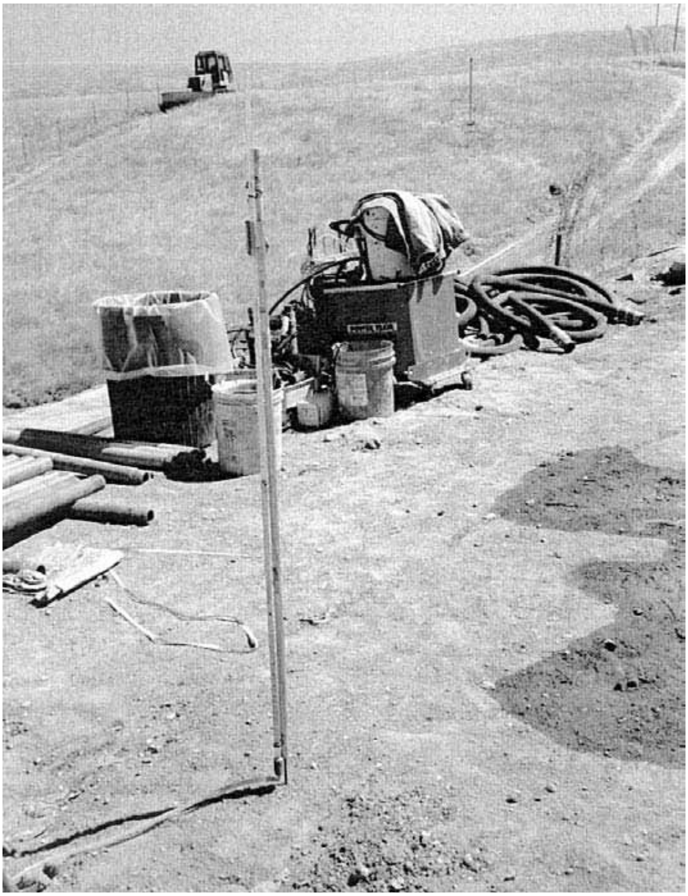

FIGURE 6B.155 Manometer tube terminal attached to fence post driven in ground.

Note the ruler that has been fixed with it, which will enable easy reading of the exact magnitude of any movement.

Although they do not necessarily provide immediate surface movement measurements, conventional survey instruments can be quite useful, especially for the scanning of large areas. Figure 6B.159 shows part of a large array of prisms that have been set into the slope of a dam. They are being constantly swept by a total station located in the small protective shelter shown in Figure 6B.160, in order to detect any movement of the slope.

Simple, inexpensive devices can also be very effective. A string line tightly stretched across an injection site can be used for reference of the surface elevations. Small pieces of masking tape fixed over cracks will either deform or break if there is movement of the crack. For evaluation of the exact amount of movement, rulers or crack monitor gauges can be secured over cracks in structures (Figures 6B.161 and 6B.162).

Nothing, however, replaces the eyes and ears of competent and alert grouting technicians. Figure 6B.163 shows a worker "sweeping" the ground surface of loose material so as to expose a hardened

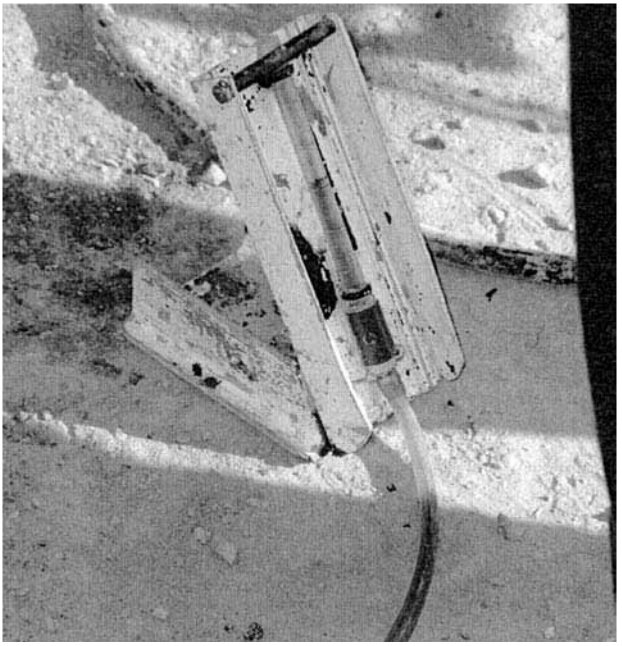

FIGURE 6B.156 Readout terminal mounted on small stand.

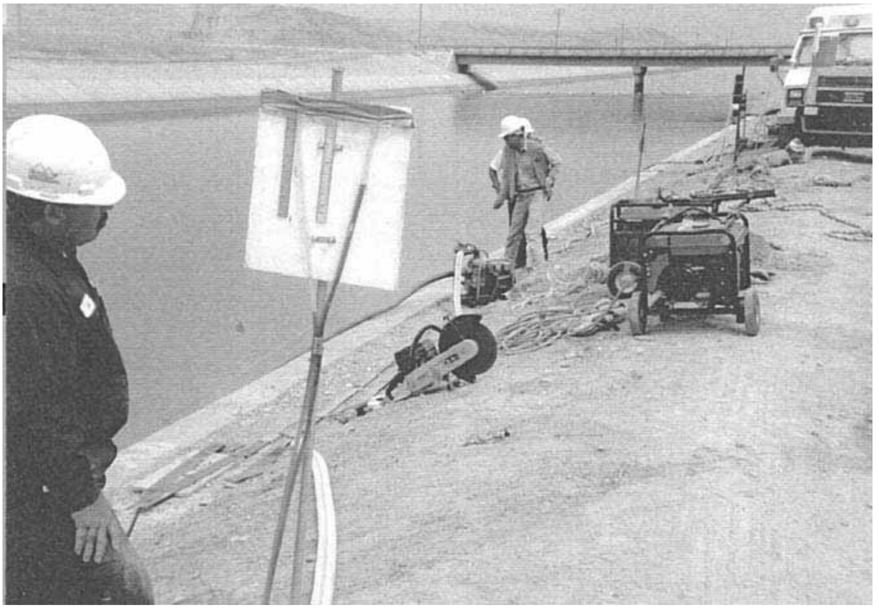

FIGURE 6B.157 Terminals from several reservoirs are placed together for easy readout.

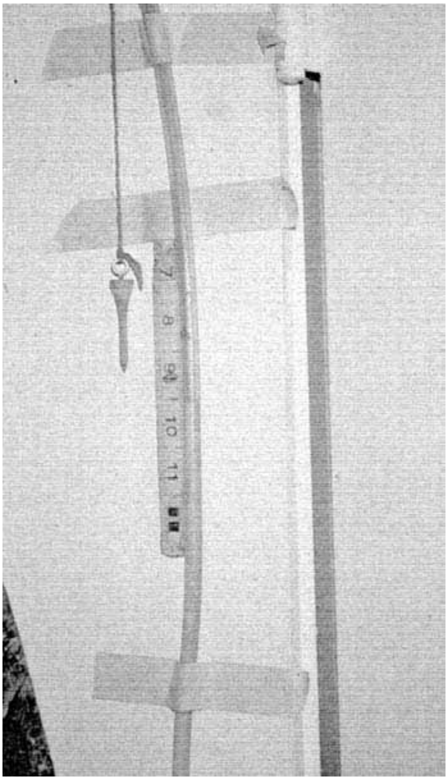

FIGURE 6B.158 The terminals can be attached to walls or other members of structures.

surface. By very close visual observation of the surface over the injection zone, members of the unusually observant crew were able to detect surface movements, even before they were indicated by the multiple instrument systems used.

6B.11.3 Injection Records

Maintenance of complete and appropriate records of all grouting operations is imperative if optimal results are to be achieved. Their greatest value is during the actual work, to provide a timely appraisal of the injection behavior, which indicates the actual conditions existing in the formation. This allows quick evaluation by the grouting personnel on site, as shown in Figure 6B.164, allowing appropriate changes for the execution of subsequent work to be made. As discussed throughout this chapter, flexibility of the work as it progresses is imperative, so as to allow adjustment of the grouting parameters, as required, for optimal achievement of the work objectives. It is also important to maintain the records long term, in order to provide pertinent information in the event of future work. And sadly, with litigation so common in the real world in which we all work, a good record of any operation is important, should that unfortunate event occur.

At a very minimum, the records should indicate grout pressure, the volume injected, and the

FIGURE 6B.159 Part of a large array of survey prisms covering the face of a dam during grouting.

FIGURE 6B.160 Prisms were constantly scanned from a survey total station inside protective shelter.

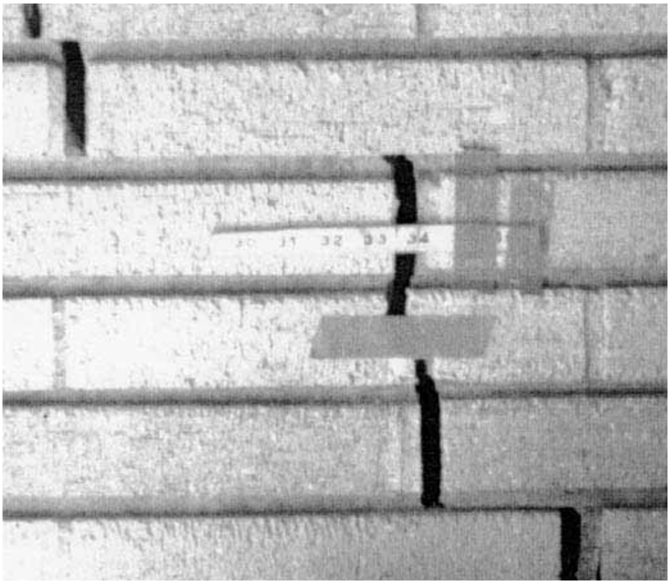

FIGURE 6B.161 Ruler fastened over crack in a wall to monitor any changes in its width.

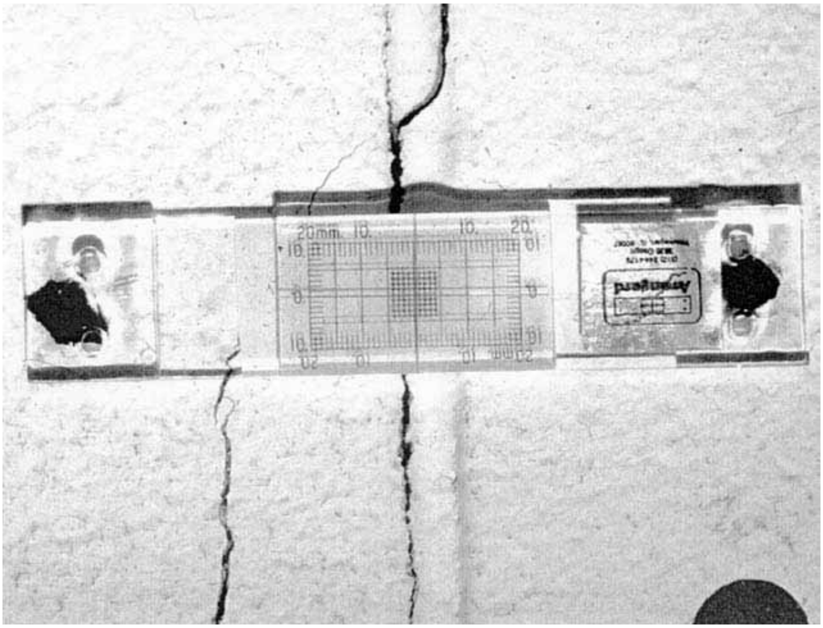

FIGURE 6B.162 Crack monitor can give very accurate measurements of any crack movement.

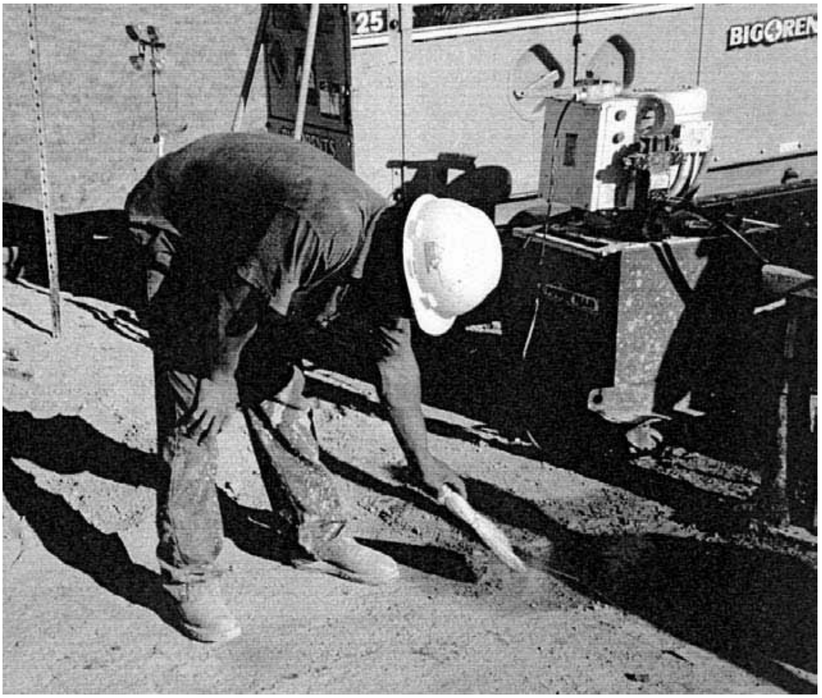

FIGURE 6B.163 Sweeping loose material off of earth surface to enable closer visual inspection of surface.

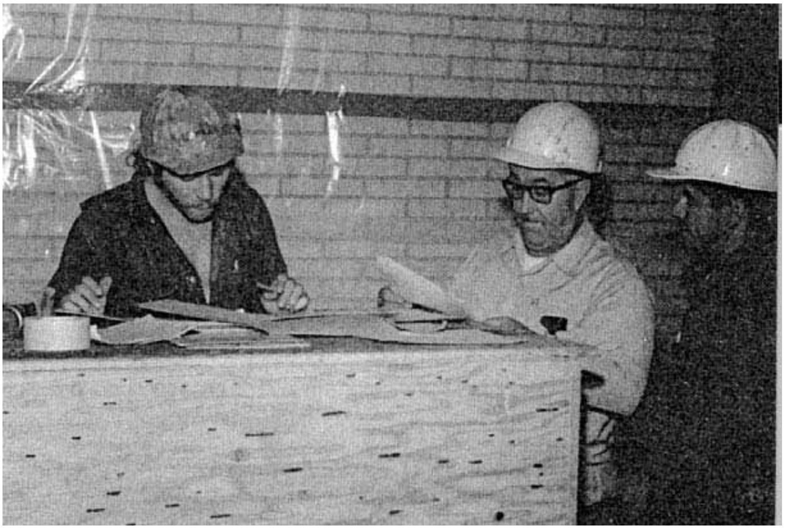

FIGURE 6B.164 Supervisors analyzing injection data as it is procured.

time at regular intervals. Obviously, the date, time, hole number, and grout stage depths should always be provided. Although it is convenient to also have the grout flow rate recorded, this can be back calculated if the time and volume are noted. Where suspension grouts are being used, continuous monitoring of the grout density will provide precise values of the ratio of water to solids. This can thus be an excellent quality assurance procedure.

If the records are being generated manually, entries should made at all pressure movements, such that the pressure behavior can be plotted against time. Figure 6B.165 shows a typical form for manual recording of individual grout holes. It is also important, to consider the highlights of all holes and their interrelation. This is conveniently done on a form such as that provided in Figure 6B.166, where the overall grout performance can be viewed. It has been the writer's experience that many grouting professionals tend to analyze in great detail the activity of individual grout holes, but sometimes fail to observe the trends of the overall performance. It is important to occasionally stand back and view the overall grouting operation. Understanding how the individual holes behave and interact often clarifies scatter in the individual hole data.

On large or important projects, either continuous chart recorders or automated data acquisition systems with direct connection to a computer should be used. Computer software programs are now available, and others are being developed, especially for the monitoring of grout injection. Also, some grouting specialty contractors have their own proprietary programs. A substantial advantage of computer monitoring is the ability to immediately output the data in a suitable form for closer observation or further analysis. Also, data formats that are computer generated can usually be adjusted in exaggerated scale or otherwise manipulated so as to aid in interpretation of specific events.

Even where the data acquisition is manual, it is advantageous to enter it into a computer database, such as Microsoft Excel, so that graphical output is facilitated. Figure 6B.167 illustrates bar graphs from one of the first holes grouted on an actual project. As can be observed, the grout take and pressure are shown side by side as a function of depth. It is noteworthy that grout takes were quite low at depths greater than about 28 feet, even though relatively high injection pressures were used. Following similar behavior on a number of distributed primary holes, a decision was made to reduce the depth of the drilling and grouting for the remainder of the work.

Similar data for a grout hole on another actual project is shown in Figure 6B.168. Here it will be noted that in spite of the injection rate and pressure remaining generally uniform, very large grout quantities were injected in a region at about mid-hole depth. By magnifying the grout volume figure, as in Figure 6B.169, the areas of high take are even more obvious. When several such records are assembled on the wall according to their relative positioning on the site, as illustrated in Figure 6B.170, it becomes very easy to recognize the relatively limited zone of high take and thus faulty beginning soil. The writer has found this format especially useful for displaying the data as it is developed on the jobsite. Records so posted, for each hole or hole stage, allow for even the lower level technicians to visualize the position of the overall grout deposition zones. This ability to quickly comprehend the overall project status is very helpful in determining how best to proceed. As an example, the posted data shown in Figure 6B.170 would suggest that depth of future holes need not be as great, as long as they extended through the zone of large take.

6B.11.4 Be Prepared for Changes!

Aside from the fact that geology and geotechnical engineering are not perfect sciences, from a practical standpoint, it is never possible to have a complete understanding of the subsurface conditions into which grout will be injected. As has been amply mentioned throughout this chapter, flexibility of a grouting program is thus mandatory if the optimal performance is to be realized. It is not unusual to find large grout takes in only a portion of the soil zone to be treated, as was the case in the project for which the grouting records are depicted in Figures 6B.168, 169, and 170. The ability to change both the grout hole spacing and grout deposition stage lengths and depths can be of great advantage in optimizing any grouting program. This is particularly true when working in older urban areas that have historically experienced considerable underground construction, for which the de-

PREPARED BY ______________ JOB ______________ JOB NO. ______

DRILLING	HOLE NUMBER ______ STAGE ______ DATE DRILLED ______		GROUTING		MATERIAL ______ SEQUENCE ______		HOLE NO. ______ DATE GROUTED ______
Depth	Material	REMARKS	Quantity	Rate	Press.	Time	REMARKS

FIGURE 6B.165 A typical form for manual recording of grout injection data.

SUMMARY OF DRILLING & GROUTING

PREPARED BY__________ DATE __________ JOB __________ JOB NO. __________

DRILLING					GROUTING						
HOLE NO.	DATE DRILLED	STAGE	PROCESS	REMARKS	DATE GROUTED	SE-QUENCE	DEPTH / FLUID	PRE-TREAT	C.F. PLACED	MAX. PRESSURE	REMARKS

PLATE __________

PLATE __________

FIGURE 6B.166 Placing all data on a summary form enables overall interpretation of operation.

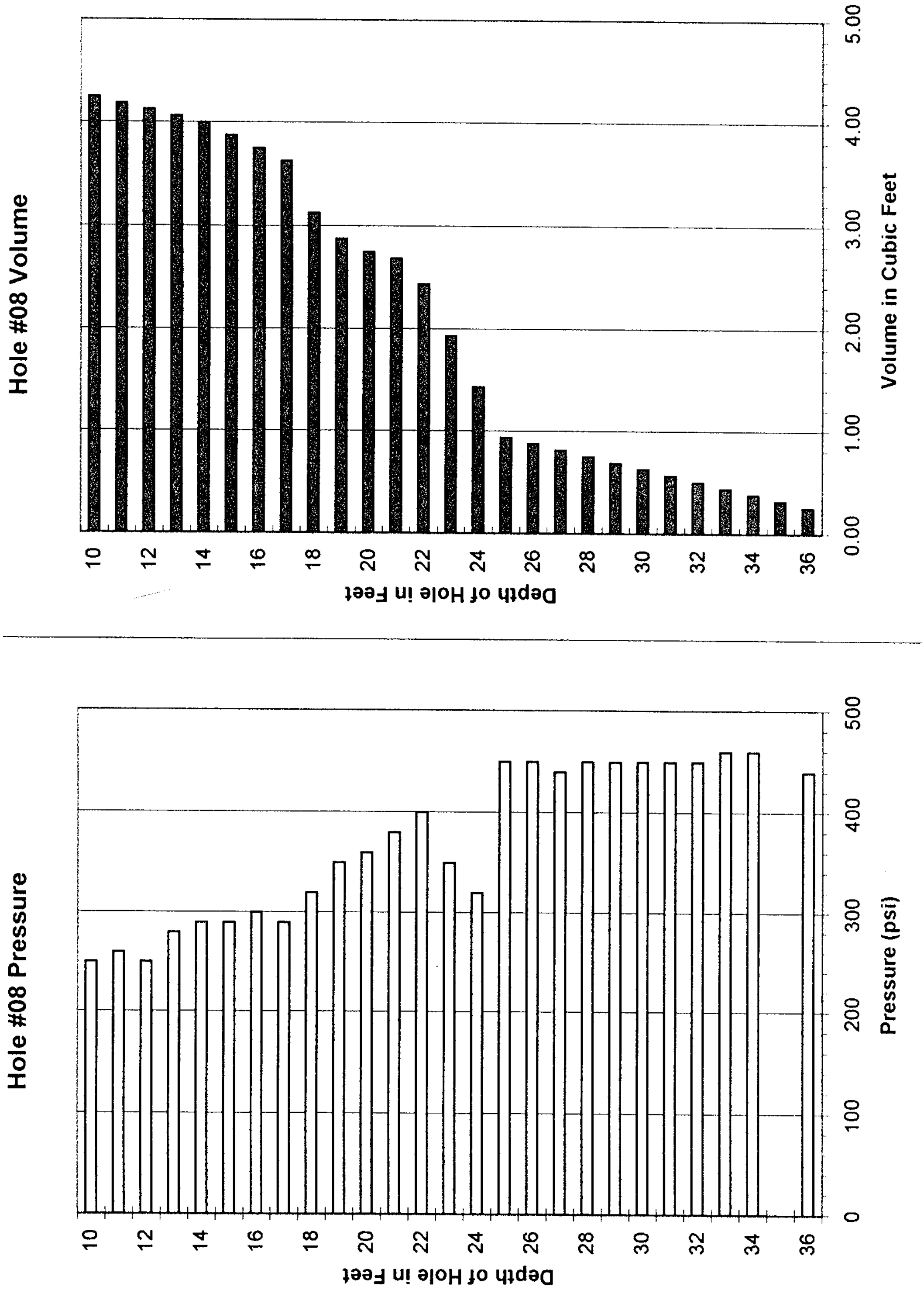

FIGURE 6B.167 Computer database-generated graph showing both grout volumes and pressures for a grout hole.

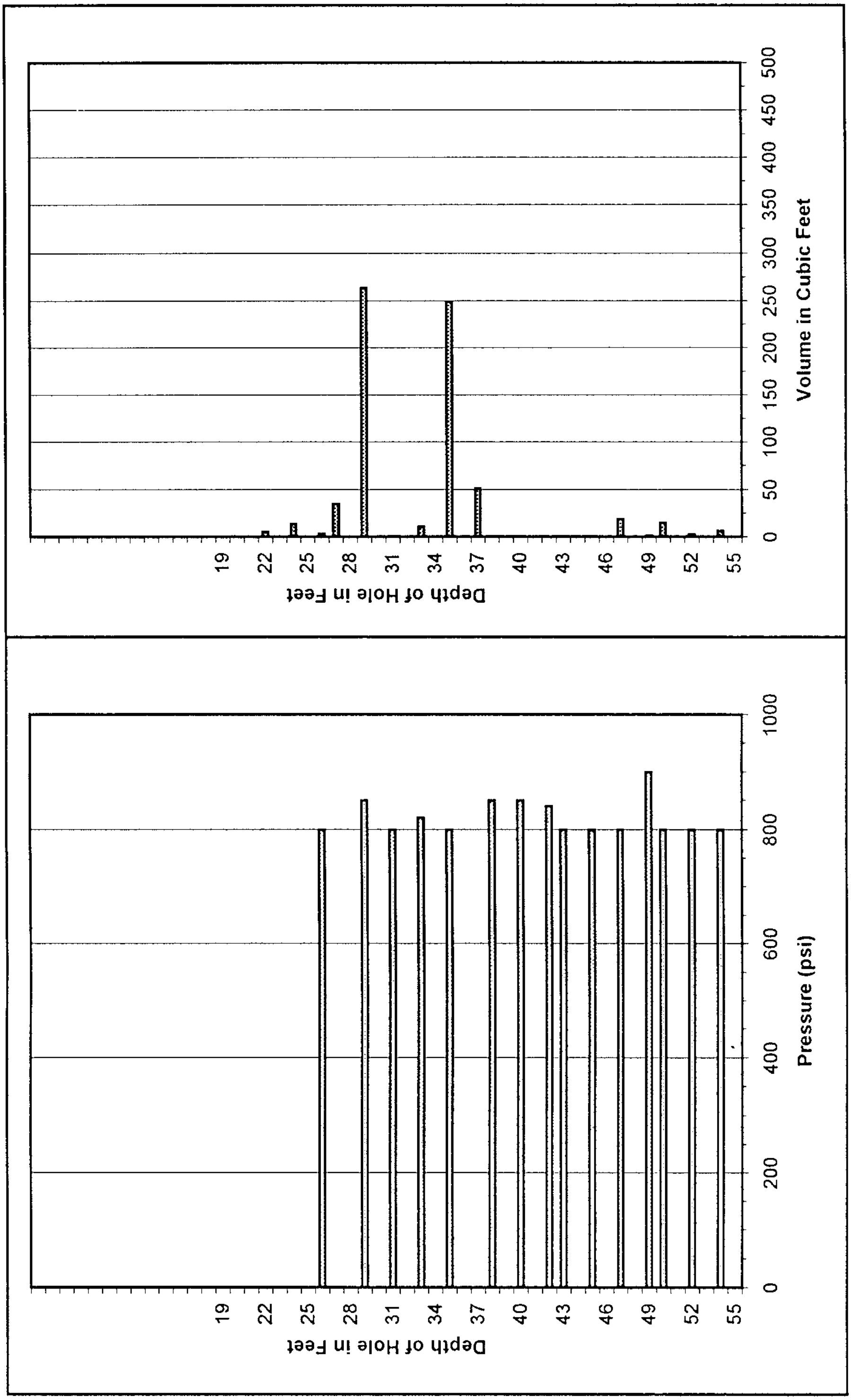

FIGURE 6B.168 Similar output graphs. Note large grout takes are limited to mid depth of holes.

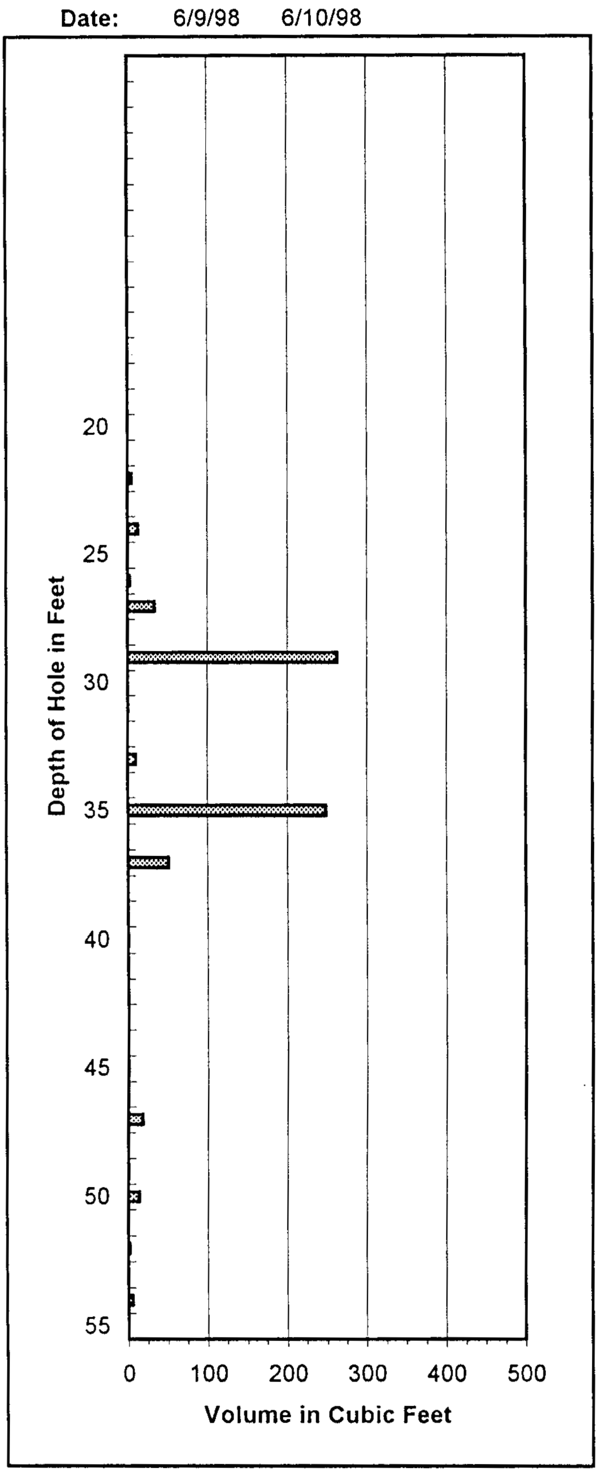

FIGURE 6B.169 Blowup in exaggerated scale of the right portion of Figure 6B.168 highlights grout distribution in the grout hole.

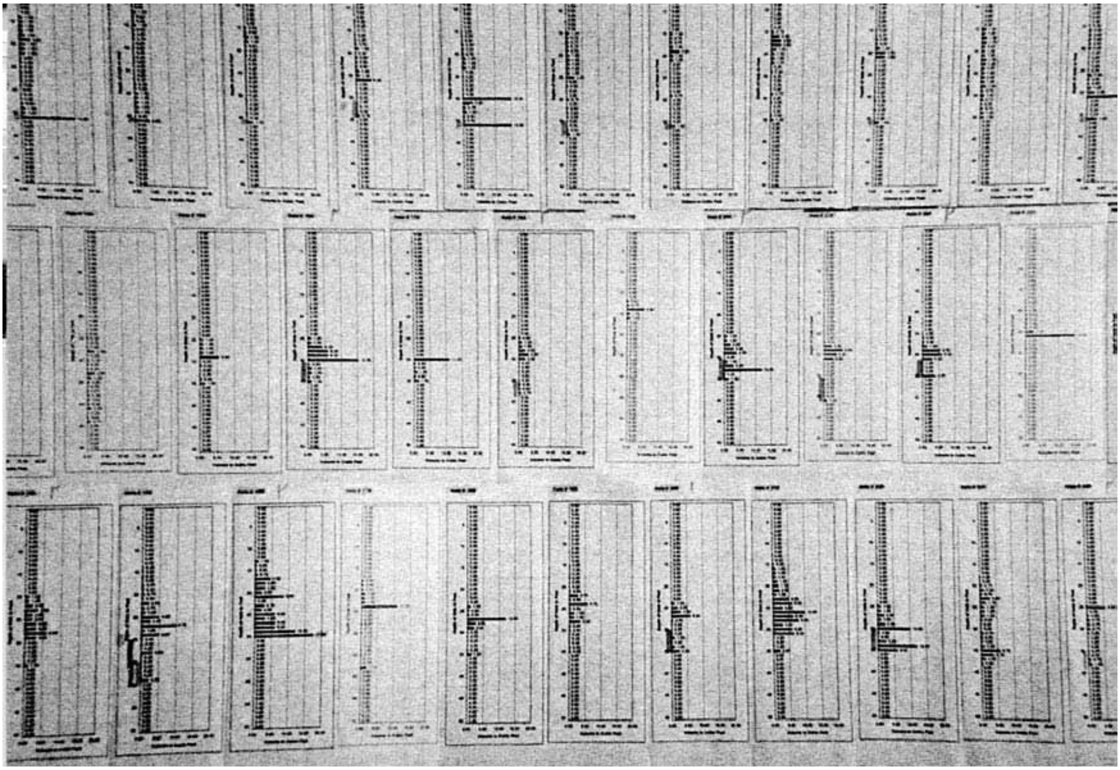

FIGURE 6B.170 Assembling volume graphs in the same layout as the grout holes allows easy recognition of the grout deposition locations.

tails are not known. Even in those cases where good records of prior underground works are available, important details may be missing.

As a very young contractor, in the mid 1950s, the writer had a job to re-level a settled swimming pool by grouting. The pool was constructed in clayey-silt soil, which filled the basement of a building that had previously occupied the site. Leakage from the newly constructed pool had saturated the fill soil, which had apparently not been properly compacted. At the time, grout improvement of such fine-grained soils was not considered possible, and the work was to be limited to leveling the pool shell. On commencement of grout injection, a steady flow of water entered the pool from adjacent grout holes. This continued for many hours before the pool finally began to rise as planned. About nine times the quantity of grout, which by calculation would have raised the pool as planned, had been required! And throughout the injection, what was thought to be a similar amount of water was bailed out of the pool.

The only logical explanation was that the force of the grout "squeezed" the water out of the soil, thereby densifying it. Although the job was a financial disaster for the contractor, who had agreed to do it for a lump-sum price, the geotechnical engineer considered it the most wonderful event of his career. You see, he explained, it's not supposed to be possible to squeeze water out of soil so easily; this is wonderful! And that was the beginning of the writer's research, which led to his current knowledge of the compaction grouting mechanism. Wonderful it was, though it didn't seem so at the time.

In another memorable project for the fledgling firm, remedial grouting in connection with the settlement of some railroad tracks was to be performed. Many years prior, a small tunnel had been excavated, about 30 feet under the tracks, in which a sewer pipe was placed. The soil was very sandy and although the records were sketchy, it was believed the tunnel was solidly shored with continuous sets of timber planks. The tunnel, which was only about 25 feet long, was said to have been backfilled with "jetted" sand. Payment for the grout, which was specified to be a cementitious suspension, was by the bag of cement. The contract quantity estimate was for 350 bags.

Penetration of about three inches (7.5 cm) of timber over the tunnel roof during drilling confirmed the expected shoring. A drop of the drill steel upon penetration of the timber, however, was a surprise. The contractor's people reported what they felt were voids, varying from a few inches up to more than one foot. The owner's inspector, who had not shown much regard for the novice grouter, however, insisted the contract estimate was correct, with a cryptic comment to the effect "when you get more experience, you'll know the difference between loose sand and voids."

Four thousand bags of cement later, the voids were filled. For the young contractor it was a very profitable job, and almost offset the losses on the pool! But more importantly, it offered a valuable lesson. Subsurface conditions are not always what we expect, and don't ever believe, you really *know* what you *think* you know (as did the arrogant inspector). Surprises do occur in the grouting business.

Another interesting example is in the failure of a four year old sewer, which had been constructed quite deep in an area of generally poor soils with a high groundwater level. Following the development of a sinkhole over the line, internal inspection disclosed some minor distortions of the invert alignment, suggesting questionable stability of the pipe bedding, which consisted of a 12 inch (30 cm) thickness of one inch gravel. The selected repair provided for injection of a cementitious grout into the pore spaces of the bedding gravel, to in essence solidify it into a rigid concrete. After a few days of uneventful injection, a large pressure drop occurred. The first thought was that the grout was entering the pipe, but inspection found that not to be the case. After careful evaluation determined that no other substructure existed in near proximity to the area of grout leakage, injection proceeded, but with very close observation of the pressure behavior. It was indicative of the filling of a small open pipe for about four cubic feet of grout, after which normal behavior returned. Subsequent discussions with the inspector of the original work disclosed that during construction the contractor occasionally embedded slotted pipe in the bedding rock, as part of the dewatering effort. No records existed of this detail, and had it not been for the memory of the inspector, the otherwise harmless anomaly would have remained a mystery.

This brings to mind a similar incident, which did not turn out to be so harmless. In the early 1960s, the writer's contracting firm was retained to do some remedial grouting under the main courthouse in Los Angeles. A tunnel had been excavated under the existing basement to provide access from a new underground parking structure. The tunnel passed under the heavily loaded slab-on-grade floor of an evidence storage area, by a depth of only about four feet (1.2 m). The floor slab had settled about three inches (7.5 cm) over the tunnel alignment. Slabjacking with a low-mobility grout was to be used to raise the floor to its proper grade, followed by compaction grouting of the soil down to the tunnel roof. Because this was in an active public building, all work was done at night. Grouting had a rather poor reputation at the time, and the owners' maintenance staff was concerned about possible damage to the existing facilities. They thus had a plumber present at all times during grout injection.

One night the grouting foreman observed pressure behavior indicative of the grout entering a pipe. He halted injection and informed the plumber that he thought the grout was getting into a pipe of some sort. On his manual record form was a comment, "stopped grout—told the plumber I think I'm in a pipe," with the time noted. About 10 minutes later, a further comment was entered, "Plumber says everything OK," with the time noted. Injection was resumed, but a further note after about ten minutes stated, "Told plumber know I'm into something—stopped grouting—hole abandoned." Following that were notes addressing further word from the plumber that he had checked all lines, they were clear, and there was no reason to stop grouting.

Some months later, when the first rain of the season fell, it was found that the grout was indeed in "something" and the plumber was wrong. A 10 inch (25.4 cm) cast iron pipe that served as a collector for the entire roof drainage system and crossed the tunnel alignment was filled with grout! In repairing that problem, it was found that the probable cause of the original settlement had been leakage from that pipe as a result of disturbance during tunneling. Fortunately for the contractor, the vigilance of an extremely competent foreman and his notation on the grouting records relieved him of responsibility for what turned out to be a very expensive event. As stated many times throughout this chapter, continuous pressure behavior monitoring and recording should be mandatory in all grouting work!

In another instance, differential settlement of several inches had occurred in a large commercial building. The building, which was only about a year old, had been built on one edge of a deep canyon fill. The geotechnical investigation did not reveal any obviously poor underlying soil, but as the fill was not up to the specified density, compaction grouting was called for. Primary grout holes were spaced about twenty feet on center, along the outside of the foundation of the wall that had experienced the greatest damage. Drilling indicated nothing noteworthy, and only minor amounts of grout, with very uniform pressure buildup, were injected in the first several holes. But then came a surprise! At a depth of about 40 feet (12 m), one of the holes exhibited very irregular pressure behavior, and took a very large quantity of grout. This was followed by similar behavior at a depth of about 35 feet (10.5 m) in another hole.

Reevaluation of the available site grading data suggested that a pioneering haul road might have treaded up the canyon under the building. Working on this assumption, grout holes were established over the suspected haul road location. Indeed, large quantities of grout were taken in only one or two stages, at various depths, in those holes. Plotting those locations indicated a uniform gradient, typical of a haul road. Apparently, loose slope-wash and fill existed on the downhill side of the haul road. With this knowledge, the grouting program was reevaluated. Further work was limited to the area of the suspected haul road, reducing the cost of the work to less than half of that which was expected going into the job. This resulted in a very happy owner. Although the grouting contractor had the job cut short, he more than made up for the lost revenue by referrals and future work from the owner and his geotechnical engineer.

6B.11.5 Verification of Grouting Effectiveness

Much has been written and discussed about verification of the effectiveness of grouting, and in fact the American Society of Civil Engineers GeoInstitute Committee on Grouting had an entire session on the subject at a 1995 meeting, the proceedings of which are available (Byle and Borden, 1995). This notwithstanding, although many grouters talk verification as part of their promotion, the sad fact is that relatively little effort has been shown by grouters to really understand the quality (or lack of quality) of their work, especially in the United States.

From a practical and economic standpoint, the most meaningful verification of the results of virtually all grouting operations occur as the grout is being injected. This is through continuous observation and control of the various injection parameters, especially the rate of injection and pressure behavior. It is the writer's considered opinion that no amount of postinjection investigation or testing can duplicate the benefit of careful monitoring and control during the actual grout injection. There are however, a variety of postgrouting tests that can, to a varying degree, confirm the completeness and the general quality of the grouted mass.

In water-control grouting, if the leaking areas are clearly visible, such as leakage into an underground pipe or structure, direct observation of the cessation of the leakage is certainly acceptable verification. If the site of the leakage is not clearly visible, such as an impermeable curtain under a dam, downstream boreholes will be required.

Where permeation grouting has been used for strengthening of the soil, test excavations or borings can be made. The completeness of the solidification can thus be readily observed, and specimens of the grouted soil can be obtained for laboratory evaluation of the strength. It must be remembered that the fundamental strength, which is the strength under a continuous load, of solution chemical grout solidified soils is only a small portion of that indicated by most standard laboratory compression tests. Such masses should thus be subject to long-term creep tests, as discussed in Section 6B.3.1.1

For compaction grouting, wherein the grout remains in individual globule masses, density testing of the improved soil between the grout masses can be performed. Although in the early days of the procedure, split spoon specimens were often procured for laboratory evaluation, this is a time-consuming procedure that is not terribly practical, and is thus now seldom used. Likewise, Standard Penetration Tests can be made both before and after the injection has been performed, but this also is somewhat disruptive and time-consuming.

Cone Penetration Testing of the soil following grouting has proven to be a reasonable method to determine postgrouting density. Its one limitation, is the necessity of access for the relatively large testing equipment, so it usually isn't suitable for use inside or under structures. Where the results of compaction grouting are being evaluated, probes or tests are usually made midway between injection points, which theoretically would be the zone of least improvement. It is thus imperative to preserve the exact locations of the injection points, which often requires special effort. If the injection is through a paved surface, the hole locations will be easily observed. If the holes are drilled through a dirt surface however, they can become obliterated quite readily. It is thus important to provide markers such as the long lag bolts being placed in Figure 6B.171 or the flags as seen in Figure 6B.172. The normal operation of equipment, and dragging of hoses over the surface, which is inherent in grouting work, can easily destroy hole markers, so they must be quite tough and resilient.

A variety of geophysical methods have been promoted, and some have been successfully used, for the verification of all types of grouting. The most commonly used involves evaluation of the ve-

FIGURE 6B.171 Long bolt used to mark location of grout hole.

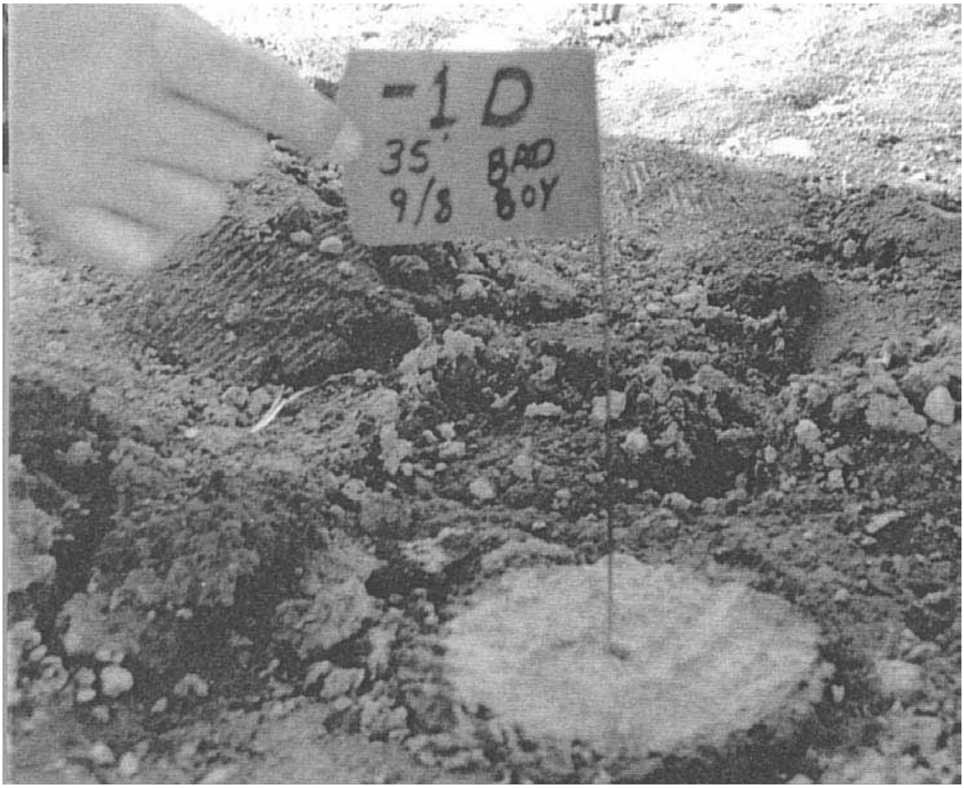

FIGURE 6B.172 Small surveyor's flag to mark location of grout hole. Note that the hole identification and date grouted are also shown, as is the comment "Bad Boy," which apparently indicates some difficulty with the grouting.

locity of stress waves traveling through the formation, commonly referred to as seismic tomography. Simply stated, sound travels faster through a hard medium than a soft one. Thus, a densified or solidified mass will usually possess a higher velocity than adjacent soil that has not been grouted. By analysis and interpretation of the velocity and attenuation of a large number of sound waves driven at different inclinations through a profile of the soil formation, the relative density or stiffness of the various components of that profile can be determined.

In application, evaluation is usually preformed between two bore holes. A transmitter in one of the holes initiates a pulse, which is picked up by a receiver in the adjacent hole. Typically, the transmitter will be moved vertically in the bore hole, so as to direct pulses to the receiver at a variety of angles. The receiver usually remains at one location within its hole for a given set of data. By changing the depth of the receiver, multiple sets of data are obtained, with which a seismic profile can be developed. The waves can be either shear waves, which are perpendicular to the bore hole, or compressional waves, which are parallel thereto. Compressional waves are the most frequently used, as their analysis is simpler. Whereas stress wave velocities are usually determined with direct paths between two bore holes (cross-bore hole survey) or by analysis of reflective rays, some work has also been accomplished by directing waves from transmitters and receivers, which are both placed on the ground surface. The effective depth of the so-called surface procedure is a function of the distance between the transmitter and receiver. As the accuracy of the resulting plots, which are known as tomograms, is dependent upon the number of individual ray paths and resulting nodal intersections that are evaluated, the down hole evaluation is preferred.

To produce tomograms from the ray path data requires extensive data processing, which is practicable only with specialized computer software programs. For these reasons, such work is commonly performed only by firms who possess the highly specialized knowledge and equipment. This, plus the requirement of very large numbers of different ray paths to provide accurate determinations, makes the procedure somewhat expensive, and it is thus usually reserved for especially large or important applications. Where the budget allows for acquisition of sufficient data, however, the procedure has been found to accurately portray the existing conditions and provide valuable information.

Another geophysical method, as reported by Rehwoldt, Matheson, and Dunscomb (1999) is two-dimensional imagery, generated by electrical resistivity surveys. Therein, the conductivity of the formation is evaluated by placing a consistently spaced line of electrodes on the ground surface. They are driven into the soil to a depth of about 12 inches (30 cm) and connected to a computer-guided resistivity meter. The meter takes measurements along the electrode array, using four electrodes at a time. The depth of the evaluation is dependent upon the spacing of the surface electrodes.

Special computer software is then used to evaluate all possible combinations of current flow and analyze the data to locate areas of anomalously high or low resistivity. Soil and rock are basically nonconductive materials; however, both virtually always contain moisture. It is the moisture content that determines the conductivity of the various components that make up a particular formation. As examples, dry rock is quite resistive, whereas submerged clays are highly conductive. Accordingly, an area that was of very low density or perhaps an obvious void prior to grouting will exhibit different electrical behavior after being grouted.

Ground probing radar is another process that has been considered; however, with present technology, the accuracy and precision of its output is not as great as that of the previously described seismic methods. In radar evaluation, an electromagnetic pulse is generated on the ground surface. It is then reflected from the various interfaces at depth to a retrieval instrument on the surface. Equipment is available that can traverse the surface radiating and receiving repetitive electromagnetic pulses, so as to obtain a continuous record of the subsurface interfaces along the route traversed.

Ground probing radar instruments basically measure variations in the electromagnetic velocity of the different formation materials encountered. The procedure is thus especially useful for identifying clear interfaces, such as the presence of buried pipelines or subsurface concrete structures within a soil mass. Likewise, where the velocities of different soils in a grouted formation vary greatly, or have clearly different water contents and/or chemistry, identification of the boundaries may be viable. In most grouting applications, however, the variations of electromagnetic velocity between the grout and the soil are more subtle, and the applicability of the process is thus less advantageous. As with advanced monitoring in general, the process is in constant development and advancement. It may thus become more applicable to the verification of grouting work in the future.

6B.11.6 Grouting Specifications and Contracts

Although grouting is very much a technology, it is a specialized area of activity for most owners and their engineers, most of whom do not practice it regularly. There are however, many geotechnical engineering firms that have professionals with considerable experience, as well as several individual consultants. Perhaps those in the best position to maintain expertise in the technology, however, are the contractors who regularly perform the work. Many possess the specialized knowledge and equipment, and have the experienced personnel to optimally perform the work. Specifications should not be so limited or stringent that well-qualified contractors will be precluded from use of their innovations and special expertise.

Unfortunately, however, many grouting contractors completely ignore either sound engineering fundamentals or established grouting technology. And sadly, some continue to shroud their operations in mysticism. And then there the "high tech," black magic purveyors, who tend to use technical "mumbo-jumbo" espoused by well-educated engineers, to promote their activities, often in professional organizations and technical publications as well as through persuasive advertising. This latter group often sponsors seminars, where they provide supposedly nonproprietary handouts and guide specifications. Unfortunately, their field performance very often falls far behind the effectiveness of their marketing activities.

One must thus use extreme care in accepting advice from grouting contractors or, as many refer to themselves, "geotechnical specialty contractors." And especially, one should resist using "guide specifications" distributed from such operators for tentative projects. Such documents virtually always favor the supplying operator, and very often provide clever language that will eventually be used as a basis for a claim for extra payment. The author can often identify the origin of a specification upon viewing it. Tragically, very often such guide specifications are technically erroneous, and their use can result in very poor project performance, regardless of which contractor is awarded the work. As an example, a recent specification for compaction grouting, prepared by a well-known geotechnical engineering firm, stated that "additives such as . . . bentonite . . . can be included in the grout at the contractor's option." It has been well established, as extensively discussed in Section 6B.6.2, that inclusion of bentonite in a compaction grout will present a serious risk of hydraulic fracturing of the soil and loss of control of the grout deposition.

Specific areas that should be addressed in the specifications include:

- The clearly defined intent and limits of the zone to be grouted. A tentative layout of the grout injection holes is usually in order, although provision for changing either the number of holes or their spacing or location should be provided.
- Straightness or verticality requirements of the grout holes and the range of hole diameters that are acceptable.
- Grout mixer requirements. High-shear, colloidal mixers should be required for cementitious suspension grouts.
- Material requirements. This will vary with the type of grout being used. As an example, bentonite or other clay should be specifically prohibited from compaction grout, and an aggregate falling within the envelope presented in Figure 6B.57 should be required. Generic classes of solution grouts acceptable for water control grouting should be stated. The effects of shrinkage upon drying must be considered with any chemical solution grout.
- Acceptable range of pumping rates and a provision that the pump output will be uniform at all required pumping rates.
- Range of grout pressure that will be required.
- Requirements for monitoring and recording the various injection parameters during grouting. Consider mandated use of a record format, such as shown in Figure 6B.162, or a similar layout especially developed for the particular project or grouting methods being considered. Note that continuous pressure behavior monitoring should be mandatory in most grouting. (The only exception would be in some water-control applications.) Continuous, real time computer monitoring is preferable; however, many contractors lack either the knowledge or ability for its application, and it is thus usually reserved for large or important projects.
- Requirements for the workers and supervisors of the grouting crew. Some contractors, especially when working away from their headquarters location, send only a supervisor and/or perhaps one key worker to a project and hire locally to form the crew. This will result in less than good performance, due to the special skills required in grouting, and the required interworking of the different crew members, and should be prohibited by specification, in the writer's opinion.
- Levels of required improvement, acceptance criteria, and the methods to be used to establish compliance.

The use of open competitive bidding for most grouting work should be avoided, if possible. It serves as an invitation to lesser qualified and often shady operators, of which, unfortunately, there are many in the grouting business. Although public agencies are usually limited to the use of open bidding for their work, it is possible even for them to require meaningful prequalification of contractors for highly specialized work such as grouting, and many public projects have been so bid.

As a minimum, bidding should be limited to entirely prequalified contractors. Prequalification should not be given without a through appraisal of the individual applicants, which will include contacting the owners and/or engineers of *several* prior similar projects. Several is emphasized, as references given by some contractors in the past have turned out to be fraudulent. Additionally, even shoddy operators have an occasional project that has turned out well.

Continuity of a contractor's activity is also important. A contractor that has not preformed similar grouting work for an extended period of time is not likely to have working crews organized for that type of work. Use of a prequalification form such as that provided as Figure 6B.173 is strongly recommended. It is also a good idea to visit the contractor's headquarters, not only to confirm the statements made in the prequalification submittal, but also to see firsthand the manner in which he operates, such as orderliness of the equipment and facilities.

Where possible, rather than have competitive bidding, it is strongly recommended that a contract be negotiated with a thoroughly qualified and honest contractor. Oftentimes, these will be regional firms, which do not advertise but depend upon recommendation and repeat work instead. Locating them will thus require considerable effort, but that effort will be well placed, in that a better quality finished product, delivered in a timely manner and at a lower ultimate cost, will likely result. The importance of due diligence in selecting a contractor cannot be overemphasized. This should start with the completion of a prequalification submittal, including the information enumerated in Figure 6B.173.

In assessing the submittal, all entries should be independently verified, for very often, less than honest responses are made. The writer finds it distressing that so much dishonesty exists in the grouting business, but it does, and it must be recognized. As an example, one contractor responded in a submittal, that he was going to use a certain make and model of batching and mixing plant on a project. He further stated that he was going to rent it from Hertz Equipment Rental in Denver Colorado, and that he had a commitment that the equipment would be available to him should he receive the project. A simple phone call to the designated renter revealed that they did not own any such plants, but advised calling an 800 number, which connected to their national equipment inventory. A call there disclosed that the firm did not own any such equipment nationwide!

In another case, a contractor was awarded a project based on his proposal to use continuous computer monitoring, which involved a "proprietary software program specifically for the compaction grouting" that only he possessed. When the writer visited the project, a computer was indeed set up on a table in the injection area. Unfortunately, the only data that was being recorded was the hole number, which was entered manually, and pressure readings transmitted from a pressure transducer at the grout collar, which operated only sporadically. There was no mention of the injection rate and, of course, this rendered the pressure values of no usefulness whatsoever. The claim of use and setup of the computer on the work site did indeed look impressive. Unfortunately, this was simply a ruse to imply special competence, which the contractor did not possess. As for the proprietary software, it was nonexistent! And in the eyes of the writer, the entire operation was a sham.

The fact is, there are some brilliant advertising and proposal writers in the grouting industry. Sadly, they are not always so brilliant in carrying out the work, once it is awarded to them, and sometimes their presentations are downright dishonest. It is not unusual to find that once on the project, the formulation of often frivolous claims appears to take precedence over good performance of the work. One must be very cautious in accepting information from grouting contractors, and independent confirmation of the appropriateness and the accuracy of such is essential. Fortunately, there are some very good operators that are both honest and very knowledgeable, and therefore excellent sources of guidance.

Finally, unless the work to be done is quite routine, and the soil profile and the existing conditions are well understood, full-scale test injections are strongly recommended. Successful comple-

PREQUALIFICTION SUBMITTAL - GROUTING

Company Name________________________________Years in business_____

Address____

Phone___________________Years in specialized Soil Grouting_________

Bank_____________________________Contact___________________________

Surety Company____________________Contact__________________________

List five projects of similar size, scope, and intent, as the instant for which this submittal is being made. Provide the following information for each project.

> Project name; Dates work started and completed; Location Owners name, address, and phone; Owners Representative, name, title, and phone; Description of work done and contract amount.

Key personnel which you agree to commit to this project should you be awarded the contract.

Provide the following information for each listed position, for both primary as well as backup employee:

> Name; Number of years with the company; Number of years in this position; Number of years of experience in this type of Grouting, Special education, training, or certification, if any; Is employment by contract or day-to-day?
>
> Superintendent
> Craft Foremen
> Drillers
> Grout pump operators
> Key workers

FIGURE 6B.173 Recommended prequalification submittal requirements.

tion of a full-scale test, with exposure of the grouted soil as shown in Figure 6B.174, will not only confirm that the intended objectives can be met, but will also allow for a more precise understanding of the work to be done. Where more than one grouting method or material is being considered, they can all be evaluated in such test injections. This will also facilitate a more accurate cost estimate and, most importantly, allow appraisal of the contractor's field operations. One word of caution is appropriate, however. Unless the trials show needed changes from the anticipated methodology, allow no changes of the contractor personnel, equipment or methodology for the full-scale work.

PREQUALIFICATION SUBMITTAL - GROUTING, Page 2

Special equipment, including backup units which you will commit to this project:

List each unit; Provide make, model, and capacity, age and condition, location where unit can be inspected, and whether company owned or rental. If rental, provide name of renter and state whether you have a definite commitment that the equipment will be available for this project should you be awarded the contract.

Does your company maintain an employee training/qualification program? If yes, provide the scope and details thereof.

Does your company provide routine Quality Control on its projects? If yes, provide details thereof including any printed forms used therefor.

Does your firm maintain a testing facility for either quality control or research and development? If yes provide details thereof including a listing of equipment provided and tests performed.

Does your firm maintain a Safety Program? If yes provide a description thereof, including a listing of each accident, employee injury, or incident of property damage within the last five years.

FIGURE 6B.173 (*continued*).

There have been instances where grouting contractors have performed brilliantly on test applications, so as to qualify for the production work, but then used less costly procedures, resulting in inferior performance once the production contract was received.

6B.12 SOME FINAL THOUGHTS

When the writer first entered the world of grouting in 1952, there was virtually no established technology for the grouting of soils. The large government agencies that managed major dams each had its own ideas on how grouting should be done, although, interestingly, there was not a great deal of agreement between them. The contractors generally operated behind a veil of mysticism, and there was virtually no sharing of knowledge. Most grouting in that era, was in rock, and grouting in soil was generally unheard of.

Because grouting in soil is of relatively recent origin, and some of the main players, including the writer, were not part of the established club of "magic grouters," development of the technology for soil grouting, and especially that of compaction grouting, has been much more openly discussed and orderly. The technology for grouting in soil is to a large degree better understood, and authentic information on the technology more readily available than for some of the earlier forms of grouting. There remain many, however, especially in Europe, who attempt to adopt earlier rock grouting technology to soil.

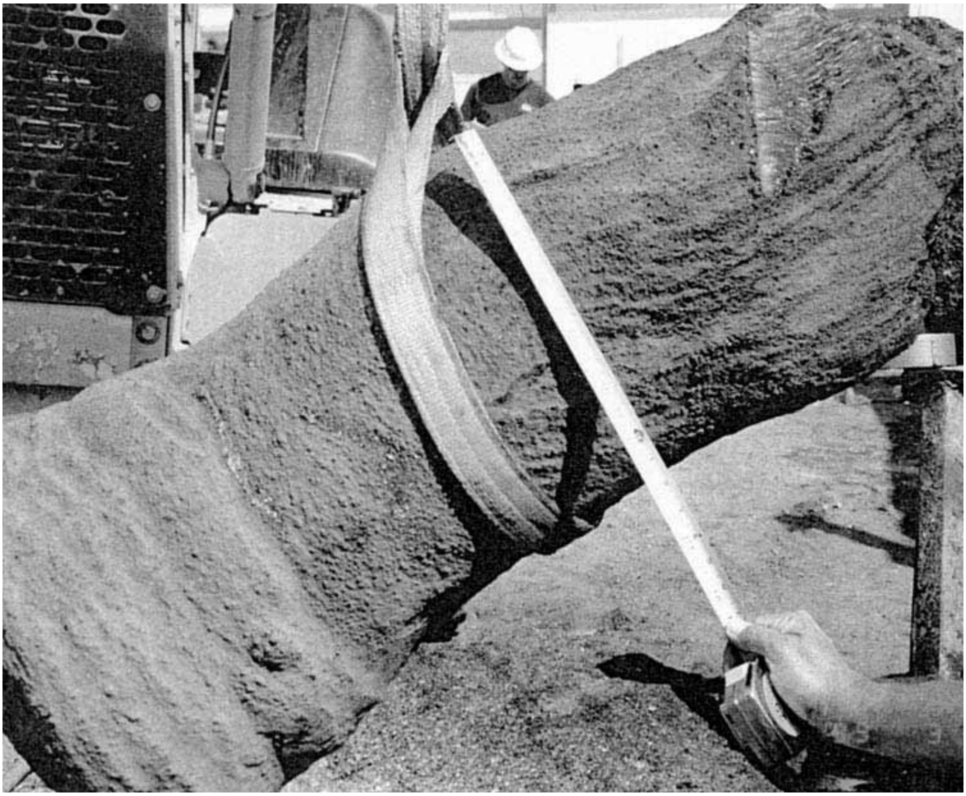

FIGURE 6B.174 Grouted soil mass from a test injection.

It is unfortunate that to this day, with but little exception, there is not much cooperation between grouting contractors. In fact, although there have been several attempts to start an organization to represent the grouting industry, such efforts have never been successful, as the different players overtly refuse to share their "proprietary" knowledge. The fact is that there is now very little grouting technology that is not well established within the industry, and thus little to hide or protect.

Grouting contractors in general, and especially the very large ones with staffs of professional engineers, often intimidate the less informed with technical jargon that may or may not be appropriate. In one instance, the defendant contractor in a lawsuit retained a university professor to develop a complicated analysis that would presumably absolve him of responsibility for the virtual destruction of a structure in which his incompetent injection had blown out an adjacent down-slope. And it is not unusual to see very misleading and even technically incorrect information in published works, including that which has been peer reviewed.

The above notwithstanding, geotechnical grouting is a fascinating science, and there is now a huge amount of well-documented technology available. Additionally, informative grouting education is readily available, such as that presented at the Annual Short Course on Fundamentals of Grouting, sponsored by the University of Florida, annually since 1978. The result is an ever-growing number of well-informed grouting professionals actively engaged in a continuing effort to advance the technology.

Geotechnical grouting can benefit many soil deficiencies. Properly applied, it can solve innumerable and sometimes vexing problems to the benefit of all involved. Because of the confusion and misinformation promulgated by so many within the industry, enormous opportunity is available to informed professionals to advance the science. Although it is not possible to cover every detail of the many facets of geotechnical grouting in a single chapter such as this, it is the writer's fervent hope that the reader has become sufficiently informed on the technology, that he will never again become confused or intimidated by other grouters and will, rather, become part of the ever-expanding group of enlightened grouters. Grouting is a science, and anybody that says otherwise is either ill informed or a liar.

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P • A • R • T • 7

FOUNDATION FAILURE AND REPAIR: RESIDENTIAL AND LIGHT COMMERCIAL BUILDINGS

As with almost every undertaking, even the best made plans occasionally go awry. This is certainly true where foundation design and construction are concerned, and especially with light foundations. This section will address those instances where the problems do arise.

Section 7A defines the common cause for foundation failures and how to identify the problems. It describes some of the more common misconceptions associated with differential foundation movement.

Section 7B presents the full spectrum of foundation repair options and special conditions that resist restoration, and it describes actual field practices.

Section 7C provides maintenance procedures which, if implemented, could prevent or foundation problems.

Section 7D suggests steps helpful to prospective buyers for evaluating the structural conditions of real estate properties. In geographical areas with widespread foundation problems, it has become routine for lenders and insurers to require structural inspections before mortgages or certain insurance policies will be issued. Information in this section can also be useful for building inspectors (see Section 9A, this volume).

Much of this information was presented in *Foundation Repair Manual,* published by McGraw-Hill in 1999. This prior work is a complete reference to foundation behavior, failure, repair, and prevention of damage. Similar information included herein is intended to help separate this handbook from other handbooks.

SECTION 7A
CAUSE OF FOUNDATION PROBLEMS

ROBERT WADE BROWN

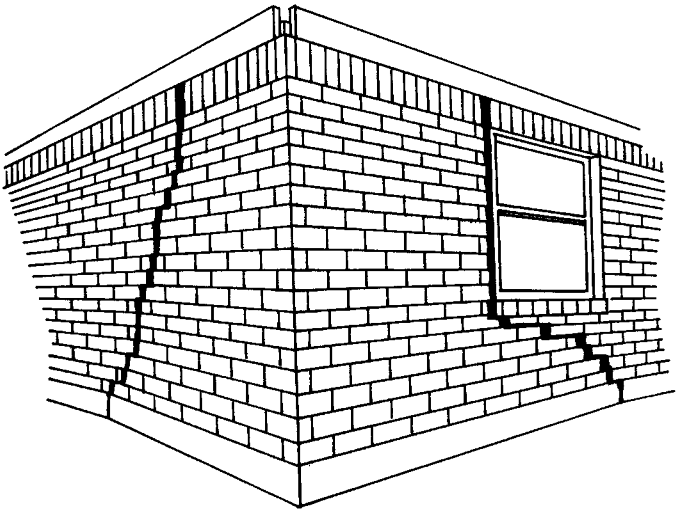
Certain of the issues discussed in this section are controversial, particularly in the sense of their true impact and influence on foundation failures. See Section 9A, this volume, for further information.

7A.1 INTRODUCTION

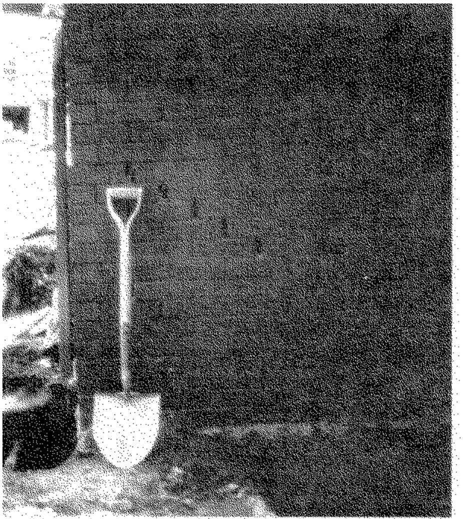
The major cause for foundation failures on expansive soil is water—too little or too much. Expansive soils suffer much more from this than nonexpansive soils, but foundations on either soil can be affected. The variation of water results in settlement (expansive or nonexpansive) and upheaval (expansive). Factors other than soil moisture variations will be the final cause to be discussed (see Section 7A.1.4). At what point does differential movement reach the point of requiring repair? The answer to this question is somewhat arbitrary, but the general consensus defines movement in excess of 1″ over 25 ft. as warrantying repair. Refer also to Section 7B.8.4.

7A.2 SOIL MOISTURE LOSS

Soil moisture loss occurs principally from either evaporation, transpiration, or a combination of both (evapotranspiration). Figure 7A.1 offers a drawing showing typical settlement as well as a photograph of an actual occurrence.



(a)



(b)

FIGURE 7A.1 (a) Typical foundation settlement. (b) Foundation settlement at the corner of the perimeter beam caused the very noticeable separation in the brick mortar. In this instance, the crack is in excess of 2 in (5 cm). Note that the brick has also slipped.

7A.2.1 Evaporation

Evaporation represents the natural loss of soil moisture through heat and wind. In expansive (cohesive) soils, the rate of loss is fairly slow and the depth of loss somewhat limited due to of very low permeabilities.

The depths to which the moisture loss occurs is referred to as the soil active zone (SAZ). Most published authorities suggest that the SAZ may extend to depths as great as 3.2 to 12.8 ft (1 to 4 m).[16,17,42,53,62,68,97,103]* However, a very high percentage of the *total* moisture loss (evapotranspiration) occurs at reasonably shallow depths. For example:

1. A study performed in Dallas, Texas, suggests that 87% of the total soil moisture loss occurred at depths above 3 ft (0.9 m).[103]
2. Holland et al. published data on an Australian soil describing an effective depth of 4 ft (1.25 m).[42]
3. Another study of several diverse areas in the United States indicates that over 80% of total soil moisture loss occurs within the top 1.5 m (4.5 ft).[68]
4. A similar study of an Israeli soil suggested that 71% of the total soil moisture loss occurred at depths above 3.2 ft (1 m).[62]
5. A study by Sowa presented data that suggested an active depth in a Canadian soil of 0.3 to 1.0 m (1 to 3.2 ft).[97]
6. A study in England described a London soil in which the range of principal moisture loss was 3 to 3.5 ft (0.9 to 1.1 m).[54]

Evaporation losses in expansive soils accounts for the very noticeable cracks in the Earth. As the soil becomes drier, the cracks grow wider and deeper until the soil moisture content might approach the shrinkage limit (SL).[16,65,99] The soil remains desiccated until water is once again made available. The soil then absorbs water and swells. The soil moisture content might then rebound to exceed the plastic limit (PL).[7,16,65,69,99] Overall soil (and foundation) movements are principally active when their in situ or "natural" moisture contents are between the SL and PL. However, shrinkage potential continues at moisture contents above the PL. In fact, the upper limit for soil shrinkage (evapotranspiration) probably extends to the field capacity, a point somewhere between the SL and PL.[69] Figure 7A.2 depicts the relationship between soil suction and volume. The range of soil moisture loss due to transpiration, which would be expected to influence foundations, is between the field capacity and plant wilt. Plant wilt is generally located at some point between the PL and the SL.[26,69] As soil suction increases, volume change decreases. (Soil suction describes the total moisture migration due to the combined forces of osmosis and capillarity. Thus, soil suction is a measure of the soils' capacity for water.) This relationship might lead one to assume that a foundation distressed by soil settlement would be *completely* restored by replenishing the lost water. This is not normally the case. Historically, as the moisture content cycles, the foundation moves up and down. However, each wet cycle leaves the foundation somewhat short of its original grade.[69] Figure 7A.3 shows shrink–swell profiles versus moisture content. In this study, the dry density was kept constant at 107.0 ± 0.6 pcf (1714 ± 9.6 kg/m^3) and initial moisture content (W%) was varied from 15.1% to 22.3%. This range of moisture essentially covers the spectrum between the SL and PL and is believed to be within the critical moisture range defined by Popescu.[69] A surcharge pressure of one psi (6.9 kPa) was applied at the initial moisture content (W%). At moisture contents of approximately 16% (SL) and 22% (PL), shrinkage is equivalent to swell. Between these points (A and B), shrinkage exceeds swell. At moisture content below Point A (SL), shrink is negligible and swell increases rapidly. Moisture contents greater than PL (B), depict a rapid decline in both shrink and swell. Only at Points A and B are the two equal at the same

*References are in Section 7E.

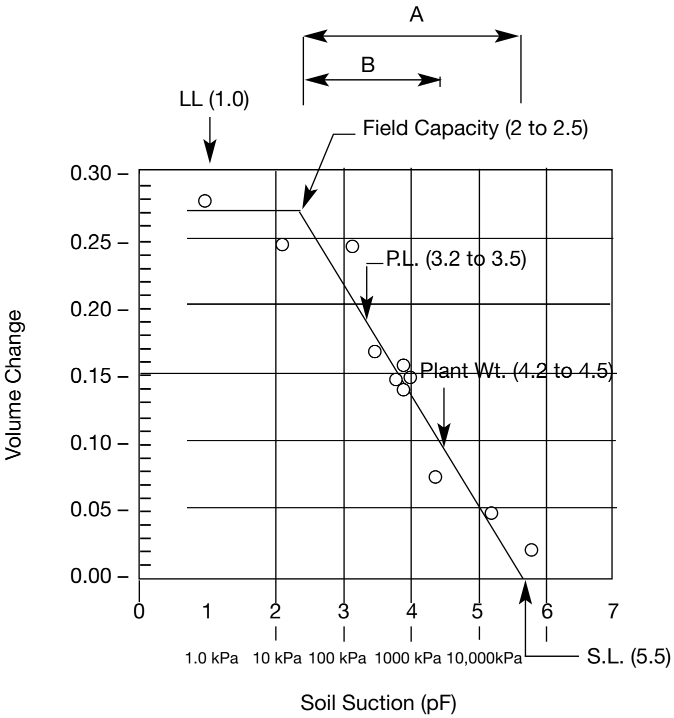


FIGURE 7A.2 Range of relative volume change. A = evaporation and transpiration; B = transpiration. After McKeen, 1992.[169]

moisture content. Hence, watering a foundation, once it has settled, might arrest further movement but will not likely bring it back to its original level.

7A.2.2 Transpiration

Transpiration is arguably a bigger thief of soil moisture than is evaporation (refer also to Section 7C). The removal of soil moisture attributed to transpiration declines substantially during a plant's dormant season, perhaps to less than 10% of the removal during the growing season. The depth of primary soil moisture loss due to transpiration is generally limited to the top shallow soils, 1.0 to 2.0 ft (0.3 to 0.6 m).[16,17,49,50,54,98,99] Under normal conditions the shallow "feeder" roots account for probably 90% of the plant's total requirement for nutrients and water. (In arid climates, deeper roots, i.e., tap roots, produce much of the water requirements for the plant. However, deep-seated moisture losses do not generally influence the stability of lightly loaded foundations). The lateral extent over which the shallow roots are active is generally limited to the canopy area of the plant or tree.[15–17,42] As a matter of curiosity, what would be a reasonable estimate for the soil moisture loss due to transpiration? This question has many answers, since transpiration depends largely upon the type, size, and density of vegetation as well as the ambient wind and temperature. Nonetheless, it is possible and practical to use reasonable assumptions and arrive at a representative number. Dr. Don Smith, Botany Professor at the University of North Texas, Denton, Texas, suggested that a "reasonable" value of water input for a 10 in (25 cm) diameter *Quercus stellata* (post oak) would be about 50

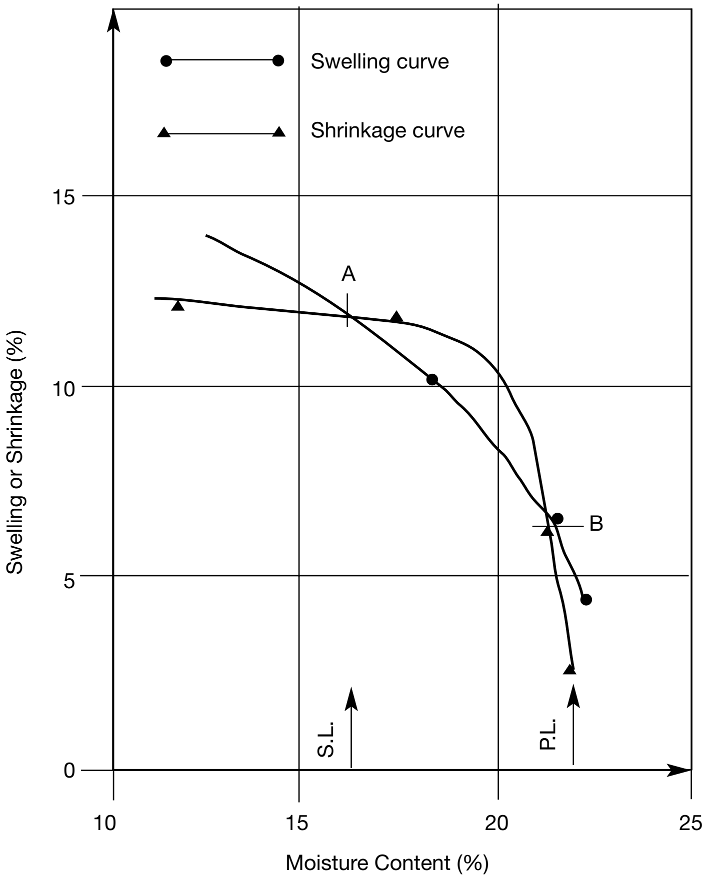


FIGURE 7A.3 Effects of moisture content on swelling and shrinkage. After Chen, 1988.[26]

gal/day in a semitropical climate. Assuming a 30 ft (9 m) canopy (root spread), the moisture loss per surface square feet would be 0.07 gal. (2.34 L) or 16.3 in^3 per day. A normal lawn watering program for the same geographical area would distribute 144 in^3 (0.62 gal) per square foot, twice weekly. Over a one week period, transpiration by the tree would account for 114 in^3 per square foot and the watering would add 288 in^3 per square foot (0.093 m^2). The surplus sprinkler would go into dead storage within the soil until removed by evaporation or transpiration. The main point of this is that tree roots would have adequate moisture and would not seek the inhospitable soil conditions beneath the foundation. This analogy excludes both the effects other vegetation might have on overall transpiration and the natural sources for precipitation (dew, rainfall, etc.). Generally speaking, if the vegetation has a healthy green color, the root moisture need is being satisfied.

The biggest threat that roots pose to foundations might be the presence of surface roots beneath a shallow foundation beam. As the root grows, a shallow beam can be literally "jacked" upward or heaved. Tree roots do not cause the foundation problems for which they are blamed.[16,26] Pier-and-beam foundations have interior floors isolated from the bearing soil, and slab foundations have interior floors isolated from the bearing soil and perimeter beams that should extend well into or below the surface root zone.

Removing a *mature* tree in close proximity to a foundation can cause more problems than it cures. The decaying plant roots can produce both permeable channels, which might funnel water beneath the slab foundation, resulting in upheaval, or voids, which result in settlement. On the other hand, tree roots can enhance foundation stability by increasing the soil's resistance to shear.[28,107]Over the past few years, one foundation repair contractor requested his drilling and excavation superintendents to keep track of instances where tree roots were encountered *beneath* the perimeter beam. At this point, no effort was made to specifically identify the type of root. Data was also not recorded concerning type, age, size or proximity of tree to the perimeter beam. The superintendents were instructed to merely record roots and type of foundation. On the average, a minimum of 20 piers or spread footings were installed per day. Assuming only a 5 day work week, this would provide about 100 daily observations, or 5200 per year. In cases where the depth of the beam below grade exceeded 18 in (0.45 m), roots were reported in less than 0.5%. When the perimeter beam depth was approximately 12 in (0.3 m), the instance increased to about 1%. When beam depths were less than about 6 in (0.15 m), the incidence increased to about 4%. Clearly, the data do not suggest a "real world" panic or, for that matter, even a particular concern. The quality of these data might be somewhat questionable, since the controls and specifics were less than "scientific." This would be an excellent area for more research.

7A.2.2.1 Summary Tree Roots versus Foundation Distress: Slab Foundations on Expansive Soil
Botanists, horticulturists, and agronomists uniformly seem to agree that:

1. The canopy of a tree determines the aerial extent of the feeder root system. The roots will extend only a short distance beyond the drip line of the canopy. According to Don Smith, a *reasonable* number to describe the radial pattern would be $1.25C$, where C is the radius of the canopy, or the radial distance from the trunk to the drip line. (The 1.25 factor is conservative. For most trees under most conditions the factor is likely closer to 1.1 or less.)
2. Feeder roots tend to be quite shallow—no deeper than 1 to 2 ft (0.3 to 0.6 m)[49,50,98] This preference is promoted by the fact that tree roots prefer rich, aerated, and noncompacted soil.[50] Deeper roots may also contribute to the nutritional needs of the tree, depending upon the species and size of the tree and availability of water and to a lesser extent on ambient temperatures. (In conditions of drought, the deeper roots play an increased role. Shallow roots can become active when shallow moisture is restored.)
3. Soil moisture loss due to transpiration has been determined to extend to depths below the soil water belt.[16] However, in each of these cases, the *preponderant* loss (80 to 90%) occurs at relatively shallow depths, generally within the top 3 ft (1 m).[53,62,68,97,102,103] T. J. Freeman et al. suggest that soil moisture loss below 6.6 ft (2 m) may not materially influence foundation behavior.[42]
4. Chen states that "the end result of shrinkage around or beneath a covered area seldom causes structural damage and therefore is not an important concern to soil engineers."[26] This analysis obviously introduces the presence of the foundation slab into the soil moisture loss equation.
5. The presence of shallow roots can be beneficial to foundation stability because their presence increases the soil's resistance to shear.[28,107]
6. McKeen suggests that the range for relative volume change exists within the PL and SL.[26,69] Classically, this moisture loss tends to involve pore water, the loss of which does not necessarily relate to soil shrinkage.[26,28,69] [Water bonded to or within the clay particles can be transferred to pore water, but temperatures above 212° F (110° C) are required.]
7. One inch [2.5 cm of water spread over 1000 ft^2 (93 m^2)] equates to 623 gal (2360 L) of water. A tree with a canopy area (C) of 1000 ft^2 ($C = D/2 = 36/2 = 18$ ft, where D is the diameter of thre tree canopy) might require 50 to 100 gal (379 L) of water per day to remain healthy.[1] Effectively, a little over 1 in (2.5 cm) of water, once a week (whether supplied by watering or Mother Nature)

would satisfy the needs of the tree, neglecting serious run-off. Roots would not be "encouraged" to encounter the hostile environment found beneath the foundation.

The foregoing lists facts developed by the referenced academicians, engineers, and geotechnicians. Some of their work did not involve "real-world" situations. That is, the conclusions were drawn from tests: (1) made on exposed soil, (2) using slabs poured directly on the ground surface, or (3) wherein no reasonable effort was made to exclude evaporation from transpiration. Source supplying actual field data taken from situations involving real foundations suggests that the true root problem is really not much of an issue.[16] Data collected since 1964 and involving over 20,000 actual repairs suggest the following. Refer also to Section 9A.

1. During the process of underpinning ordinary slab foundations [perimeter beam depth in excess of about 14 in (3.5 cm)], roots were found *beneath* the beams in less than 2% of the excavations. (The average number of excavations per job was 13. This extrapolates to well over 250,000 observations.) In many of these instances, the roots noted may have been in place prior to the construction of the foundation.
2. Virtually never does the company supplying these data recommend either removal of trees or the installation of root barriers. Removal of a tree can create far more serious concerns than leaving it be. None of these projects require *later* removal of trees, which certainly suggests that roots were not a problem.
3. If tree roots were truly a serious problem, why wouldn't all foundations of similar designs placed on similar soil and surrounded by similar trees experience problems?[16] Section 7C and Figure 7C.4 present a discussion on the impact of tree roots on foundation stability. In all, this slab foundation [a 30 in (76 in) perimeter beam] has eight pin oak trees growing along the west and south walls. The trees average 18 in (46 cm) in diameter with a height of 36 ft (11 m) and canopy diameter of 32 ft (9.8 m). The trees are within 6 ft (1.8 m) of the perimeter beam. The foundation shows no ill effects after 17 years. The soil is fairly alluvial soil with a PI of about 42.

It would appear that many investigators confuse perimeter settlement with center heave. From hundreds of engineering reports on file, it seems that many of the investigations arbitrarily assume the highest point within the foundation to be the bench or reference point. This obviously forces all surrounding measurements to be negative. Negative readings are generally associated with settlement. The preponderant conclusion then points to settlement. In lieu of another "culprit," nearby trees become the guilty party. This practice may be the result of either intent or oversight.

As a matter of fact, settlement is rarely the cause for foundation repair.[16,26] The preponderant cause for repair is upheaval, brought about by water accumulation beneath the slab. The source for the water can be natural (poor drainage) or domestic (utility/sewer leaks). The latter accounts for perhaps 70% of all foundation repair.

7A.2.3 Evapotranspiration

Evapotranspiration is the combined loss of moisture due to both evaporation and transpiration. This is the mechanism which in the end accounts for soil shrinkage and foundation settlement. However (to quote F. H. Chen), problems relative to "normal" settlement are comparatively few and minor. Chen states, "The end result of shrinkage around or beneath a covered area seldom causes structural damage and therefore is not an important concern to engineers".[26] Other publications have supported Dr. Chen.[15–17]

7A.2.4 Remedy

Restoration of foundations distressed by settlement is fairly straightforward. One needs merely to raise the settled area back to the "as built" position. With slab foundations, this involves mudjacking and, in some cases, underpinning. With pier-and-beam foundations, the lower perimeter areas are underpinned and the interior floor reshimmed. (Refer to Sections 7B.1, 7B.2, and 7B.4.) With any foundation-raising operation, it is often necessary, and frequently wise, to remove all patches that have been used to fill prior cracks. This is particularly true for any masonry patches. The "free space" is required to permit the foundation to be safely raised.

7A.2.4.1 Recourse

If you suspect a serious problem, the best course of action is probably to consult an attorney. This does not in any form recommend or suggest litigation; however, reliable advice is quite valuable. Recourse from settlement problems must generally involve negligent behavior on someone's part. For "new" construction, this could involve builders, engineers, architects, inspectors, contractors and/or developers. The statute of limitations in many states is ten (10) years. Example of negligent acts could include: 1) Failure to properly design the foundation to accommodate the site conditions; 2) Failure to comply with the plans or specs; 3) Negligent inspections during critical states of construction; 4) Not exercising a prudent attempt to grade the lot to control drainage or 5) Insurance providers who offer structural coverage.

In pre-owned properties recourse again is generally limited to some form of negligence and could involve realtors, engineers, inspectors or neighbors. Events that might be covered here would be: 1) Negligence of inspectors and/or engineers whose responsibility it was to inspect the property to ensure that no structural problems existed; 2) Realtors and sellers can be at fault under "latent defects" provisions. 3) Neighbors can be negligent if they cause undue drainage onto your property.

7A.3 TYPICAL CAUSES FOR SOIL MOISTURE INCREASE (UPHEAVAL)

The foregoing paragraphs have related broadly to soil moisture gain or loss, with no consideration being given as to the source or cause for this moisture variation. Since upheaval is, by far, the most serious foundation concern—especially when dealing with concrete slab-on-grade foundations—this eventuality will be addressed first. Figure 7A.4 contains a drawing typifying upheaval and an actual photograph of interior slab heaval. The drawing (a) is an over simplification of upheaval but tends to serve as an example. Note the preponderant vertical cracks, wider at the top, the separation of the frieze boards from the brick soldier course, the open cornice trim and the bow in the slab. In the real world, the perimeter brick may not show the major results of upheaval but the weaker interior slab will. Often, the exterior mortar joints are mostly level. The photograph (b) shows upheaval as it normally appears. The central slab is clearly "domed." The down spout appears to be level, yet the ends are several inches off the floor.

Sources for soil moisture could include such diverse origins as:

1. Rainfall
2. Domestic sources, which include:
3. Redistribution of soil moisture from wetter to drier soils or
4. Subsurface water

7A.3.1 Rainfall

Rainfall is perhaps the principal source for bulk water. However, as far as foundations are concerned, this threat is minimized due to the mere presence of the foundation itself. First, the structure isolates the foundation-bearing soil from contact with the rain. (The perimeter beam tends to

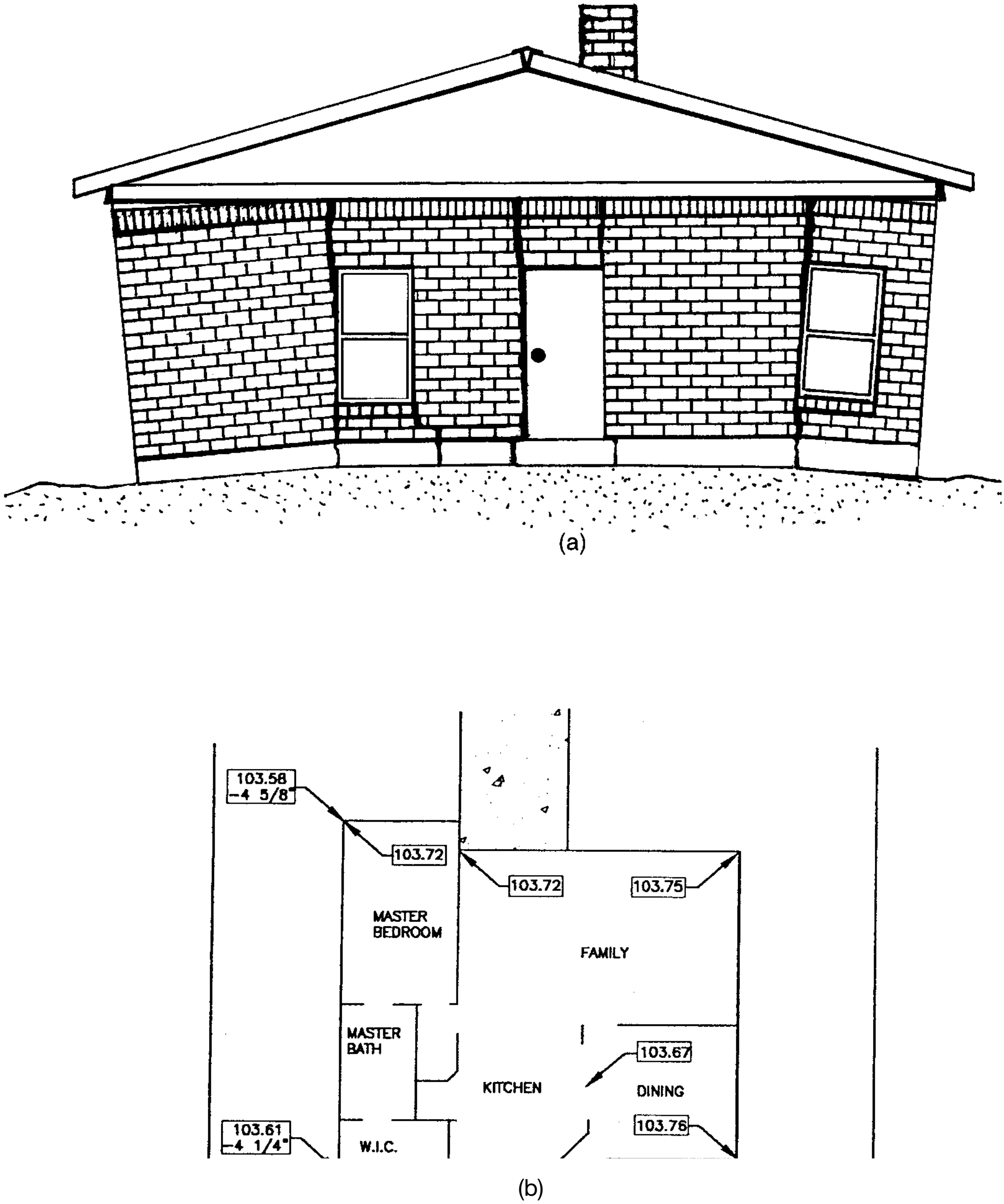


FIGURE 7A.4 (a) A drawing typifying upheaval (b) an actual photograph of interior slab heaval. In both examples, the "hump" in the floor is obvious.

restrict any lateral transfer of water.) Second, proper surface drainage adjacent to the beam should severely limit (if not eliminate) any availability of surface water to the soil beneath the foundation (run-off).

7A.3.2 Domestic Water

Domestic water is statistically the greatest concern, not so much from watering as from some form of leak. Watering can be controlled and, with the maintenance of adequate and proper drainage, any excess water can be safely directed away from the foundation. Leaks, on the other hand, are another matter. Supply line leaks can contribute a large quantity of water over a short period of time. However, a pressure leak is usually heard or noticed on the water bill and quickly repaired. The big threat, particularly to slab foundations, is the silent sewer leak.[15–17,40,52,64,86] This problem can exist years before detection. In fact, the existence of a leak is most often suspected or identified as a result of foundation damage. Upheaval can occur and advance rapidly with a rather extensive range and scope of damage. Very little water can cause serious distress. Under certain conditions, as little as six drops of water per hour, persisting over 12 months, can heave a slab foundation over 1″ (2.54 cm).[15–17,102] (Refer to Sections 7B.8 and 9A.) Actual data indicate that over 70% of all slab foundation repairs are caused by upheaval (water accumulation beneath the slab). A high percentage of the recorded cases of upheaval have been linked to sewer leaks.[15–17]

A small cadre of engineers tend to deny the propensity and extent of sewer leak problems.[15–17,26]The principal reason for this minority position might be traced to their clientele, i.e., insurance companies who have no liability for foundation losses attributable to causes other than “the accidental discharge of water from utilities” (or upheaval). Settlement is not an insured act. Refer to Texas Court of Appeals Case No. 13-92-C, Corpus Christi, December 16, 1993, styled Nicolau vs. State Farms Lloyds. Familiarity with this case is a *must* for those involved in foundation repair, appraisal, or engineering evaluation. (At the time of this writing the Appealant Court Decision has been appealed to the Supreme Court.)

Along these lines refer to Figure 7A.5. This drawing, prepared by one insurance company’s engineer, depicts floor elevations taken inside a distressed slab foundation. The peak point is indicated to be about six inches (15 cm) as shown. The basic central high as shown is 4.5 to 5.5 inches (11 to 4 cm). The engineer acknowledges the two sewer leaks but states, “since the leaks are not located near the heaving in the foundation, and there is no change in floor slope around the leak areas, I conclude that the reported underground plumbing leakage did not adversely affect the foundation.”

Review the engineering elevations: 1) the grade differential is 6″ (15 cm). This heave would require a *significant* quantity of water. 2) The sewer line and leak *are* included within the central high. 3) Water following along either the sewer ditch and/or interior beams could very well account for the accumulation at the central as well as the peak point. (Some geotechnicians believe that water can flow through fill sand or coarse base material. This could account for an unpredictable pattern.) 4) the foundation area between the cross beams (including bath leak) is somewhat aligned in the 5.0 to 5.5 inch (13 to 14 cm) high, and 5) The redistribution of *existing* soil moisture could not reasonably account for the magnitude and location of the heave.[16,26,102,103] The 6” heave would require a seemingly impossible soil suction and water availability. The only logical source would be the sewer leak. (The location of both the sewer lines and the probable interior beams was added by the author based on “best available information.”) The author had not inspected the property at the time of this analysis. If the issue goes to litigation, the author’s inspection will become necessary.

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

The range of upheaval has been recorded as high as 12″ (30 cm). [Conversely, settlement caused by soil moisture loss seldom exceeds 2–3″ (5–7.5 cm).] The general statement that upheaval is

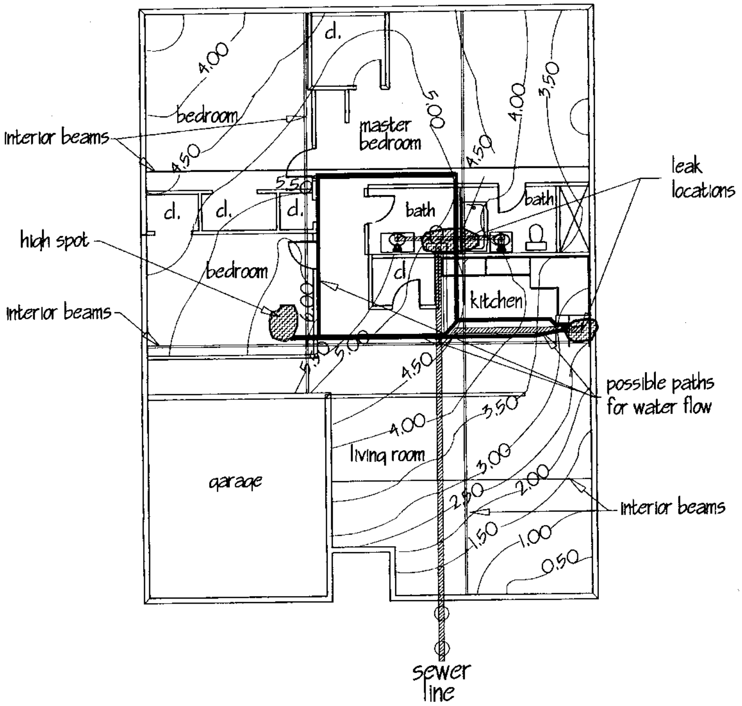


FIGURE 7A.5 Floor heave versus floor elevations.

caused by water is true. However, water introduced into an expansive soil does not *always* cause upheaval. If the in situ soil moisture is near the PL, much of the soils' swell potential may have already been exhausted. Figure 7B.6.2 gives the relationship between free swell and moisture content. The swell potential decreases dramatically as moisture content increases. (For these tests, a confining resistance equal to the assumed overburden was used.) At some point, additional moisture produces little or no swell.[15–17,86]

7A.3.3 Redistribution of Existing Soil Moisture

Redistribution of existing soil moisture can occur to some extent, particularly when dealing with slab foundations. The presence of the foundation slab prevents, or inhibits, both the evaporation and penetration of water. This, in itself, conserves moisture and tends to encourage upward capillary flow. Given time, the bearing soil confined beneath the foundation will likely increase in moisture content.[7,8] This increase will not be centralized but will be essentially constant over the expanse of the covered soil.[15,16,42,52,79,102,103] The vertical permeability in a clay soil is often approximated to be in the range of less than 1 ft/yr.[65,99] The horizontal permeability is frequently assumed to be at least ten times that or 10 ft/yr.[65,86,99] This means that lateral water flow can occur ten times more readily than vertical flow. There is a very serious doubt that the natural accumulation of soil moisture beneath a slab foundation has ever been sufficiently serious as to cause the need for foundation repair.[52,53] (Refer also to Section 9A.) Certainly, in the author's experience, spanning nearly 35 years and well over 20,000 residential repairs, no such instance has ever been documented.

Various sources for *subsurface moisture* would include: wet weather aquifers, perched water zones, ponds, springs, and/or shallow water tables. As long as the bearing soil moisture content remains constant, little or no differential soil movement would be anticipated. However, cyclic water availability often produces soil movement—swell or shrink. Water intrusion into shallow soils can cause the upheaval described above. Natural water drawdown can result in settlement. This sometimes occurs as the result of draining or lowering the water level in surface ponds or subsurface aquifers. For the former, the soils are often nonexpansive. When subsurface water exists, the problem is best resolved prior to construction and the practical solution is to initiate whatever steps are necessary to level the availability of water.

In some instances, chemical stabilization of the expansive clay might present a useful alternative.[16,17] (Refer to Section 7B.6.) Most chemicals are designed to abate swell and have, at best, limited capacity to influence moisture loss or settlement.

7A.3.3.1 Summary of Center Doming

The relationship of edge or center lift to describe potential foundation slab failures was introduced in the mid to late 1970s.[90] From design concerns, the approach seems technically sound. Problem can arise, however, through interpretation and application of the failure modes to actual foundation failure.

Edge lift rarely becomes a significant problem in foundation repair, although it remains a viable concern to design. Classically, this phenomenon occurs when the soil moisture beneath a segment of the perimeter beam exceeds that beneath the interior slab. Heave of a perimeter beam is somewhat rare, due in large part to the structure load transmitted to the beam. In residential and other lightly loaded construction, the load imposed on the beam might be on the order of 600 lb/ft^2 (290 kg/m^2) on interior floors. The added weight (load) substantially inhibits heave.[20] Further, the drastically reduced (or absence of) resistance (or load) on either side of a beam encourages the heave to be directed toward the path of least resistance. Areas adjacent to the beam might heave, but the loaded beam would be resistance to heave. Whatever the reason, problems associated with perimeter beam heave due to water are rare. (Tree roots have been known to heave a perimeter beam, particularly on pier-and-beam foundations.) Center lift actually refers to two different modes of failure: (1) perimeter settlement or (2) center lift, or as it is sometimes called, center doming.

Perimeter settlement is a concern and certainly should be an issue in foundation designs. However, foundation problems related to moisture losses in expansive soils are clearly overstated.[16,20] Soil moisture removal is slow, limited in scope, and easily abated if not reversed. When problems do develop, of all foundation repairs actually performed, less than 30% are the result of settlement and therefore are generally less costly.[16]

Center lift is a serious concern to both design and repair. The fact of center heave is not in question. The principal cause for foundation repair is, in fact, upheaval. The cause for upheaval is generally beyond question. The presence of water beneath the foundation is generally accepted to be the cause. However, there are diverse opinions as to the source of water that causes the soil to swell. Refer also to Section 9A.

"Natural" Center Doming. A few geotechicians and engineers (mostly hired by insurance companies) advocate the natural occurrence of "center doming" beneath slab foundation on expansive soils. A few seem to believe that virtually all slab foundations will experience "center doming," given sufficient time. The theory is that soil suction will increase the moisture within the central area of the soil confined by the foundation to the point where soil swell and foundation heave is inevitable. First, for this to come about, the soil moisture in deeper soils must initially be higher than that for shallow soils. Soil suction and perhaps elevated temperatures could then make water migrate to dryer surface soils, resulting in soil swell. (The shallow soils are of primary concern because these soils have the greater propensity for volumetric movement.)[36,65,86] However, the soil permeability in the vertical direction is less than one-tenth that in the horizontal direction. Tom Petry has given an estimate of horizontal permeability (K_H) to be on the order of 10 to 20 ft per year.[6] Others may have given a broad value for vertical permeabilities of 0.1 to 1.0 ft per year

(10^{-7} to 10^{-6} cm/sec) for heavy clay.[20,65,68] (Increased K_H over K_V is due largely to roots, fissures, fractures, and/or normal sedimentary planes.) The disparity between K_V and K_H would seem to preclude the "dome" theory in itself. A moisture content would more likely be distributed rather uniformly over the entire surface soil, not as a "dome."[16,52,53,78,102] This theory would prevail even if interior perimeter soils were to lose moisture to the exterior. Second, bear in mind that (1) the natural doming wouldn't be dependent on or influenced by other moisture sources. For example, the mere process of repairing a leaking sewer system would have no influence on arresting the heave. (2) All foundations of similar design, with other factors being equal, would eventually suffer the problem, and this simply does not occur. (3) Based on the extremely low permeabilities involved, natural doming, if possible, would require a long period of time to develop, perhaps in excess of 10 years. How then could this theory explain upheaval in foundations 2 to 10 years old? Reasonable design concerns can, however, resolve the problems of center heave as defined in the foregoing paragraphs.

Center Heave Due to Extraneous Water. One source of reliable data suggests that center doming is in fact a particularly serious threat to the stability of concrete slab foundations. The heave of concern involves the introduction of water beneath the foundation. This water can originate naturally (precipitation accumulated by bad drainage) or from domestic sources (plumbing leaks, supply or waste). Water produced naturally can be alleviated by conventional drainage improvements. The hidden "cancer" represented by domestic leaks, in particular the silent sewer, is the foremost problem. Perhaps the only reliable data concerning foundation repair identifies sewer leaks as the cause of 70% of all slab repairs. These data point out that once the cause of the problem (i.e., sewer leaks) has been eliminated, movement arrests in about 90% of the documented cases. (In the remaining 10%, the continued movement was traced to an undiscovered plumbing leak in over half the cases.)

A few engineers and geotechicians completely discount plumbing leaks as a cause of foundation failure. Generally, they identify settlement (usually due to root or natural doming) as the cause for the failure. Again most of these proponents work exclusively (or nearly so) for insurance companies. Although these investigators deny plumbing leaks to be the cause of the problem, all agree to immediately repair the leak, even though such repairs are typically quite expensive. To put the repair costs into some perspective, consider the example in Table 7A.1. The foundation repair cost on this same job was proposed at $4908. The insurance company paid for the sewer repairs but denied responsibility for the foundation claim. Note that the "typical" cost for sewer repair was $6,900.00 plus $2,500.00 for mudjacking. This produced a "typical" sewer repair cost of $9,400.00, nearly twice the cost for foundation repair in this example.

TABLE 7A Typical Costs for Sewer Repair (Slab Foundations)

Description	Quantity	Unit cost, $	Estimated cost, $
Dig and fill access for tunnel; includes pump protection	1	600	600
Tunnel under foundation	26	150	3900
Install pipe, fitting, and connections under slab materials plus labor	1	250	250
Backfll tunnel area	26	75	1950
Clean up		100	100
Permit/fees	1	100	100
Total		6900	

*Does not include mudjacking to complete the tunnel backfill. This "typically" adds another $2,500.00 to the costs.

These same proponents also seem to state that a sewer leak does not produce sufficient water to cause a serious heave. Without getting into the absurdity of this position (based on elementary soil mechanics or engineering, plumbing practices, the Manning equation, or common sense), it merely requires one to note that when the source for water is removed, movement ceases. Obviously, the sewer leaks did contribute to (if not cause) the problem. All other arguments seem to suddenly become moot.

The cause for the foundation failure does not influence the pocketbook of the reputable repair contractor, who gives repair estimates without concern for who pays the bill. The contractor does not have responsibility to customers and should carefully inform them about the cause of distress. This is simply because the contractor knows that if the original problem is not eliminated, the problem will recur. If the contractor correctly identifies the cause and this is not eliminated subsequent to repair, the contrators' warranty is likely null and void. The consumer is the party who suffers most. As a rule, the insurance companies are the only parties affected by the cause of problem. Refer also to Section 9A.

7A.3.4 Restoration

Upheaval represents that condition in which some area of the foundation is distorted in a vertical direction while other areas remain "as built." Remedial action involves raising the lowermost (unaffected) areas to a new, higher elevation in an effort to "feather" the crown or high area. Seldom is the end product "perfectly level," if such a term exists. However, the consolation lies in the fact that the foundation was never level. The repair procedures are those described in Sections 7B.1, 7B.2, and 7B.4 and are dependant upon whether the foundation is slab or pier-and-beam.

7A.3.5 Possible Recourse

Possible recourse due to foundation upheaval could include those cited in Section 7A.2.4. However, conditions of upheaval might also be subject to insurance coverage. Again, as with Section 7A.2.4, it is wise to consult an attorney experienced with this type of law. Until about 1996, many homeowner's B insurance policies contained coverage "for accidental discharge of water and damage resulting therefrom." This is, of course, not verbatim, but the intent is clear. However, conditions of upheaval caused by domestic water leaks are generally subject to insurance coverage.

Any act by others to subject your residential foundation to water (particularly a slab) might constitute "negligence." This could be a neighbor changing his drainage to adversely impact your property. It might be flooding caused by a broken city water line. An improperly installed sewer or water supply system might also represent culpable acts. For the same reasons as those stated in 7A.2.4 for preowned properties, sellers, realtors, engineers, and/or inspectors can be held accountable for negligent acts.

7A.4 FOUNDATION PROBLEMS NOT MOISTURE-RELATED

7A.4.1 Lateral Movements

Serious soil movement can be the result of some variety of lateral displacement. Lateral movements can be the result of soils eroding, sliding, or sloughing. This is generally associated with construction on slopes with unstable bearing soils. Movement is often precipitated or exacerbated by the intrusion of water into the soil to the extent that both cohesion and structural strength are threatened or destroyed. California mud slides are one prime, though extreme, example. The structural damage caused by these problems is often complete destruction. When remediation is possible, the situation is addressed in a manner generally consistent with that for acute settlement. Severe lateral movement is generally beyond the methods available to the repair contractor. However, the contractor can often provide measures to stop lateral movement. This could involve the placement of retaining walls, earth anchors, terraces or other such measures. This problem is generally considered beyond the scope of this book.

7A.4.2 Consolidation or Compaction

Settlement can also occur as a result of *consolidation* or *compaction* of fill, base or, subbase materials. With respect to residential construction, the most common problem deals with construction on either abnormally thick fill or a sanitary landfill. In either case, over time, the intended bearing soils fail due to consolidation. Normal settlement of fill is often active for periods up to 10 years and is somewhat dependant upon the cycles of precipitation and drought. Sanitary landfills can be active for longer periods of time due to voids continually provided by the decay of organic materials.

Consolidation of nonexpansive soils often results from the removal of pore water. One example of this might be instances where the water levels of lakes or ponds are lowered, allowing water to drain from the surrounding soils, often sand or coral. Elutriation of soluble material (usually salts) from soils, facilitated by the invasion of water, can create voids that at some point collapse and cause consolidation. In many cases, deep grouting is a required remedial procedure for the correction of either problem. Refer to Section 7B.6.[16] Once the deep-seated cause has been addressed, procedures common to foundation settlement can be used to "relevel" the structure.

7A.4.3 Frost Heave

Frost heave occurs as a result of soil water freezing with sufficient expansion to cause the foundation member to heave, or in some cases, as with basement walls, to collapse inward. Soils in subfreezing climates are the most susceptible to this problem. Frost heave can best be addressed in new construction design.

Frost heave in existing structures can result in slab heave or, in the case of basements, collapsed walls. Both should be very carefully treated, with the first step being to consult a geotechnical engineer who is skilled with this particular problem as well as the geographic location. Refer particularly to Sections 7A.3.4, 7B.2, and 7B.4.

7A.4.3.1 Basements and Foundation Walls

Failure in basement walls occurs in areas with or without expansive soil and is often preponderant in colder climate regions. Where concern for frost lines exists, the propensity toward construction with basements is enhanced. As far as expansive soils are concerned, Colorado is one principal area where basements are common. As stated earlier, frost heave can exert sufficient lateral load on a basement wall to cause inward collapse, much the same as hydrostatic loads (expansive or nonexpansive soils) or the expansion of expansive soil. Regardless of the cause, the repair approach would be similar. Refer to Section 7B.5 for details.

It might be interesting to note that above-grade foundation walls (with built-up floor systems, i.e., dock-high) suffer failures similar to those described in this section, except that the foundation failure (rotation) is toward the exterior and temperature is not a factor. Repair or restoration procedures are often much the same as those described in following examples. As a rule, the retaining or aligning procedures are installed on the external side of the foundation wall where property lines permit. Refer to Section 7B.5 for details on repair.

7A.4.4 Permafrost

Permafrost is a reverse problem to frost heave. Both require the same conditions, i.e., prolonged very cold climate and lenses of water trapped within the bearing soil. The end result of melting permafrost is subsidence. Unless the permafrost melts, there is no problem (foundation-related). Once the problem has occurred, a possible solution might be deep (intermediate) grouting, as discussed in Section 7B.3. This approach could be successful, if the free water is eliminated. Again, the first and best option is to consult a local geotechnical engineer familiar with the specific problem, whether it be of a design or remedial measure.

7A.4.5 Construction Defects

Construction practices or mishaps often create conditions that are conducive to foundation problems. Where possible, these factors are grouped for either slab or pier-and-beam foundations.

7A.4.5.1 Slab Foundations

There seems to be a number of events inherent to the original construction that can possibly result in foundation problems or in other cases impede the desired repairs. Some of these are:

1. *Utility leaks* beneath the foundation (refer also to Section 7A.3)
2. *Pouring the slab foundation off grade*. Figure 7A.6 presents grade elevation at various points over the area of the foundation. Note the differential of –4⅝ in (12 cm). Interior surfaces of the dwelling show only minor cracks in sheetrock and only a few doors are out of plumb. If leveling repairs were attempted based on these elevations, the structure would suffer severe distress. In fact, the extent of true differential movement noted in this foundation is “nominal” by most standards. Refer to Section 7C. At least part of the problem with this job originated from the fact that the brick ledge was used as a “bench mark”. This is not acceptable. The brick ledge is most often off grade. In fact, brick masons use the first few courses of brick in order to attain a level mortar joint. Elevations should be taken off the mortar joint on top of the fifth brick course. Reliance on faulty information would encourage frivolous litigation.
3. Faulty slab design or construction. Slab foundations poured with: a) insufficient slab and/or beam thickness, b) undersized, improper placement or absence of reinforcement, c) *too much water* in the concrete, which results in poor quality and substantial loss in strength are faulty. Not only do these defects encourage foundation problems but they also hamper (if not prevent) proper repair.
4. *Add-on slabs* poured in contact with another slab that already suffers differential deflection. This situation is most difficult to improve. The common joint poses a real problem. If the faulty slab is raised, the add-on will be low. If the add-on slab is also raised, the existing framing will be destroyed. A rule of thumb, “Never construct an add-on unless proper remediation has restored the original foundation.” This situation affects slab and pier-and-beam foundations alike.
5. *Faulty exterior grade*, nonconsistent watering practices, location and design of landscape plants each can promote foundation problems. See Section 7C.

7A.4.5.2 Pier-and-Beam Foundations

The problems of foundations constructed with the pier-and-beam design probably happen with the same frequency as those of other foundations but their *degree* is much less. The crawl space provides access for correcting minor grade problems experienced by interior floors. Other factors not so easily addressed include:

1. *Limited or no crawl space*. See Section 7B.1.5.
2. *Insufficient ventilation*. Refer to Section 7B.1.5.
3. *Water collected in crawl space*. This problem requires both proper drainage at the perimeter beam to prevent accumulation of excess water and adequate ventilation to control normal amounts of water. This is particularly important when the pier-and-beam foundation is of the “low profile” design. (See Section 4D.)

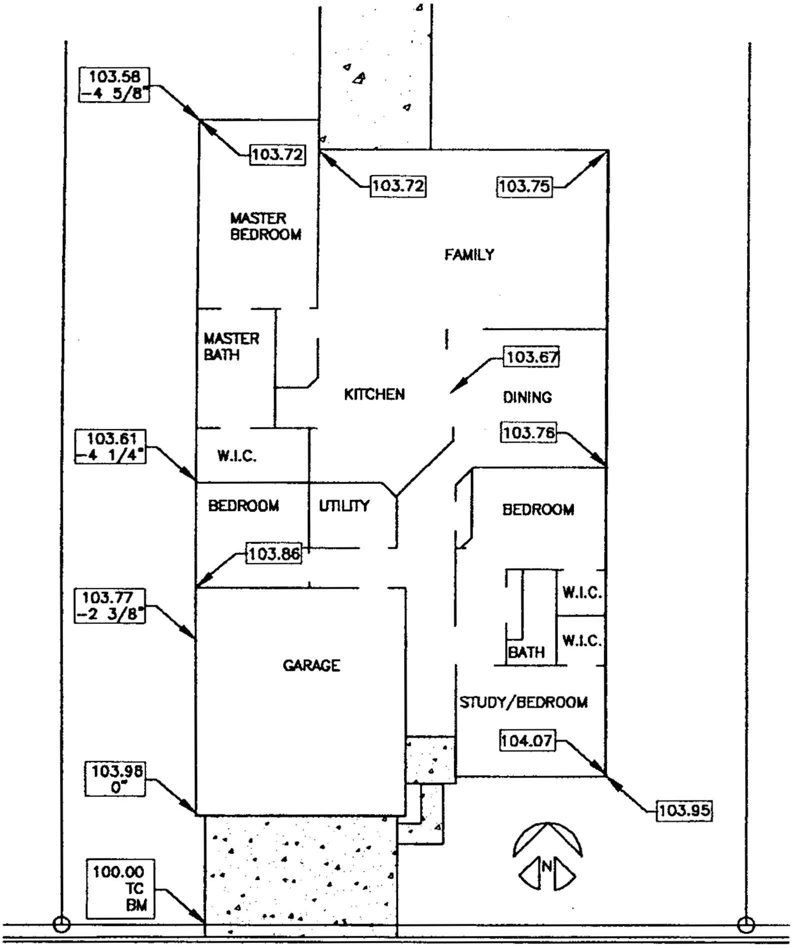


FIGURE 7A.6 Foundation elevations.

4. *Warped wood*, particularly affecting the joists and/or girders. When the wood substructure is subjected to a prolonged distortion (particularly in the presence of moisture), the individual wood members are subject to warp. If the warp is severe or well set in the wood, it is not likely that this condition can be reasonably cured. That is, leveling by reshimming existing pier caps (or even adding supplemental ones) is not likely to produce level floors. Some improvement is generally possible but some compromise is required.
5. *Faulty placement or design of piers and/or pier caps*. In some cases, deficient materials were used to support the wood superstructure. This is particularly pronounced in older foundations where wood "stiff legs" were used to support the foundation. These members were subject to deterioration as a result of decay and insect infestation. In modern-day practices, it is not particularly uncommon for a pier and pier cap to be located such that the girder is not properly supported. Sometimes the pier cap may be off-center, tilted, short, or miss the girder altogether.

7A.4.6 Negligent Maintenance

Negligent maintenance could refer to instances where

1. Standing water is permitted to approach and/or invade the foundation. Moisture accumulating in the crawl space of pier-and-beam foundations acerbates wood warping and contributes to instability of interior piers and pier caps. A substantial warp will prevent proper leveling (shimming pier caps) and cause increased costs. Water beneath slab foundations frequently causes upheaval.
2. Failure to address problems caused by erosion before they become critical. This concern would also include embankment failure.
3. The neglect of proper maintenance procedures is probably the foremost issue in the category.

Proper maintenance is fully discussed in Section 7C.

SECTION 7B
FOUNDATION REPAIR PROCEDURES

ROBERT WADE BROWN

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-

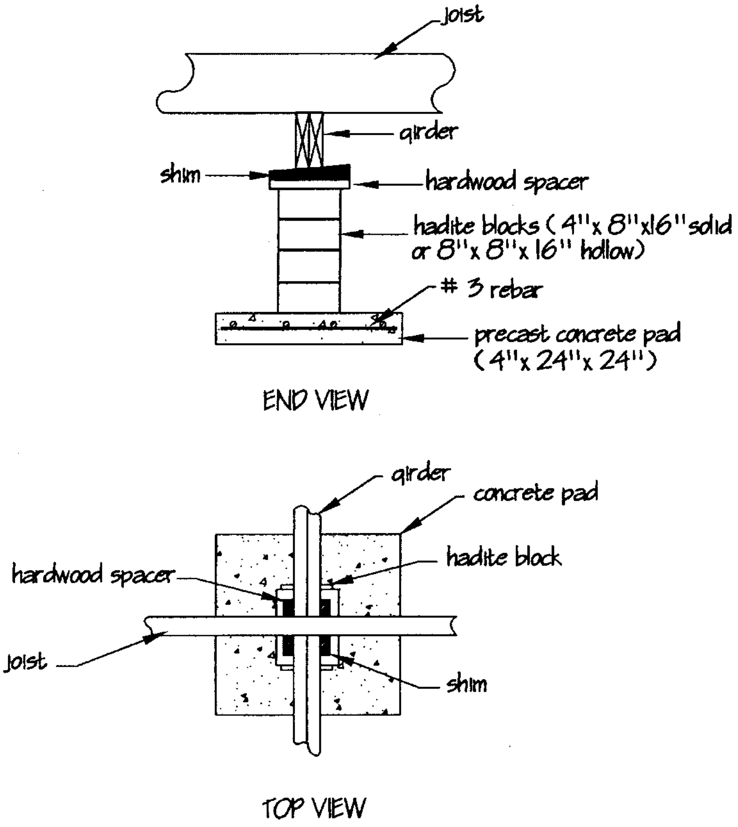


FIGURE 7B.1.1. Supplemental interior pier cap (typical).

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

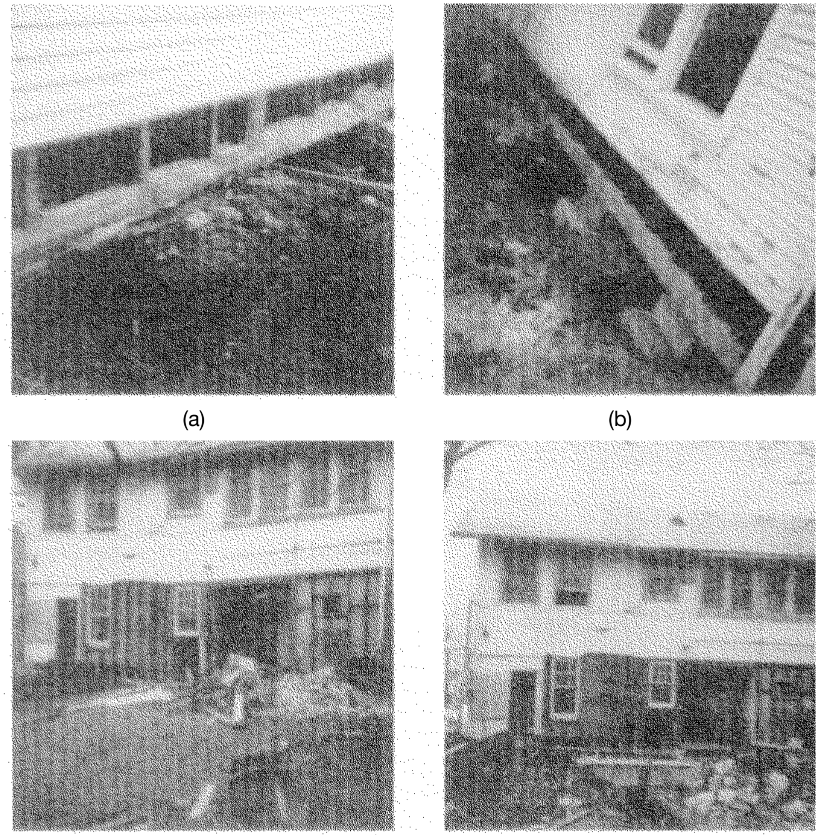
will be discussed in following paragraphs. For limited access, moderate tunneling can generally provide the access required for simple jobs.

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).

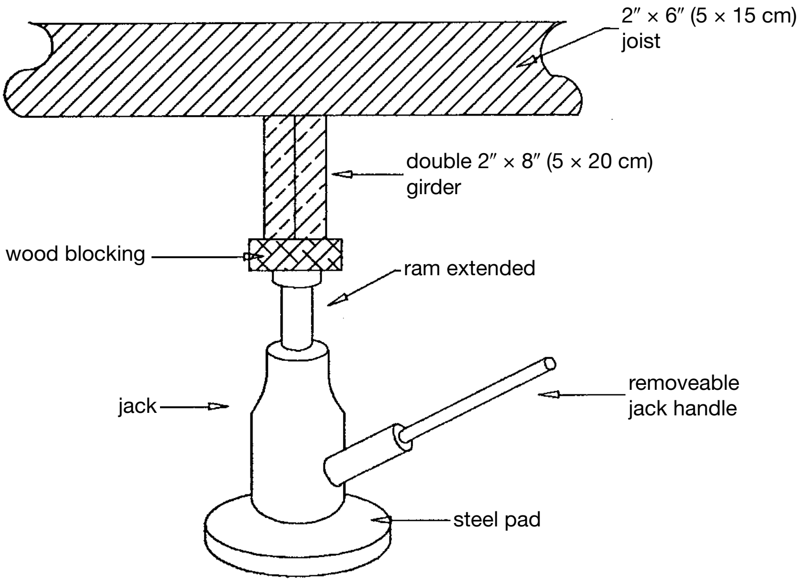


FIGURE 7B.1.3. Jack in place to raise girder.

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

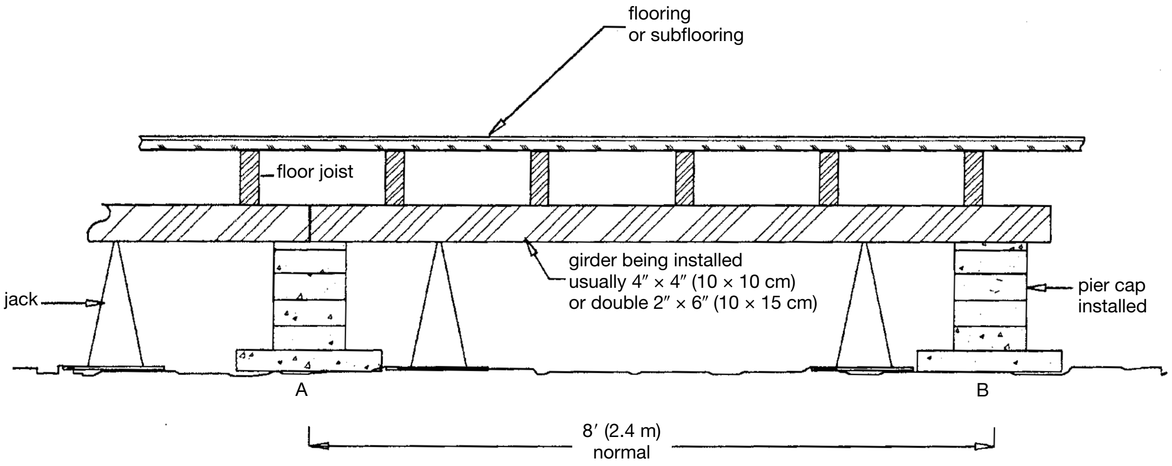


FIGURE 7B.1.4. Installing a wood beam to act as a girder to support floor joists. Note splice at pier cap A. Jacks are removed when pier caps have been installed

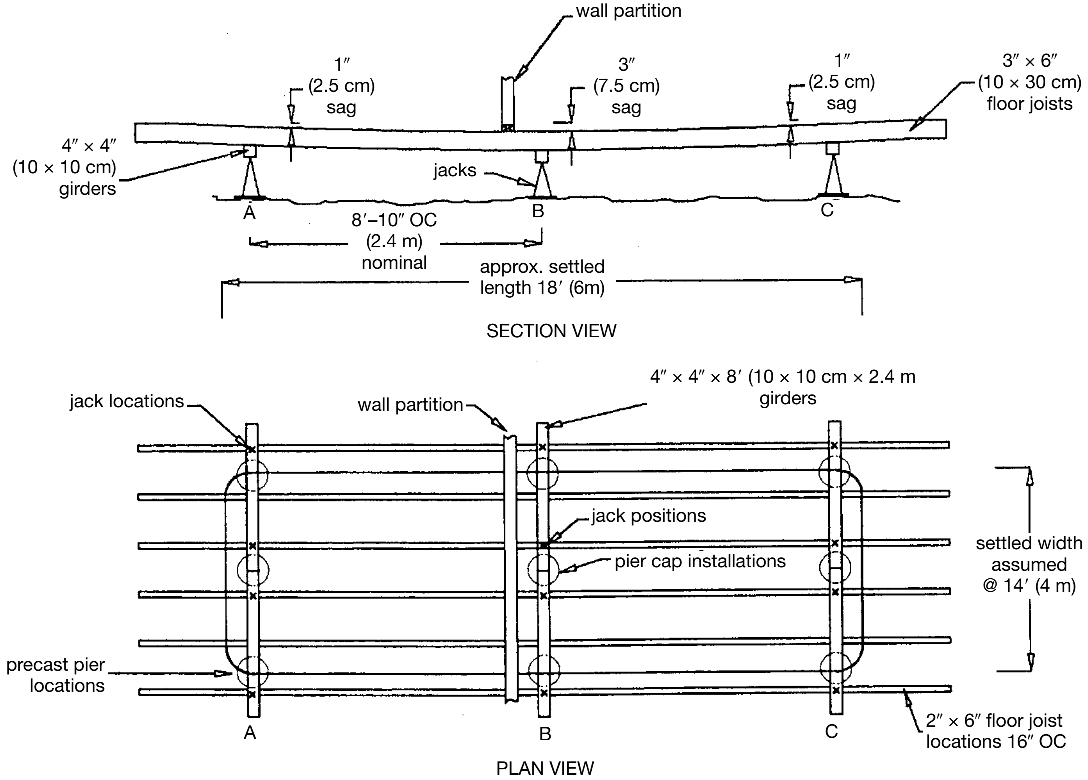


FIGURE 7B.1.5. Positioning supports for leveling floor sag. Refer to Figure 7B.1.4 for detail at splice.

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.

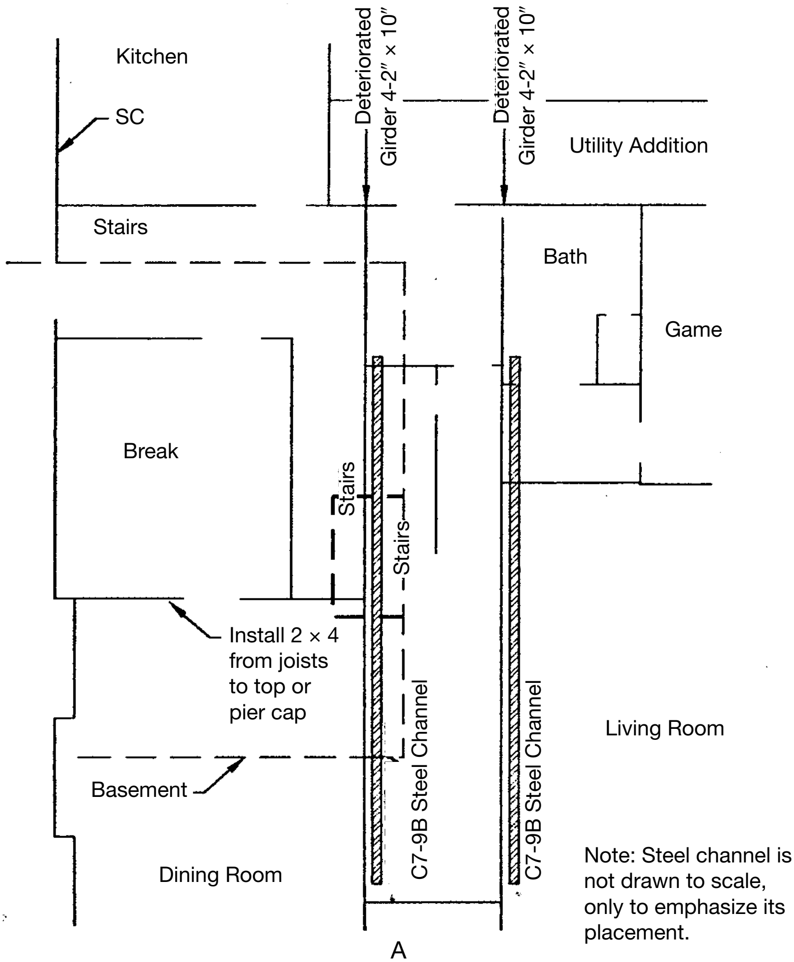


FIGURE 7B.1.6. (A) Floor plan showing placement of channel iron. (Figure continues)

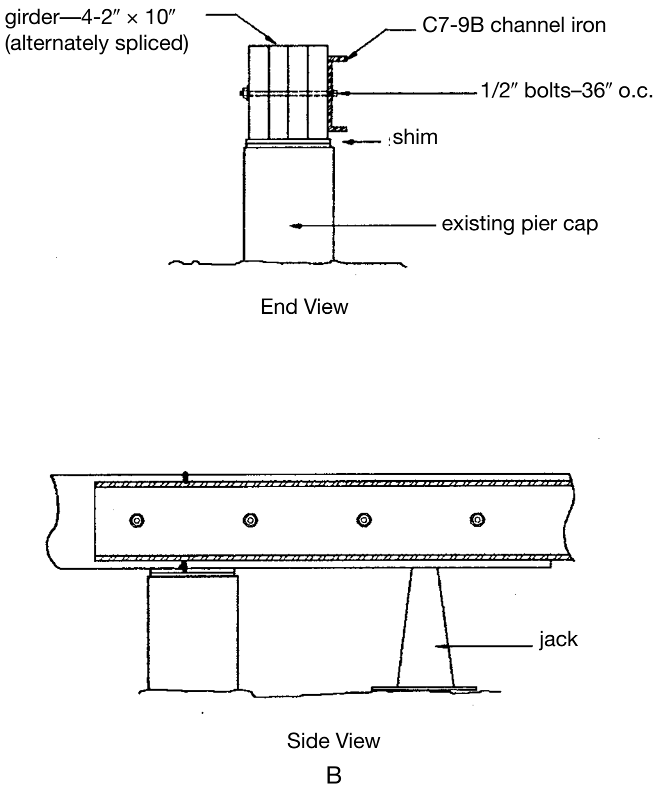


FIGURE 7B.1.6. (*continued*) (B) Installation of channel iron to reinforce wood girders.

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.

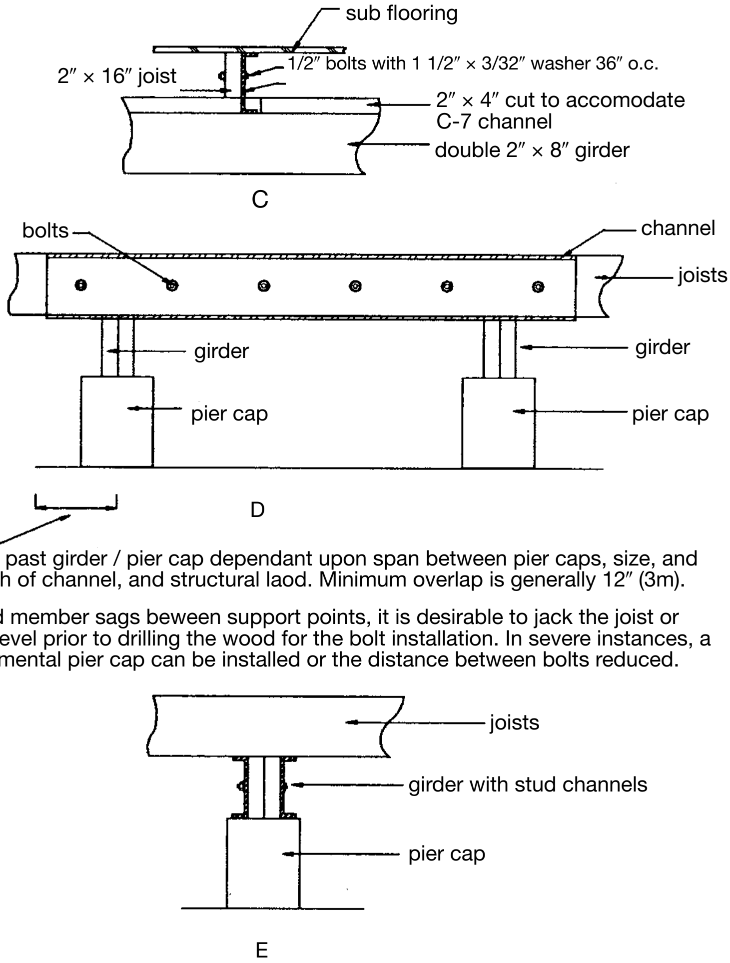


FIGURE 7B.1.6. (*continued*) (C, D, & E) Channel iron used to stiffen joists or girders.

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

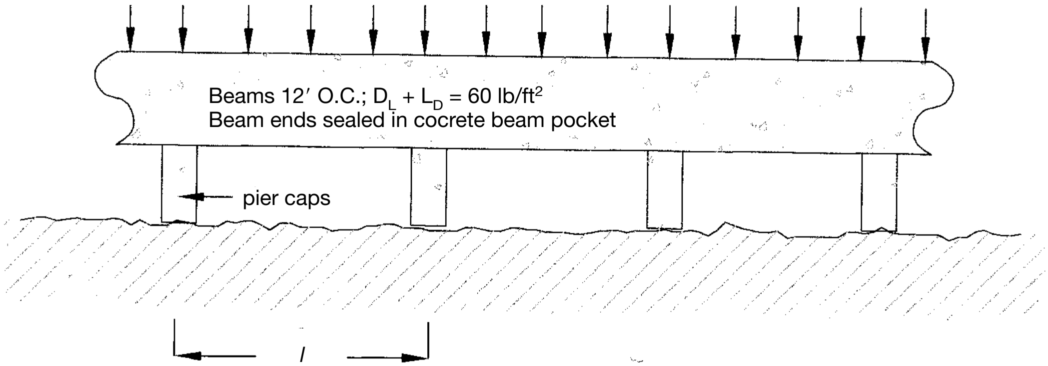


FIGURE 7B.1.7. Beam load distribution.

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

2″ × 8″ Wood
8 ft. span (96″) $(96\text{ in})^4 = 84.93 \times 10^6\text{ in}^4$

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
$f_V = 3/2\ V/A = (3/2)(3600)/10.88 = 496\text{ lb/in}^2$, fails
$F_V = 180\text{ lb/in}^2$, 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″ $(120)^4 = 207.36 \times 10^6\text{ in}^4$

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
$f_V = 3/2\ V/A = 3/2\,(4{,}500)/10.88$
$= 620.4\text{ lb/in}^2$
$f_V\,(620.4) > F_b\,(180)$ ∴ 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.

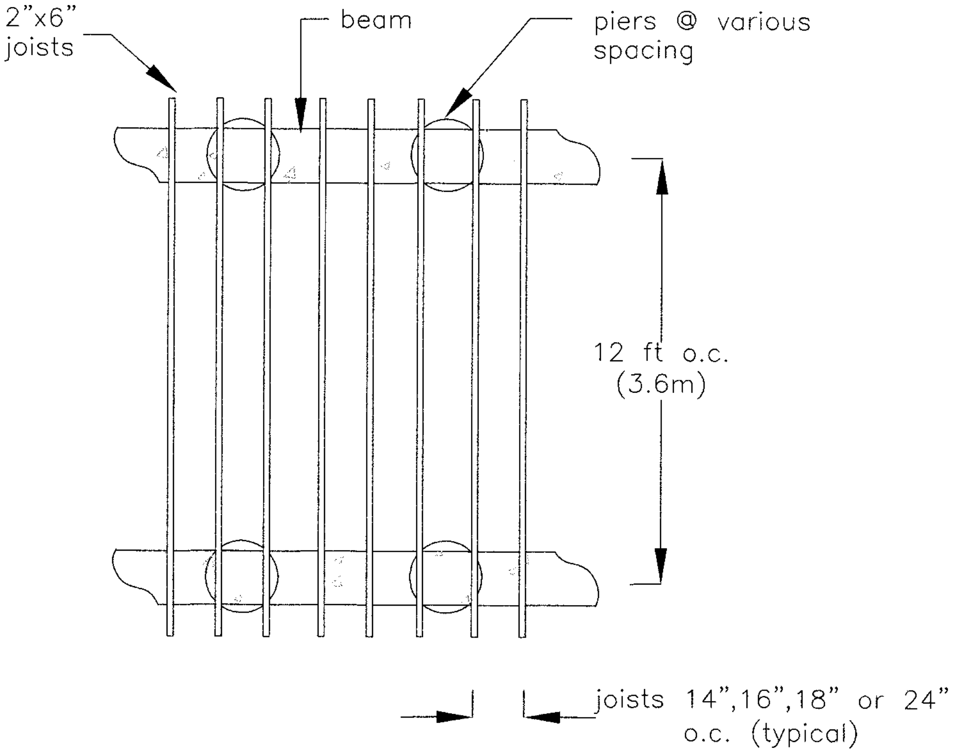


FIGURE 7B.1.8. Joists supported on beams.

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)
$\Delta = 0.828 \text{ in} \times 7.4/21.1$ or 0.290 in
0.290 in $\gg$ .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

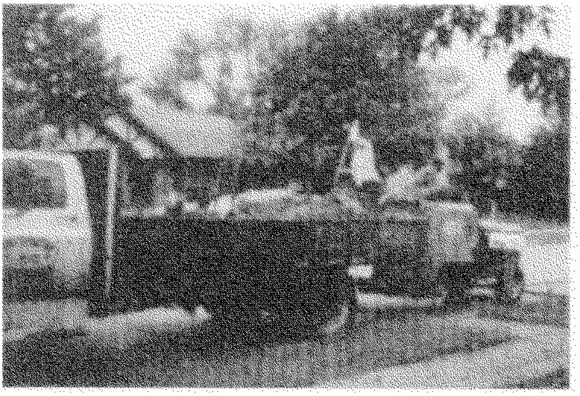
Slab foundations suffer the problems of both settlement and upheaval. The general approach to the perimeter is much the same as that described in Section 7B.4. However, due to design, mudjacking is required to fill voids and raise and stabilize the foundation.[15–17,19,23,36,60,79,86,102]

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.

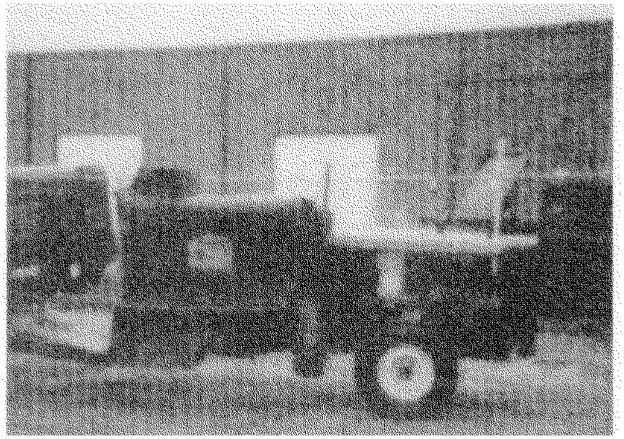
For all practical purposes, conventional deformed bar and posttensioned slab foundatons are treated as the same. In the latter case, it sometimes becomes necessary to also repair or retension defective cables.

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

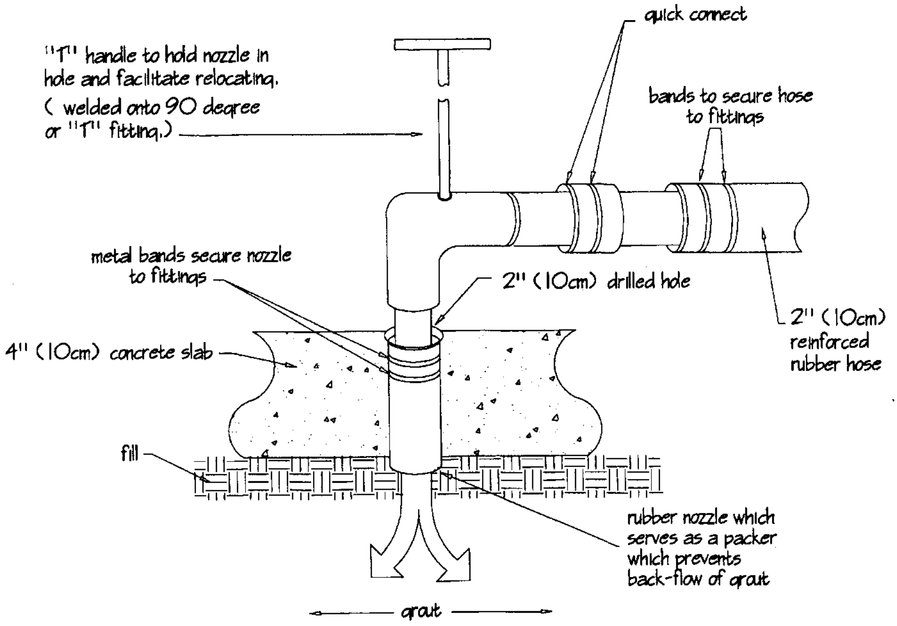


FIGURE 7B.2.2. Nozzle pack-off.

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.

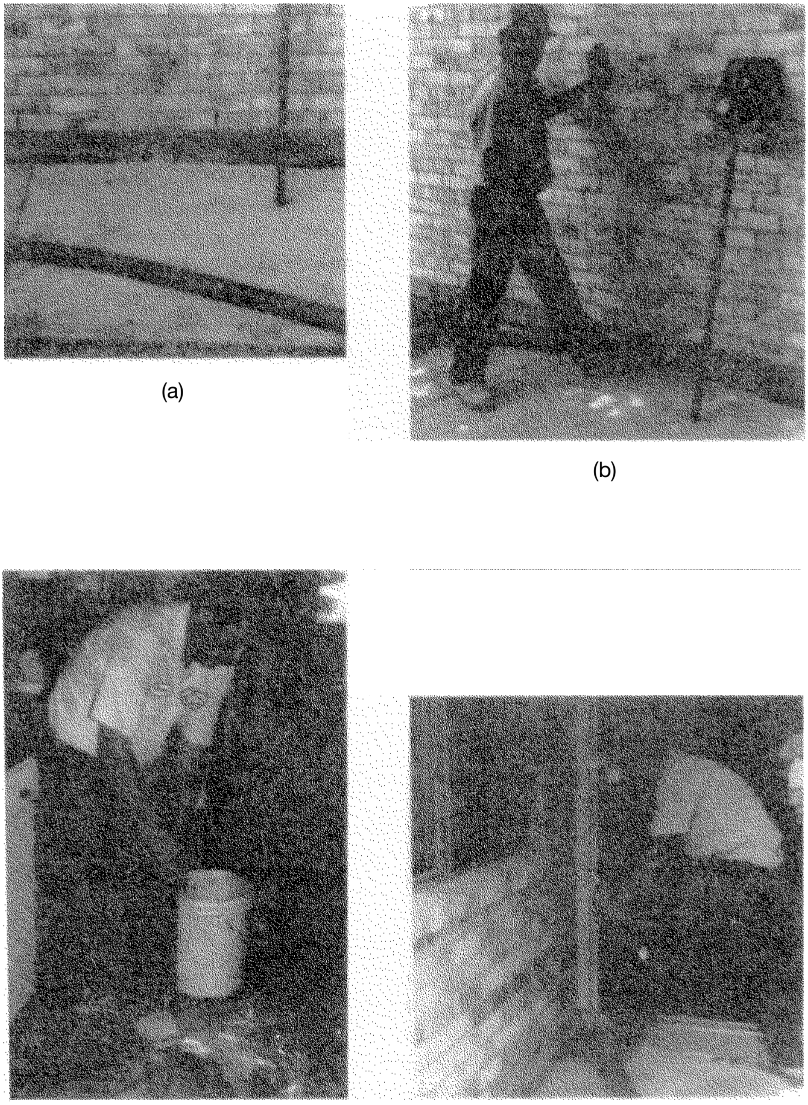
$$HHP = kBHP = \frac{Q \times P}{7.2}$$

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.

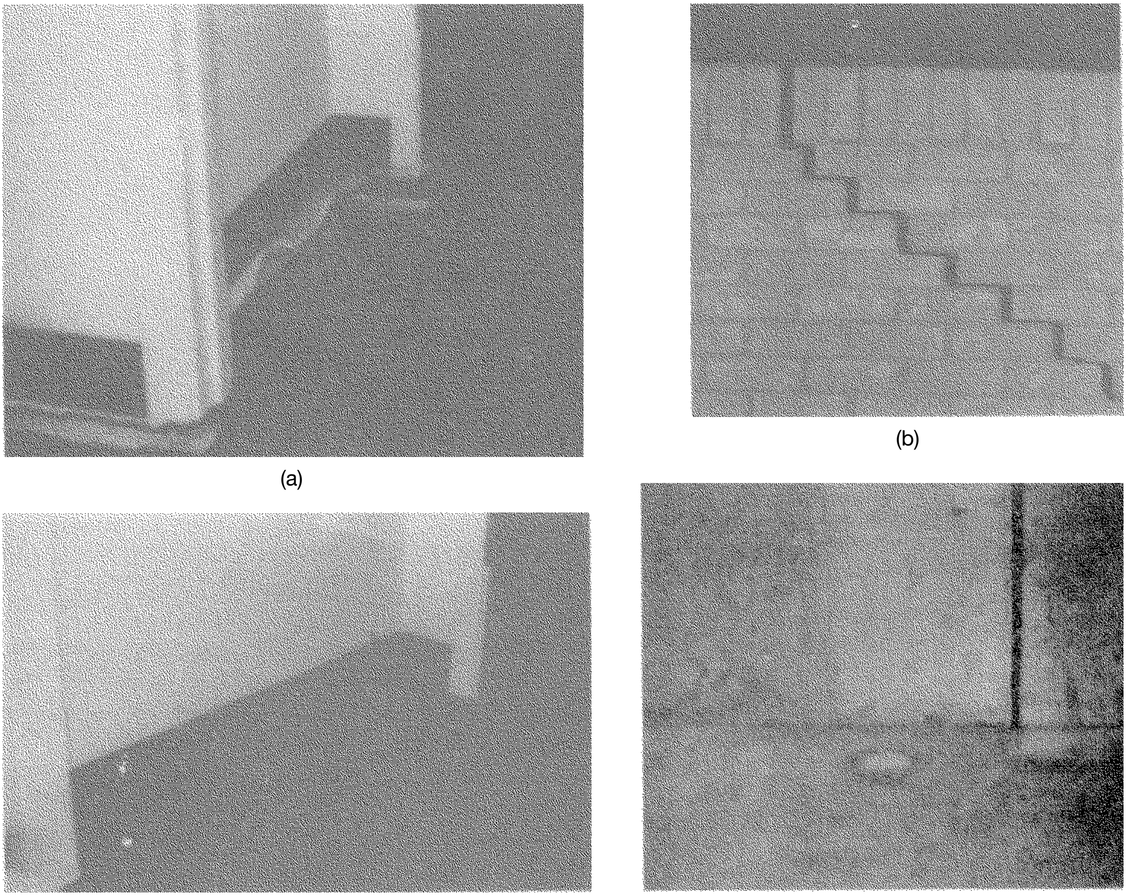
7B.2.6.1 Settlement

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-



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

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.

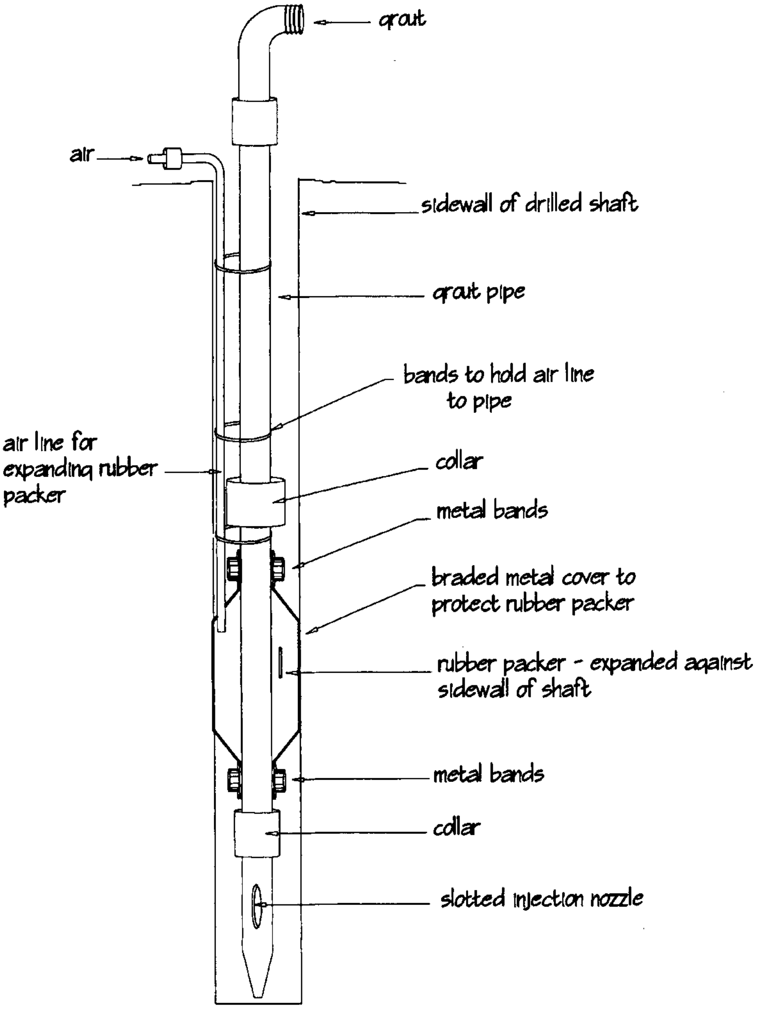
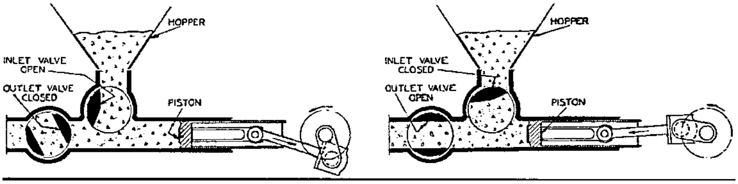


FIGURE 7B.3.1. Down hole packer for grouting.

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)

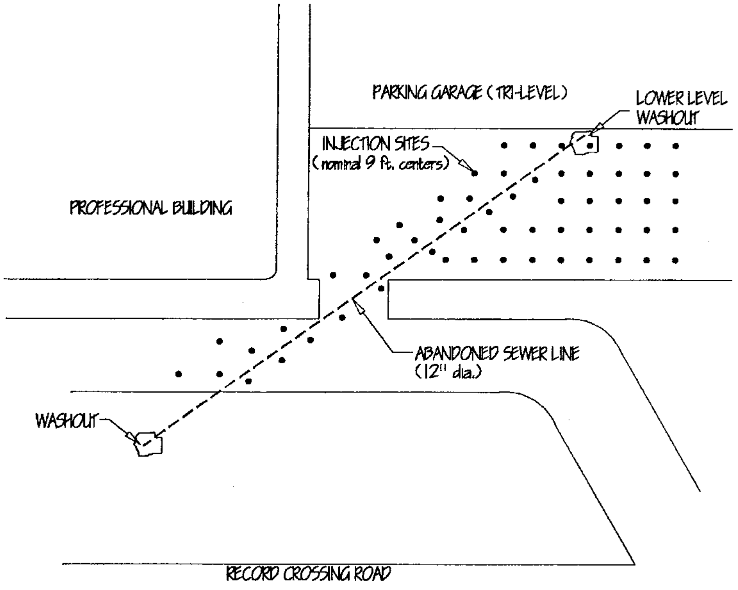


FIGURE 7B.3.3a. Dallas Hospital streets–injection pattern.

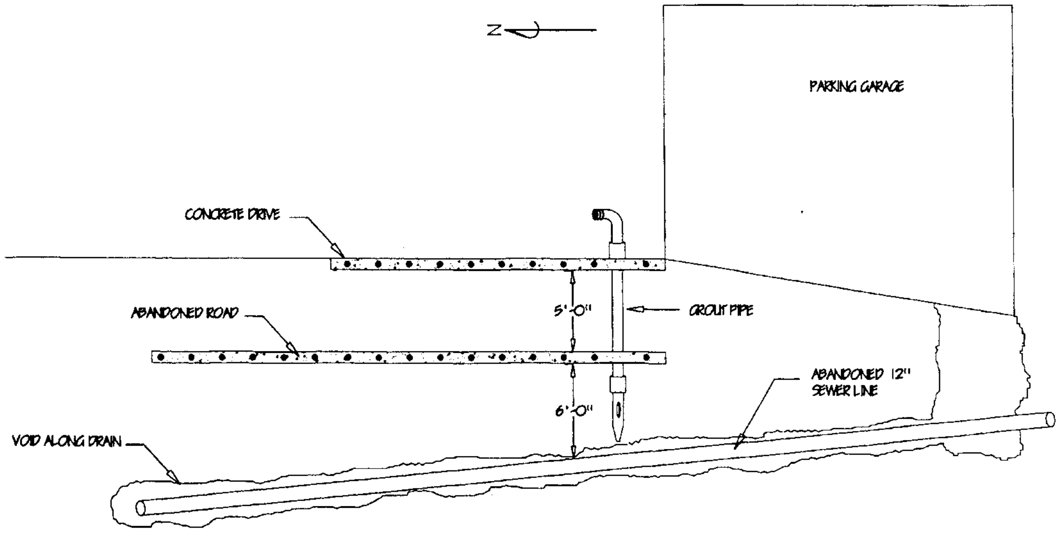


FIGURE 7B.3.3b. Hospital grout pipe placement.

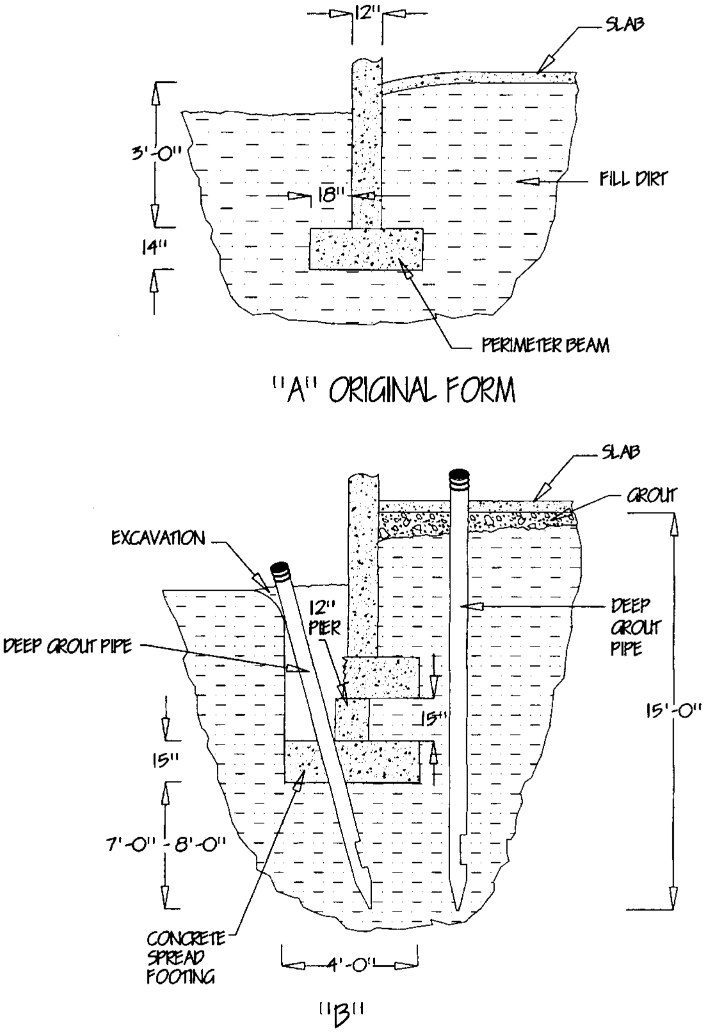


FIGURE 7B.3.4. Louisiana salt mine hoisting house project.

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

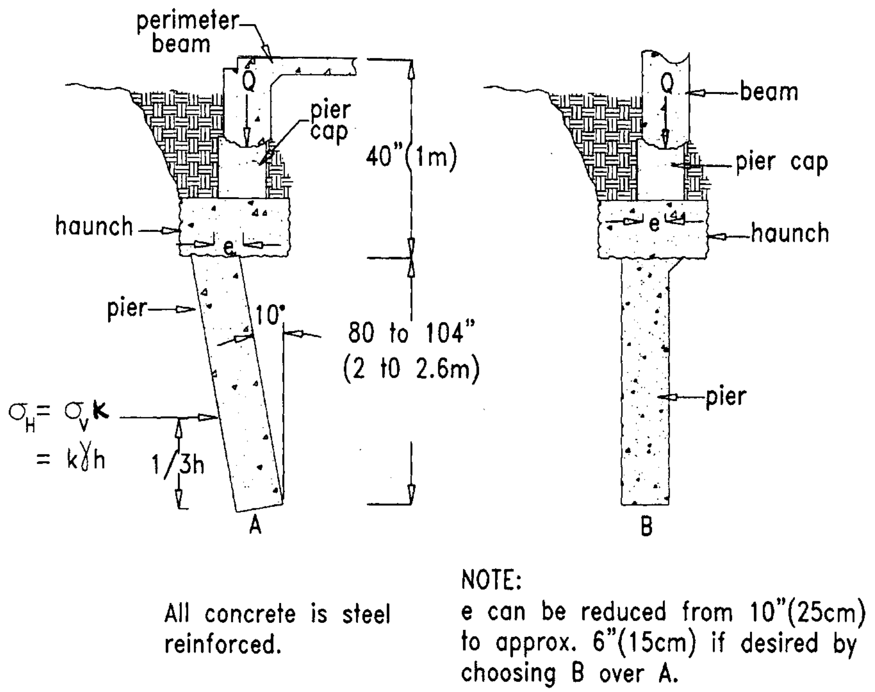


FIGURE 7B.4.1. Two acceptable drilled piers.

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	B
$Q_L = 500$ plf × 8 ft	$Q_L = 4$ k
= 4000 lb = 4 k	
$M_L = e \times Q_L = 7'' \times 4000$ lb	$M_L = 10/12$ ft × 4k = 0.833 ft × 4 k
= 28,000 in-lb = 2.3 ft-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.

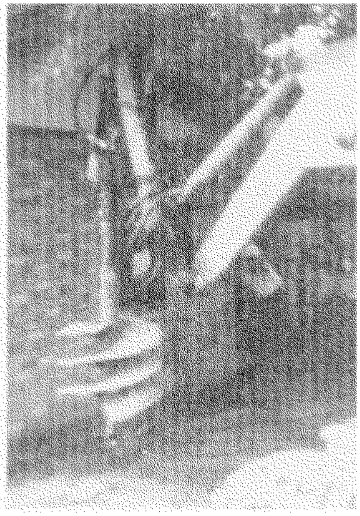
7B.4.2.1 Drilling Pier Shafts

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)

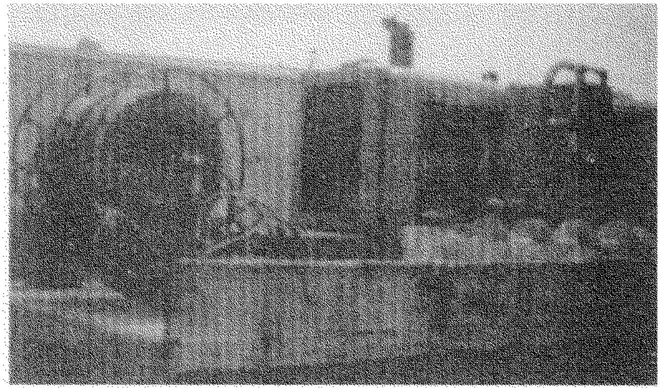
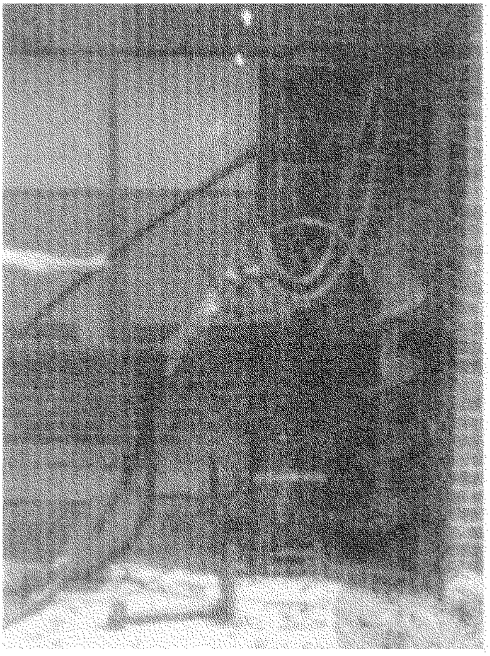

(c)

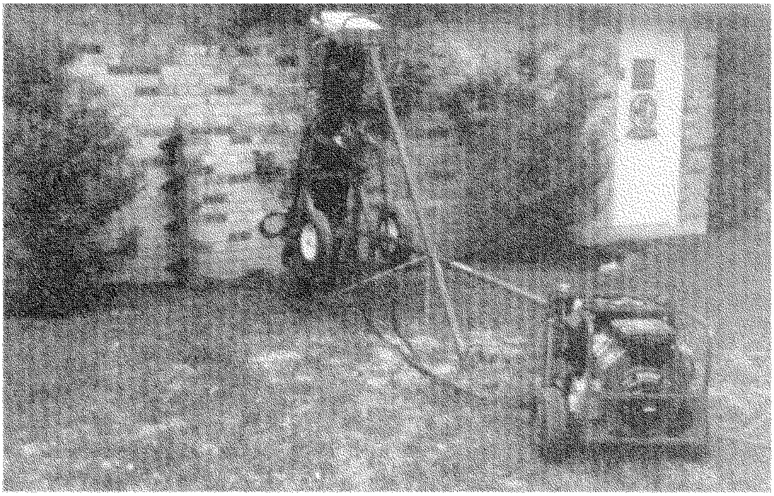
FIGURE 7B.4.2. Pier drilling equipment. (a) Truck-mounted drill; (b) tractor drill. (c1) limited-access drill.

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.

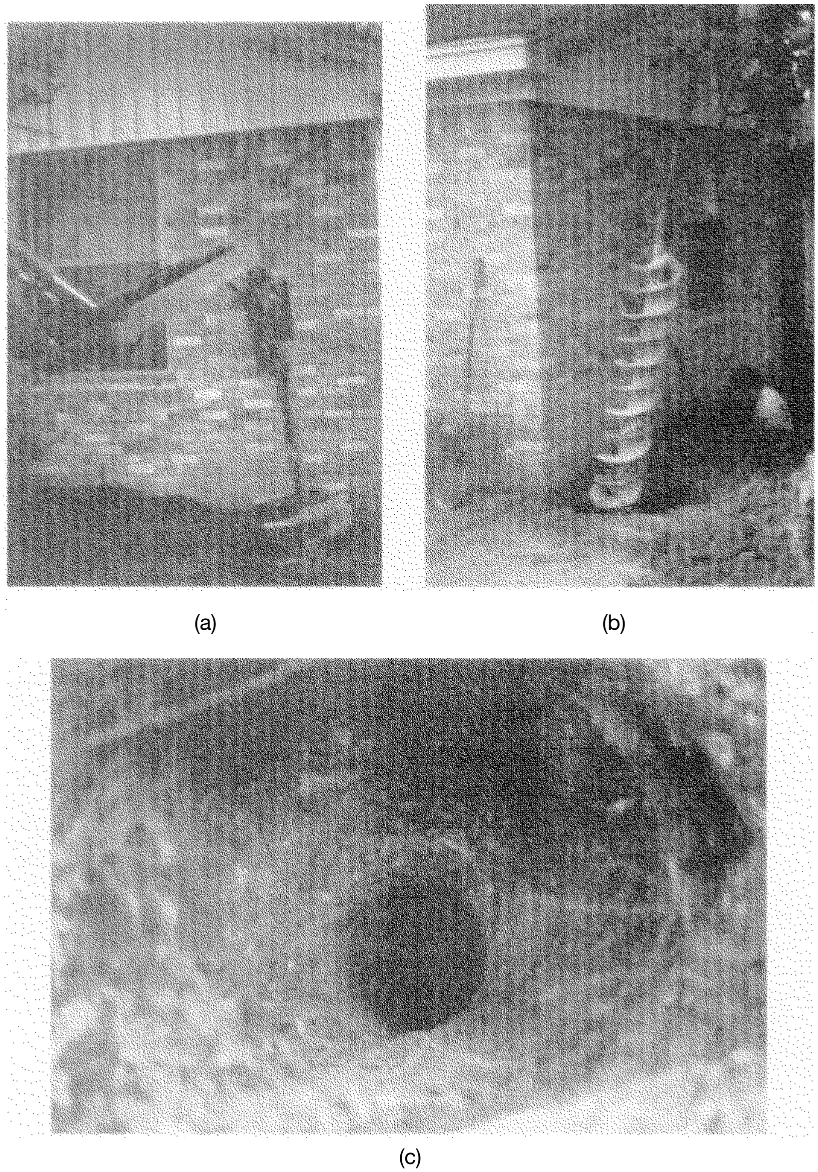
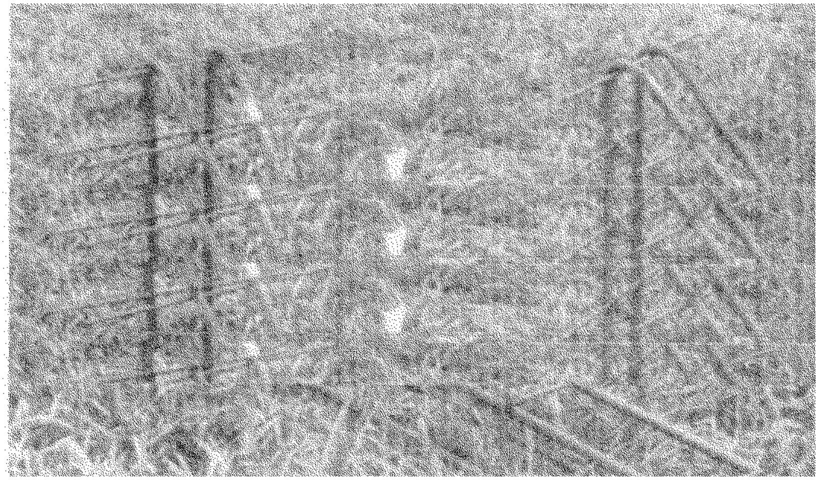


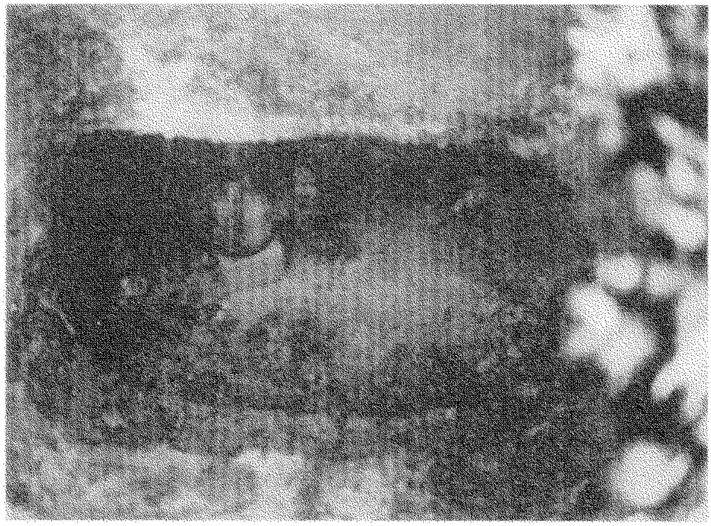
FIGURE 7B.4.3. Drilling sequence used for preparation of concrete piers for underpinning. (a) Drilling the pilot hole for haunch with 24 in bit. (b) Drilling the 12 in diameter shaft to 15 ft depth. The man at right is enlarging and shaping the haunch. (c) Haunch and pier ready for reinforcing steel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



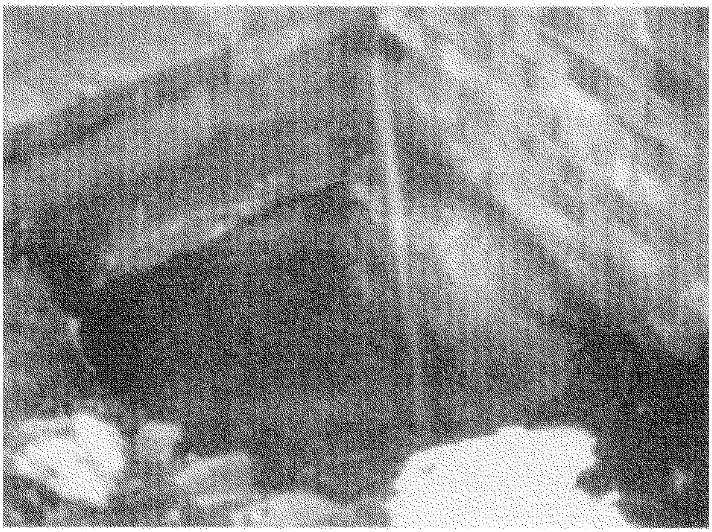
(a)

(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 (3/8 in or 0.9 cm)	
Pier:	4 × 10 ft + 4 × 2′ (ties)	= 48 ft
Haunch:	12 × 1.5 ft	= 18
Pier Cap:	1 × 1.5 ft	= $\underline{1.5}$
		67.5 ft (20 m)

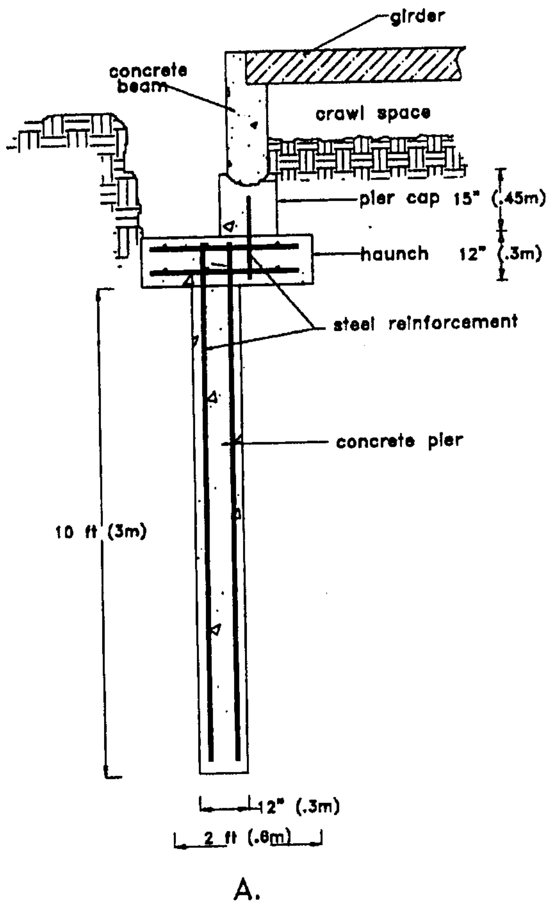


FIGURE 7B.4.6. Sustaining the perimeter beam.

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-

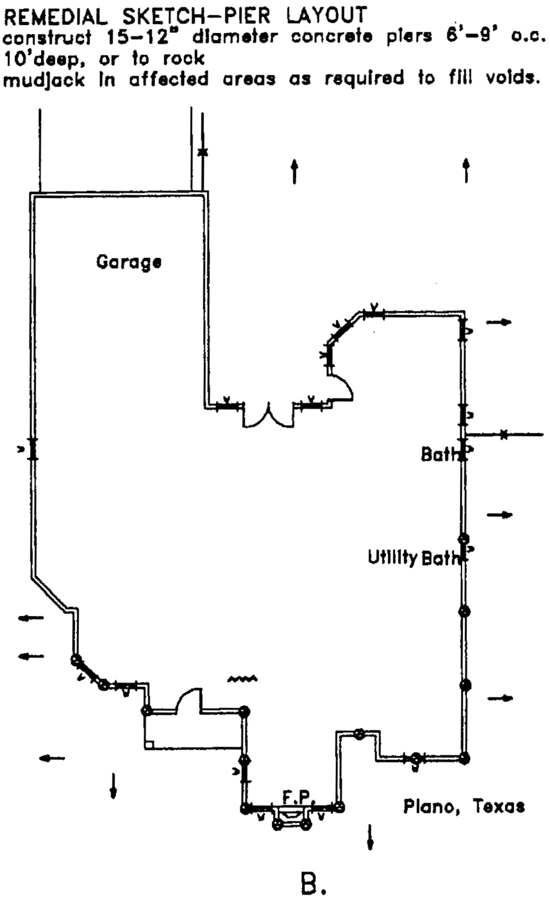


FIGURE 7B.4.6. (*continued*). Raising the perimeter beam.

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.

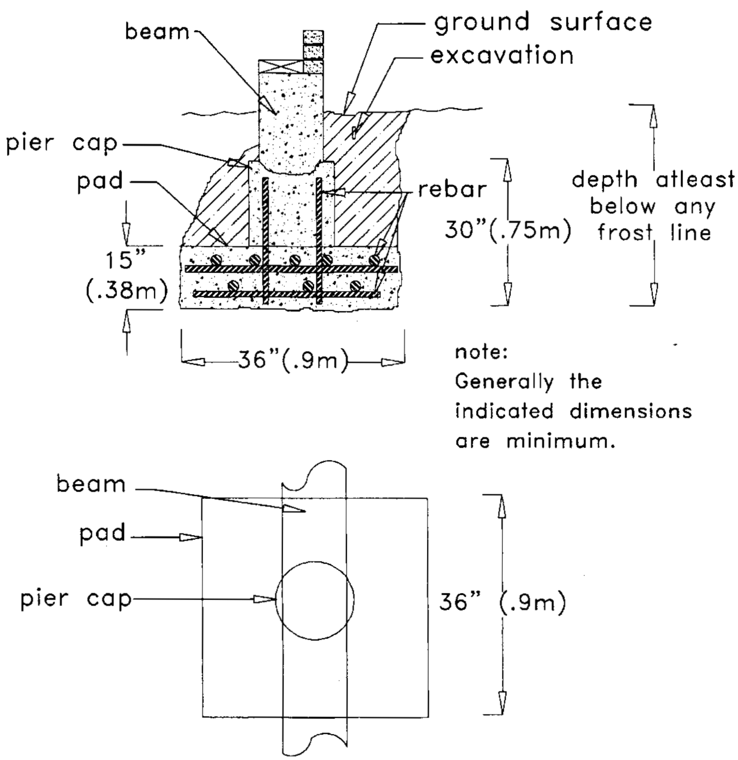
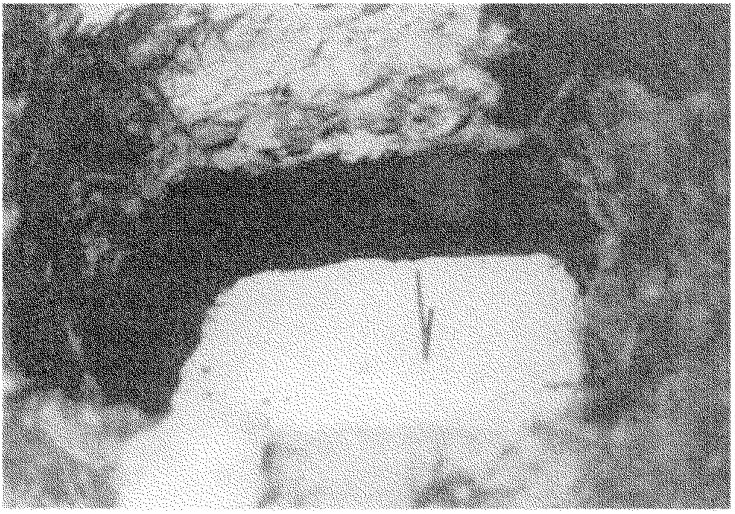


FIGURE 7B.4.7. Spreadfooting.

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:	$2\frac{1}{2}$ ft × $2\frac{1}{2}$ 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.

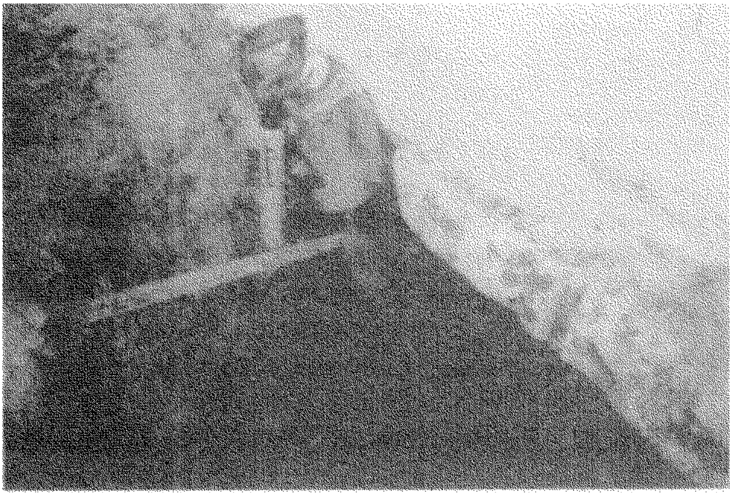
The first pile section would be forced outward (away from load Q_L) due to greater moment (2475 ft-lb vs 84 ft-lb resistance, Q_F). The second joint of pipe would then push the first joint at an outward angle with a vectored vertical force (resultant). The mathematical analysis suddenly becomes more 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.



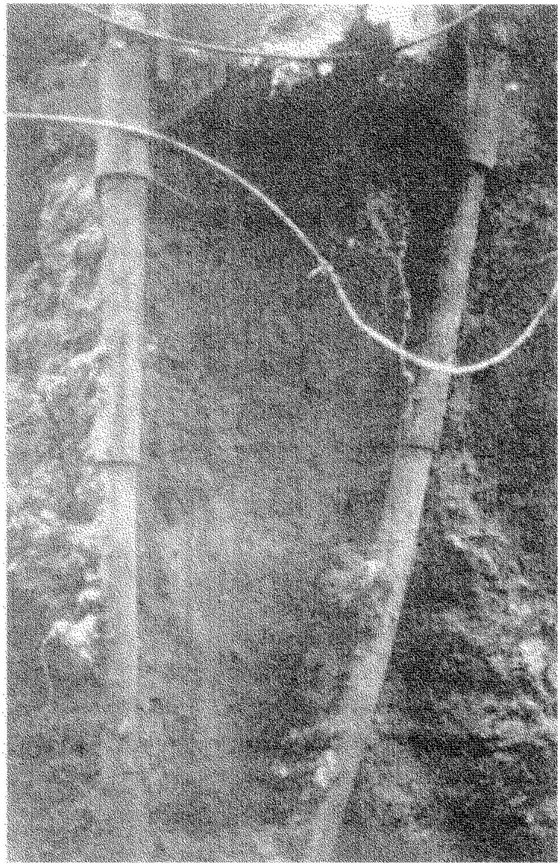
(a)

(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

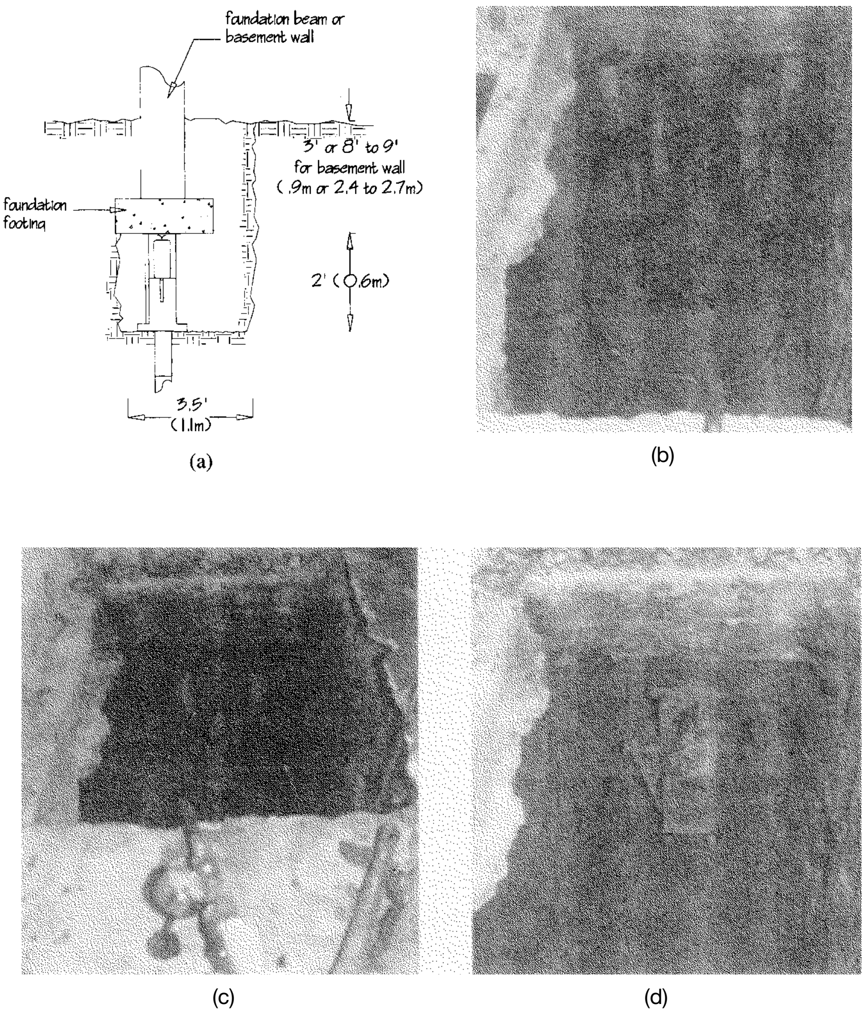


FIGURE 7B.4.11. Concentric pile technique. (a) Drawing of equipment in place; (b) photo of equipment in place; (c) photo of beam being raised; (d) beam and pier locked in place.

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").

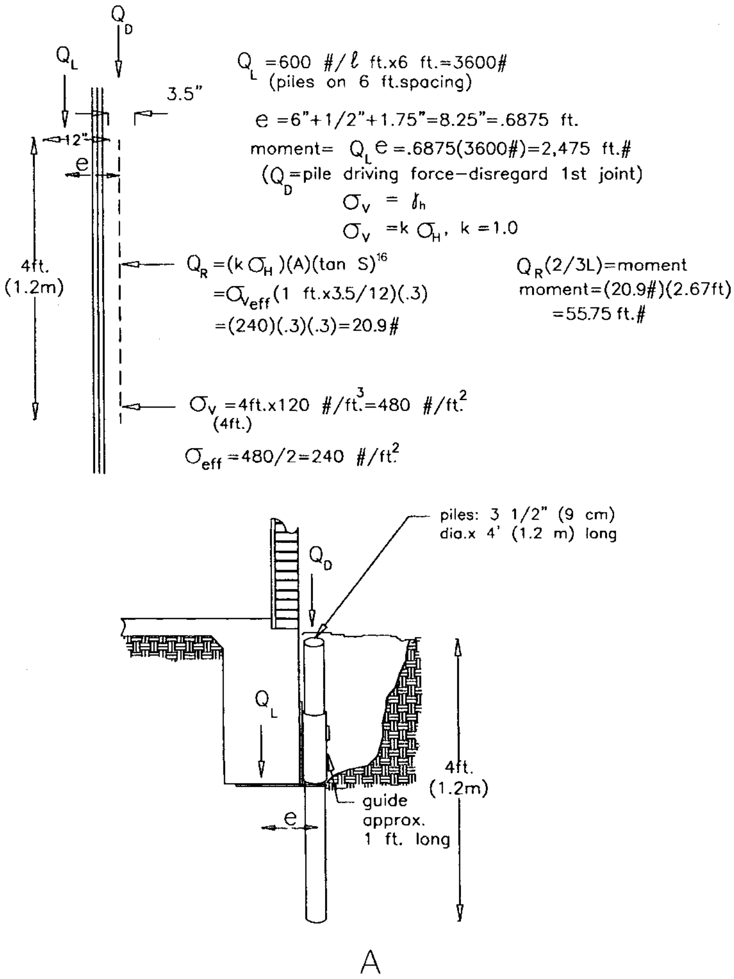


FIGURE 7B.4.12. Eccentrically driven piles.

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."

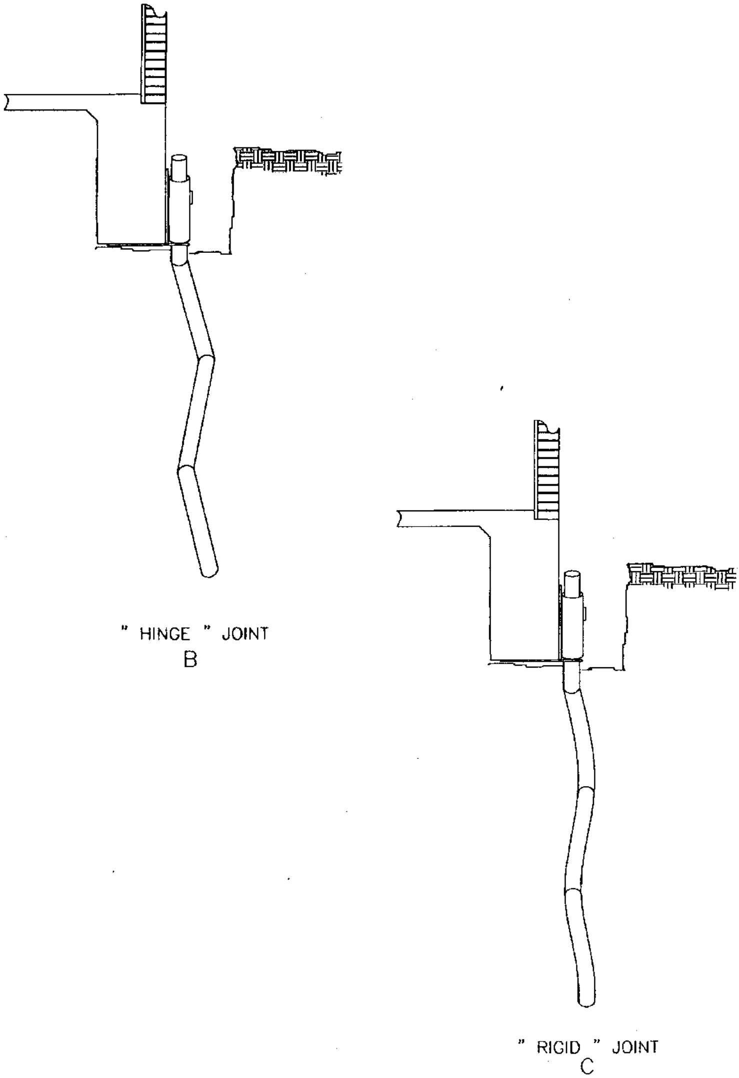


FIGURE 7B.4.12. (*continued*). Eccentrically driven concrete piles.

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

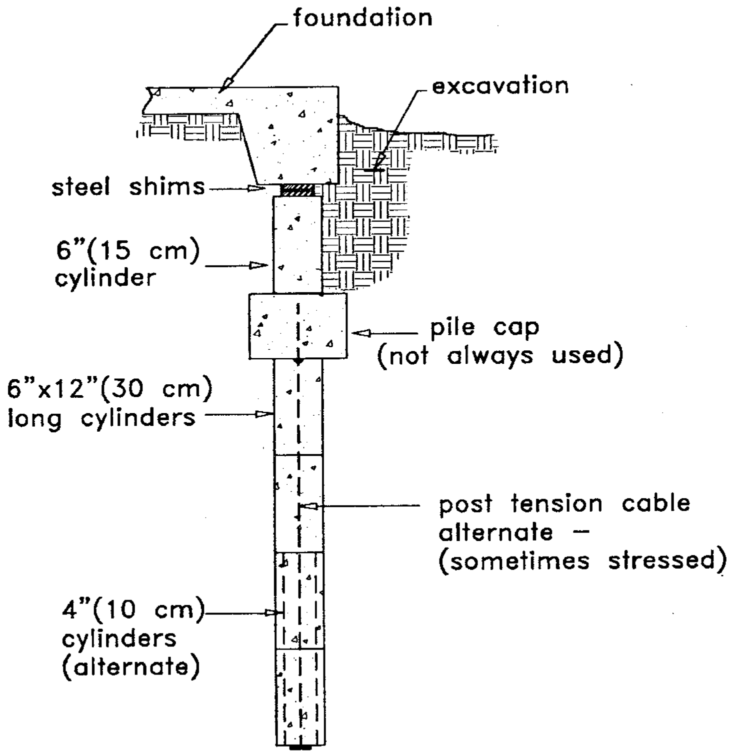


FIGURE 7B.4.13. Hydraulically driven concrete pilings.

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.]

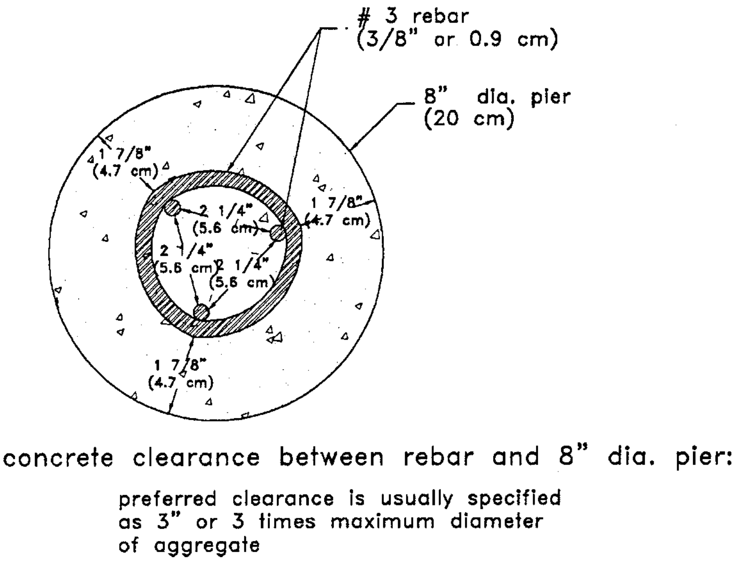


FIGURE 7B.4.14. Poured concrete slim pier.

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

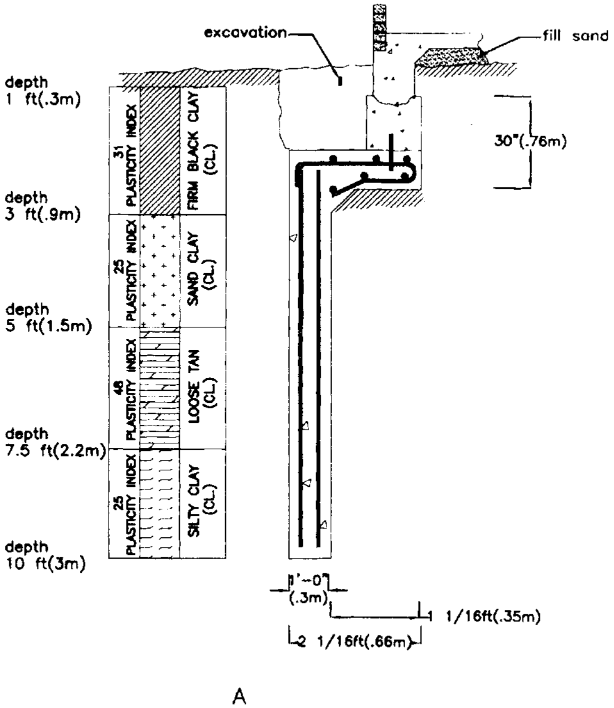


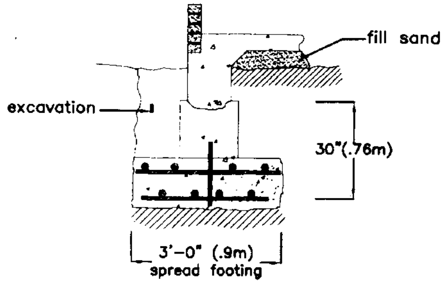
FIGURE 7B.4.15. (A) Conventional pier.

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.

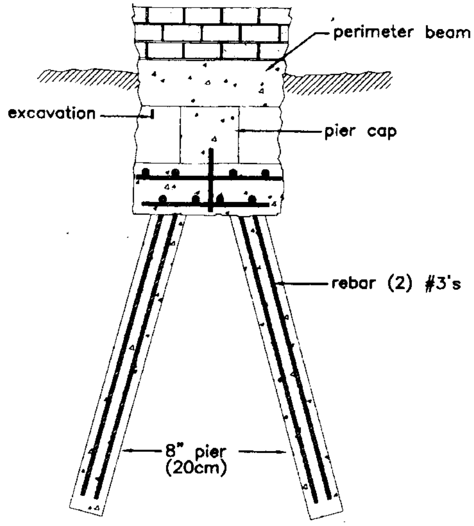
When dual piers are used, several alterations are required. First, the pier diameters are generally limited to about 8″ (20 cm). Second, the dual piers must be raked (deviated from vertical). This rake should never exceed 12°, but in practice often exceeds 20°.[37] The small diameter pier introduces 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:



B



C

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

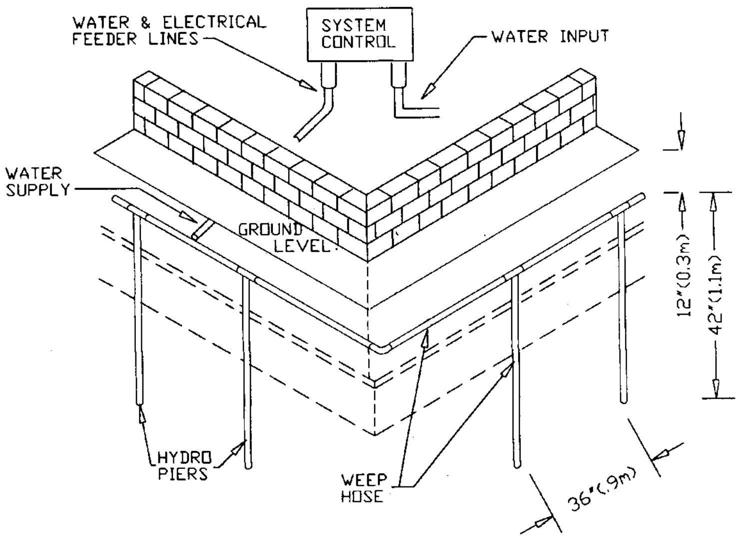


FIGURE 7B.4.16. Typical installation of hydropiers.

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.

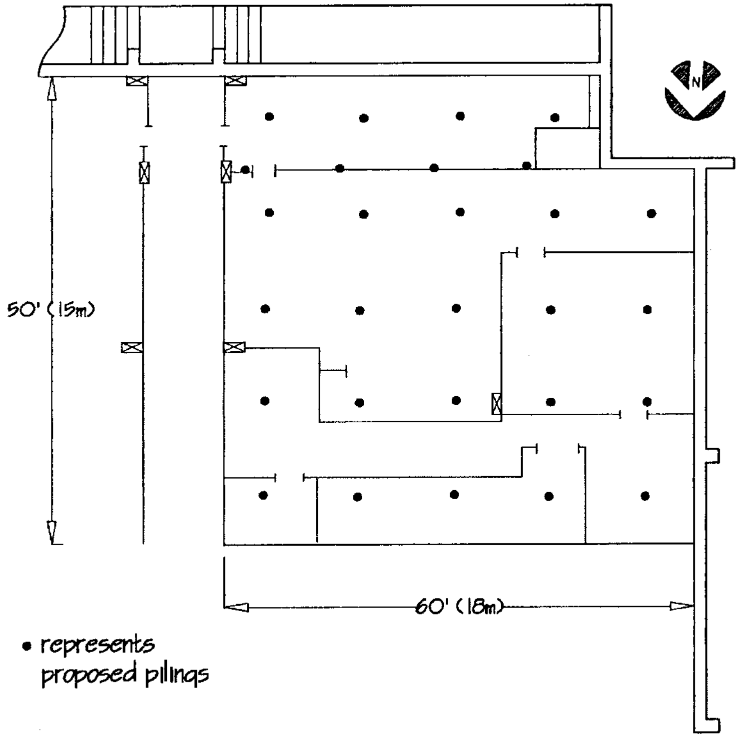


FIGURE 7B.4.17. Underpinning a slab foundation versus mudjacking.

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 $A = 0.785D^2$

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

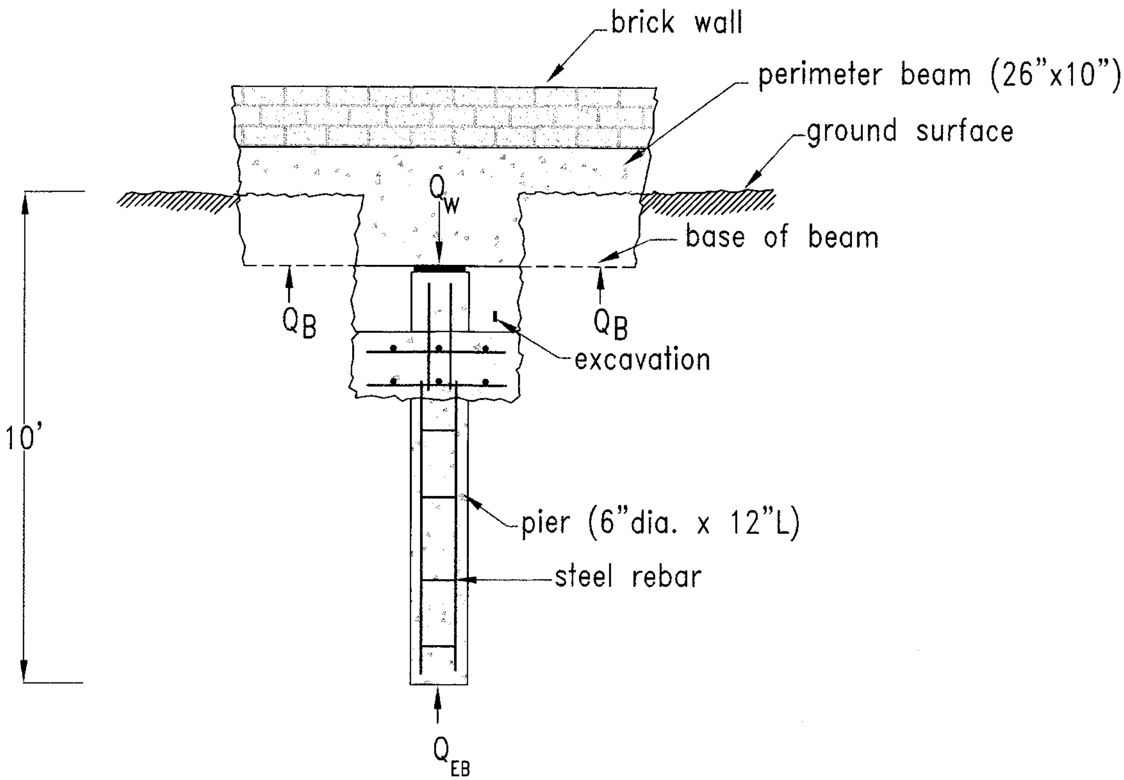


FIGURE 7B.4.18. Soil load capacity.

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:

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

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

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

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 $M = wl^2/\ 120$ in-lb $= 1083(96)^2/120 = 83{,}174$ in-lb
$f_B = M/S_X = 83{,}174/506.2 = 163$ psi
$F_B = 1350$ psi

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.

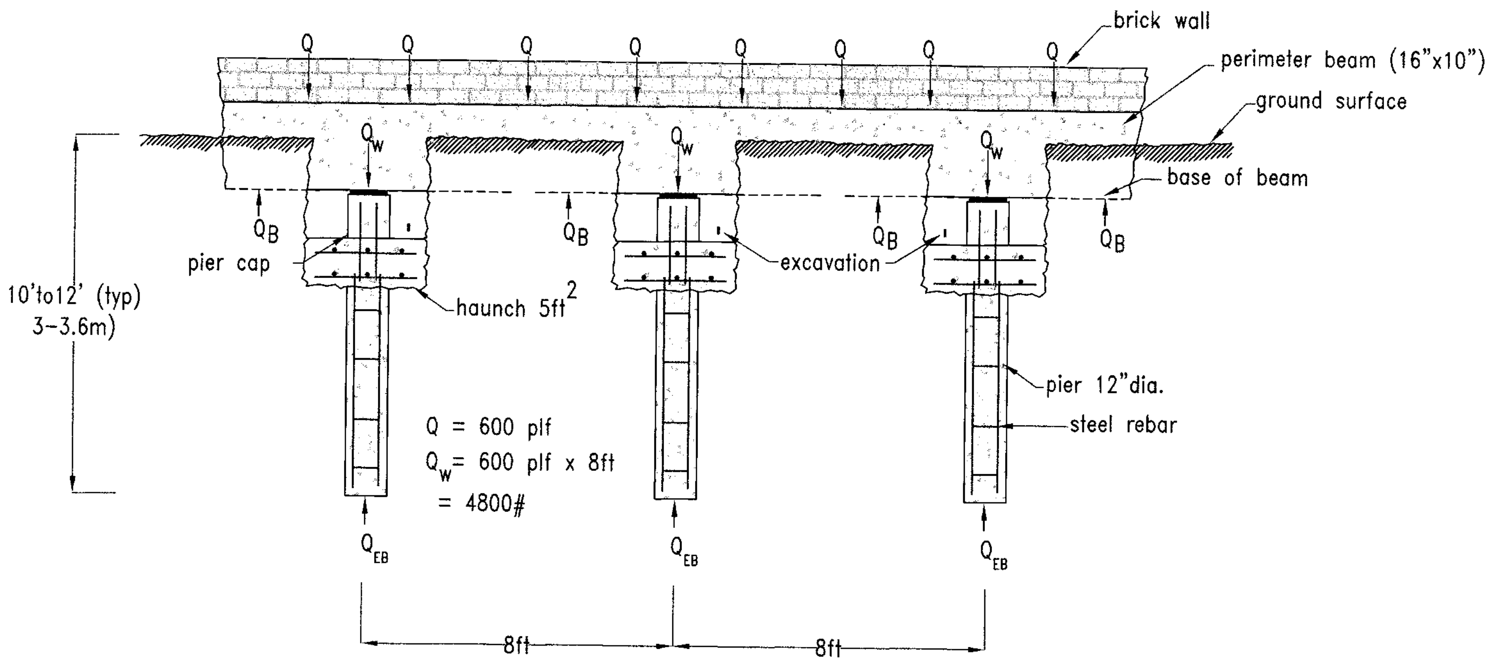


FIGURE 7B.4.19. Load evaluation of concrete beam during underpinning. Note: During the raise, jacks are set on the haunch and used to lift the beam. At, or near, the conclusion of the raise, the concrete beam is not likely to have contact with the supportive soil ($Q_B = 0$). (This contact is ultimately 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)
$(120)^4 = 207.36 \times 10^6$
$= 1083(120)^4/1743(3 \times 10^6)(3413) =$
$\Delta = (1083)(207.36 \times 10^6)/(1743)(3 \times 10^6)3413 = 0.01'' < 0.267''$
∴ OK in deflection

Stability Check:

D/b = 16/10 = 1.6 < 2 ∴ 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 $< 0.267''$
∴ 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 ∴ 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) ∴ 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 ∴ 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.

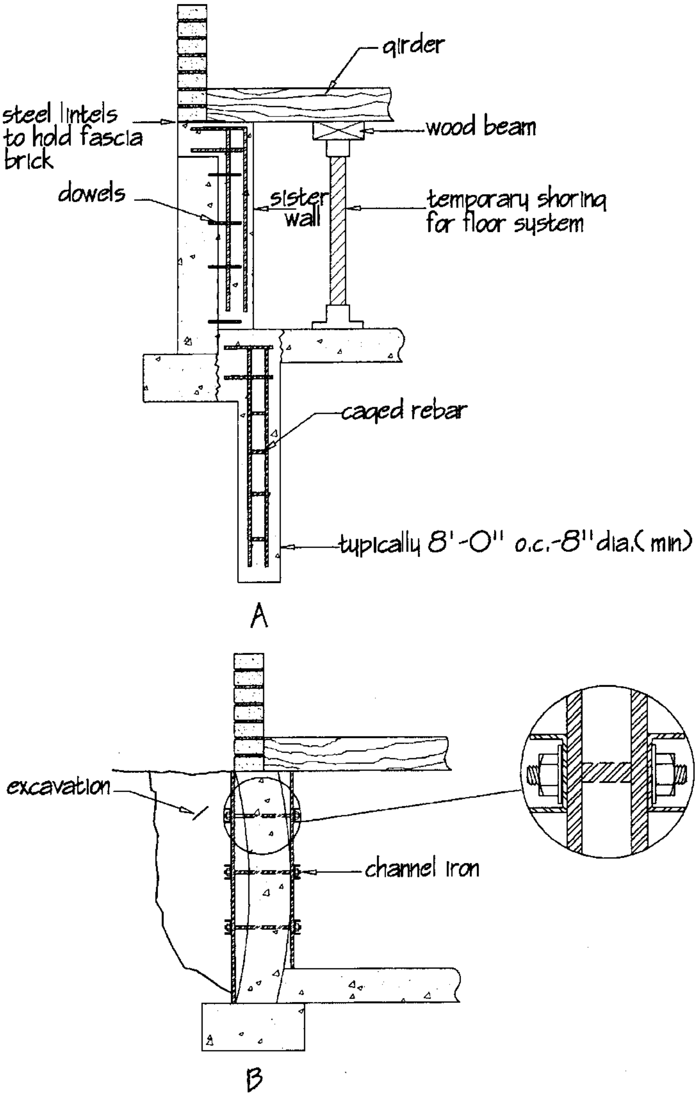


FIGURE 7B.5.1. Basement/foundation wall repair. (a) Failure in foundation (basement) wall, transfer or load; (b) failure in foundation wall, plumbing desired.

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

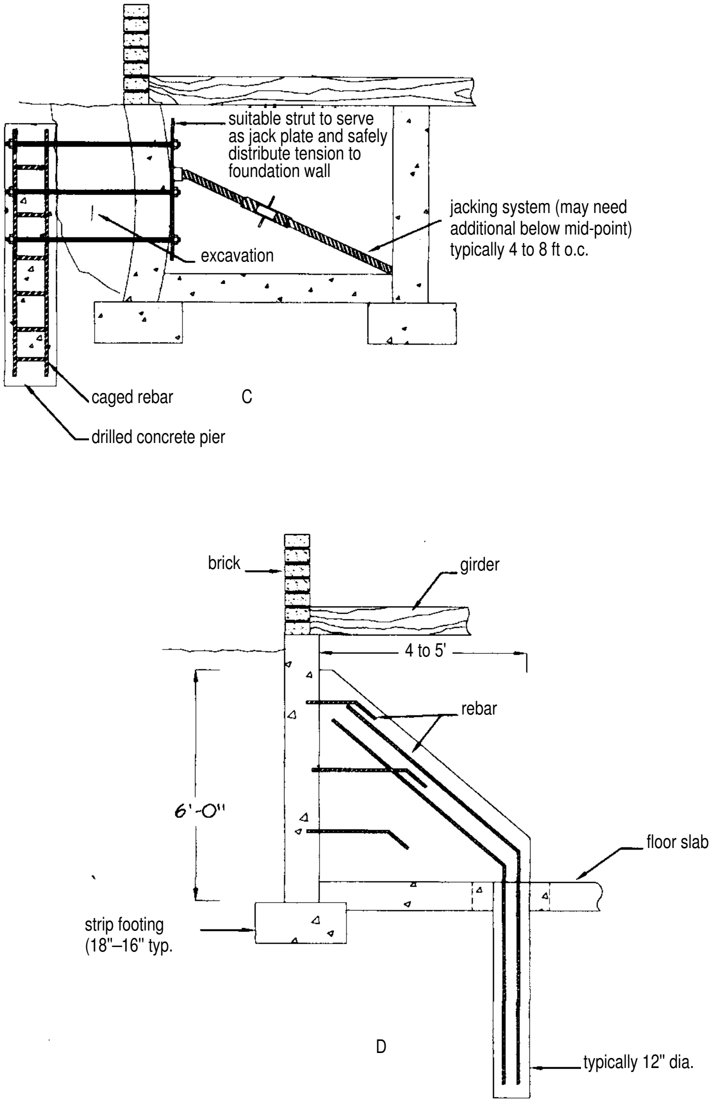


FIGURE 7B.5.1. (*continued*). Basement/foundation wall repair. (c) Failure of basement wall, plumbing desired; (d) knee brace to control rotation.

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

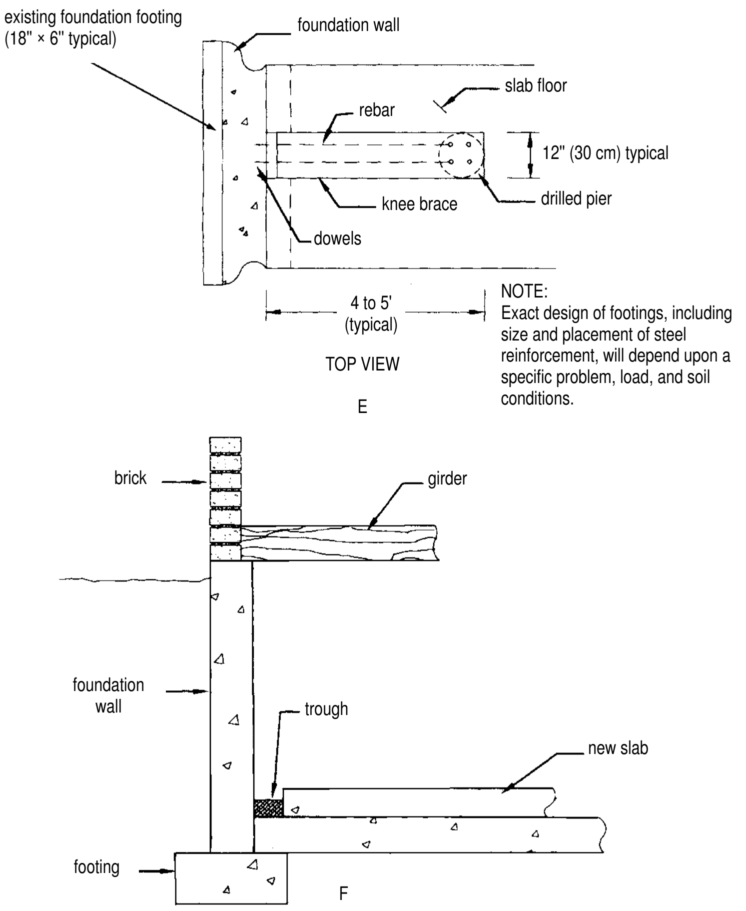


FIGURE 7B.5.1. (*continued*). Basement/foundation wall repair. (e, f) Water control.

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.

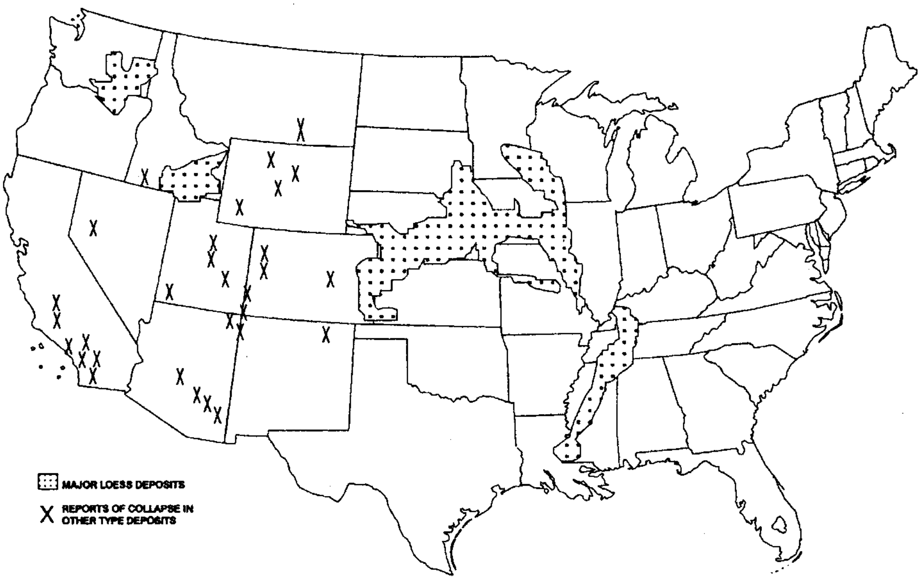


FIGURE 7B.6.1. Location of major loess deposits in the United States. (Adapted from Dudley, 1970; used with permission of ASCE.)

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.

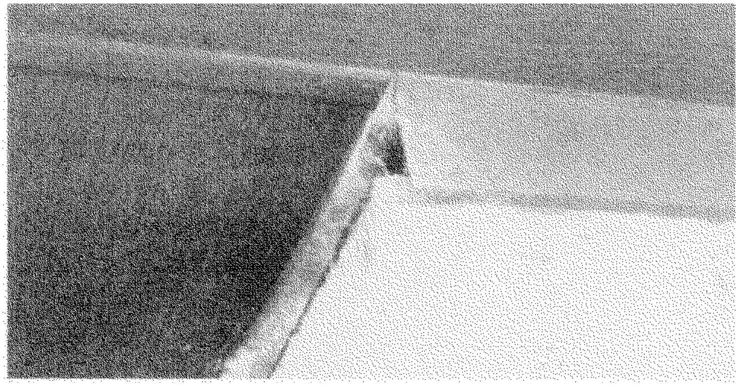
7B.6.7.2 Inorganic Chemicals

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 ($3CaO$-Al_2O_3-$3CaSO_4$-32

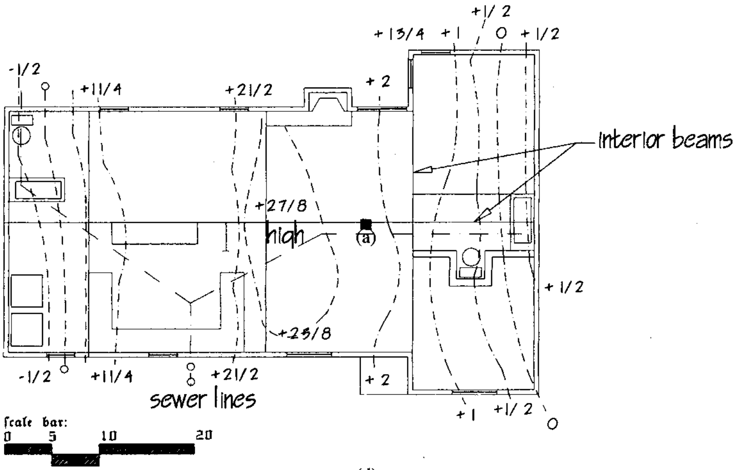
(a)

(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.

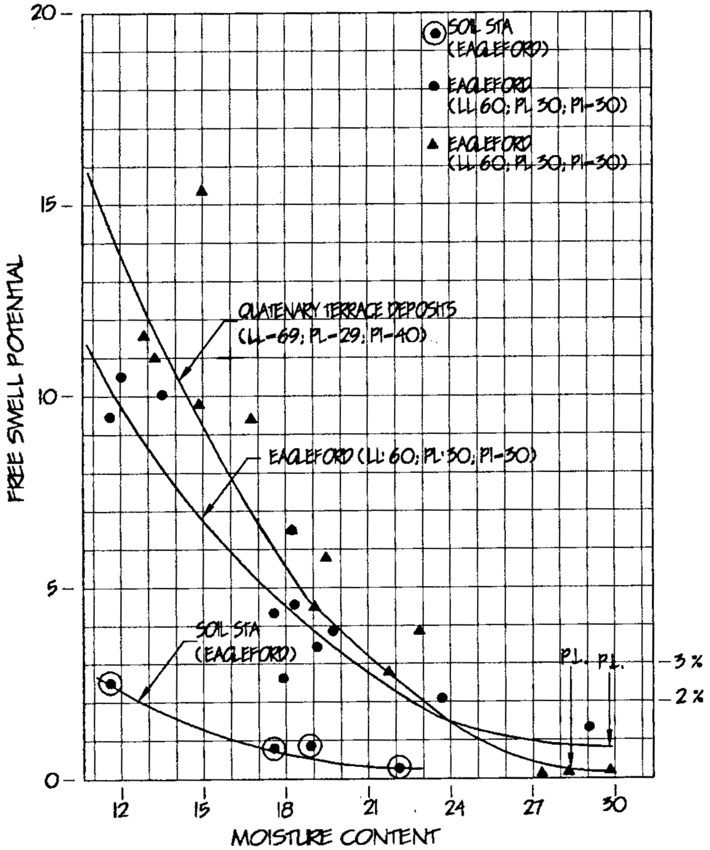


FIGURE 7B.6.3. Free swell versus moisture content.

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-

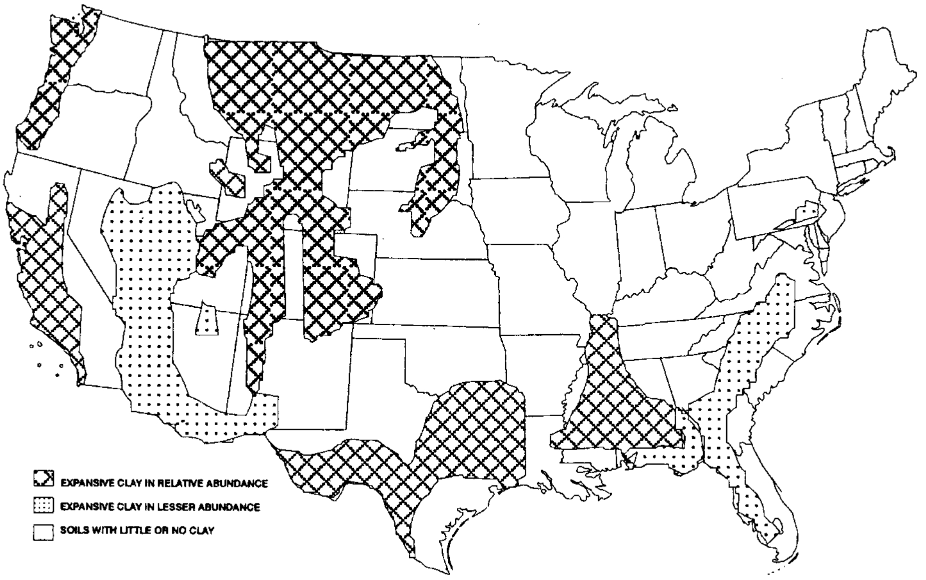


FIGURE 7B.6.4. Distribution of expansive soils in the United States.[57]

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.

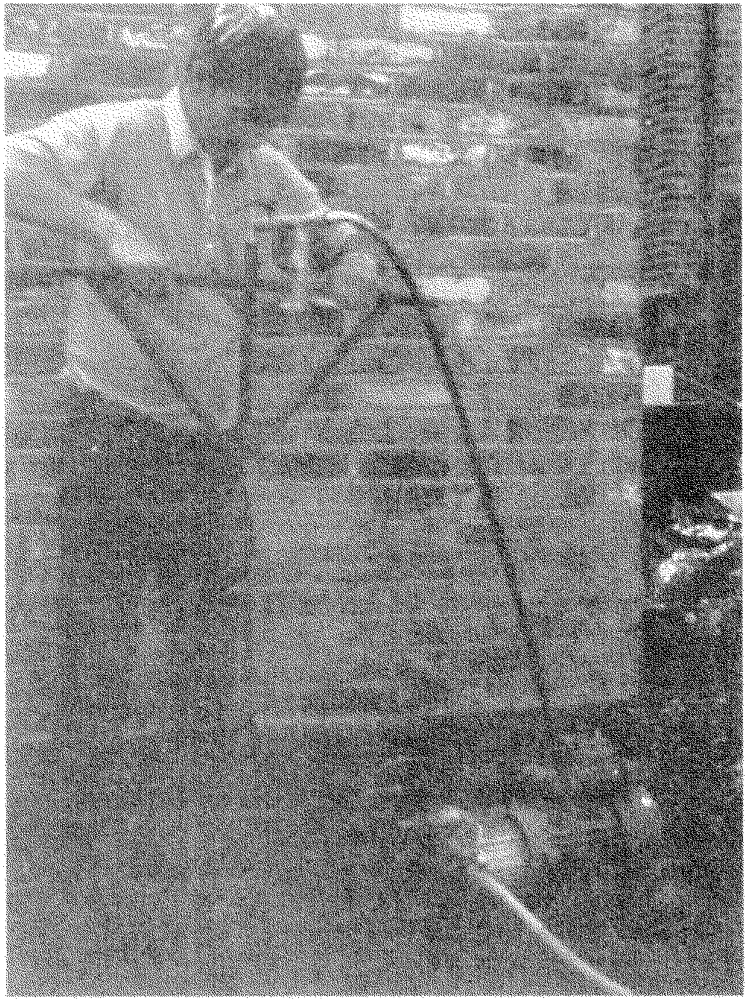


FIGURE 7B.6.5. Chemical injection through a specially designed stem.

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.

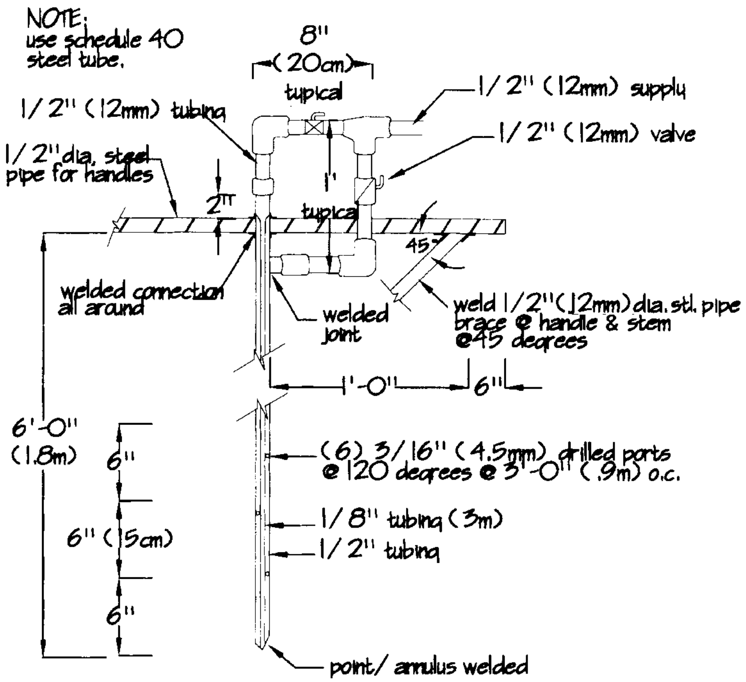


FIGURE 7B.6.6. Pressure injection stem.

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

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

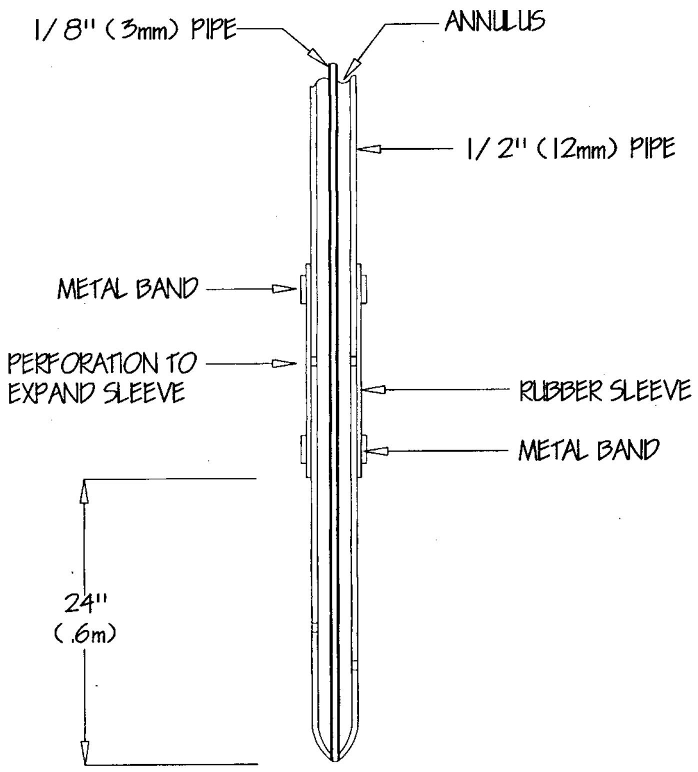


FIGURE 7B.6.7. Injection stinger with pack-off

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

$$V_p = Q/A_pC_D$$

where $A_p = D_p{}^2n/4$. 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.

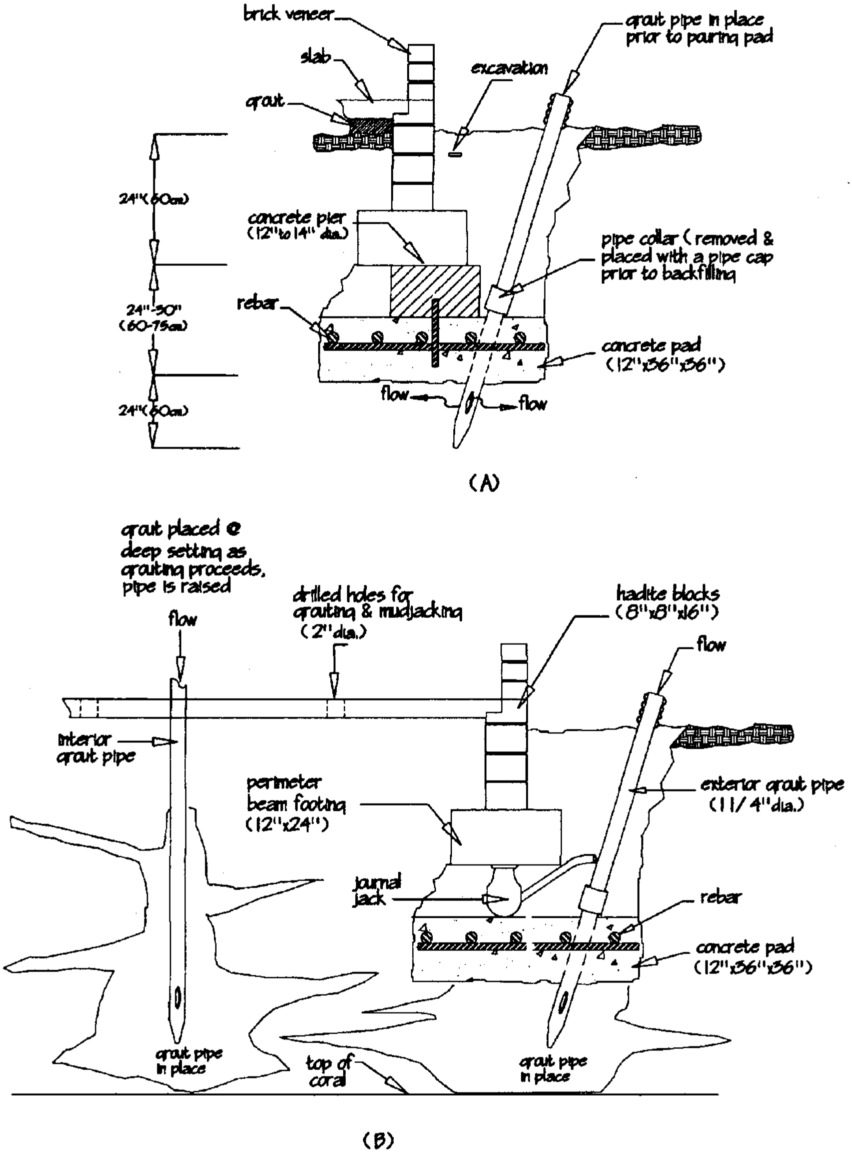


FIGURE 7B.7.1. Florida repair. (a) Grout pipe in place; (b) typical deep grouting procedure.

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

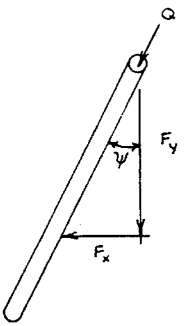
The field photographs in Figures 7B.4.9 and 7B4.10 show steel pipe placed by a hydraulic driver and pinned by bolts through the lift bracket. Figure 7B.5.9a shows the pipe slanted inward at over

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.)

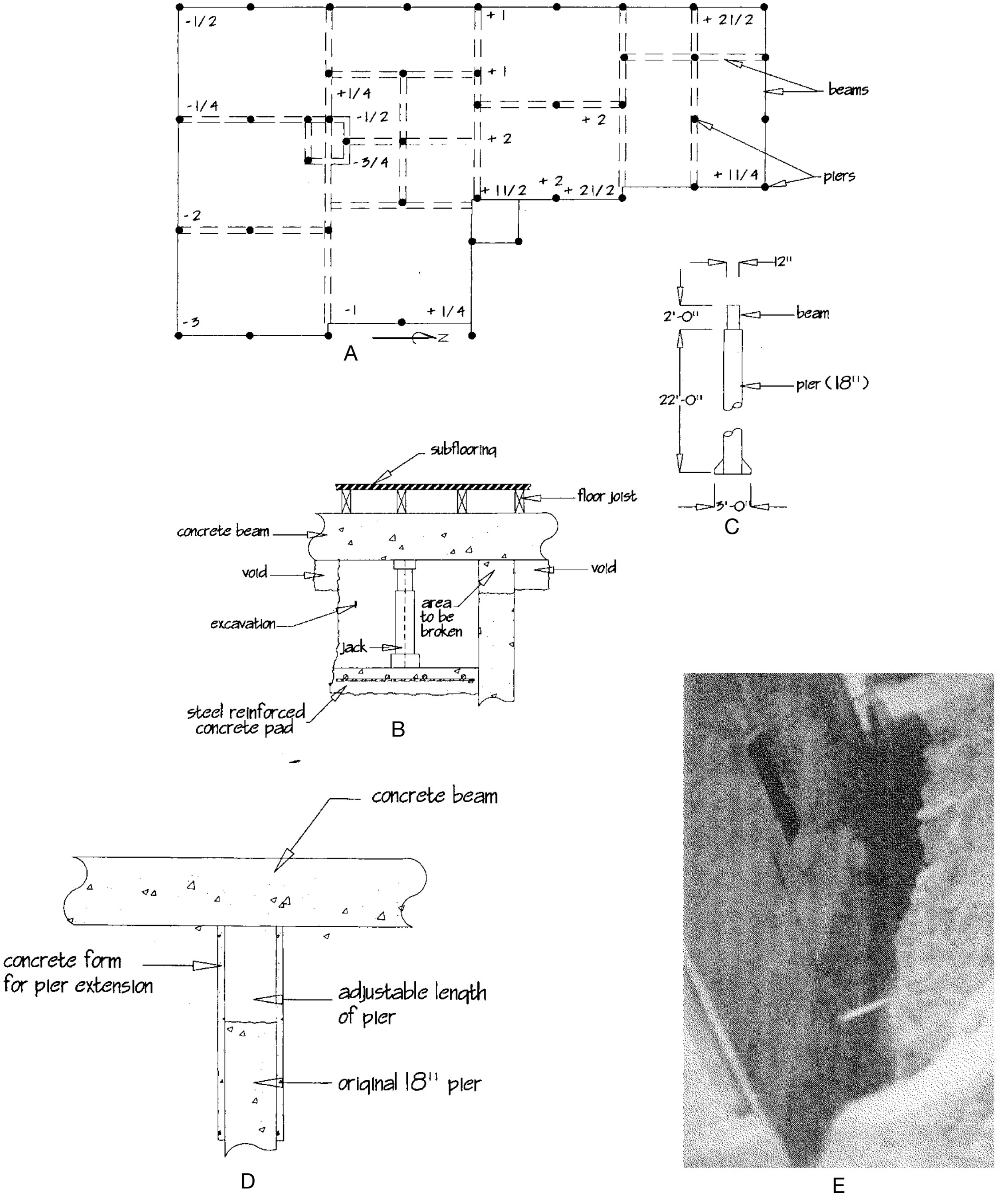
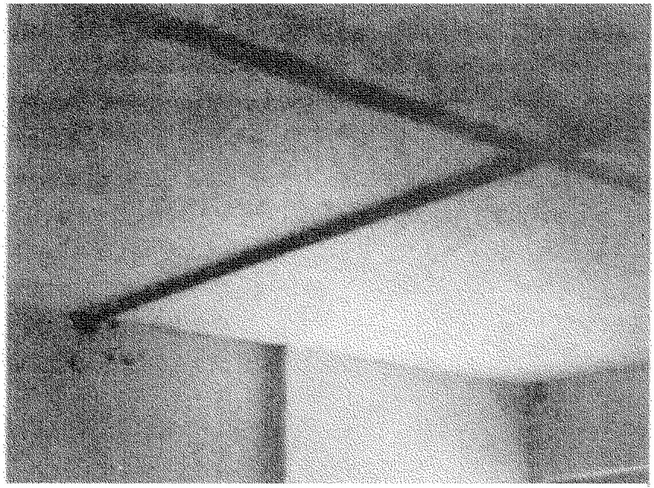
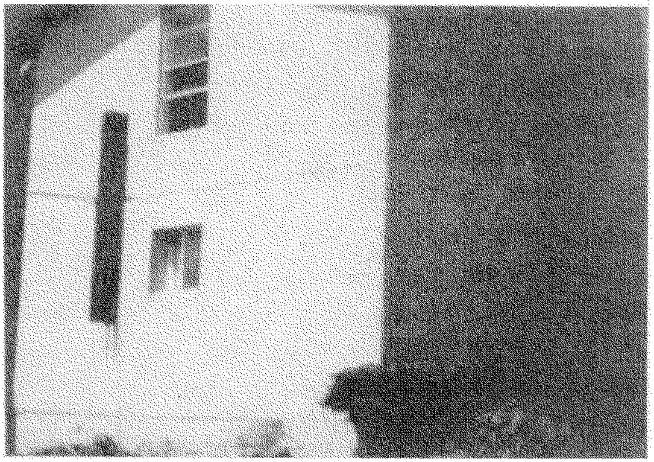


FIGURE 7B.7.3. Raising and lowering a foundation. (A) Foundation plan and elevation; (B) schematic drawing of mechanical setup for raising foundation; (C) pier detail (existing). (D) Preparing to pour new pier cap extension. (E) North wall of garage undermined. Note existence of original pier approximately 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)

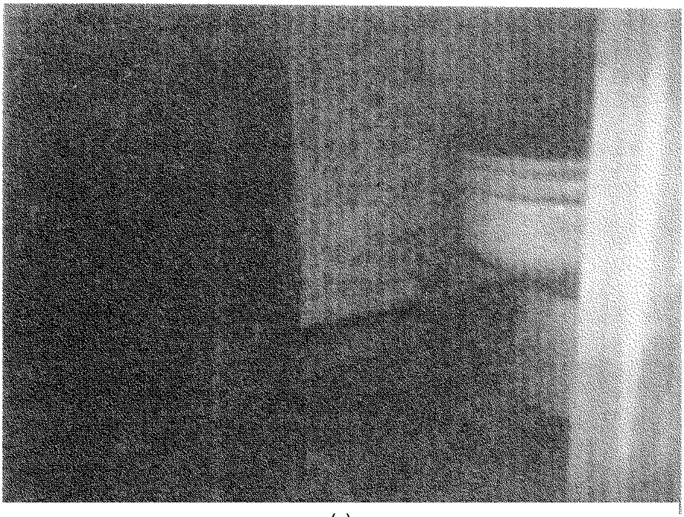
FIGURE 7B.7.4. (*continued*). Apartment building repair. (c) Steel beams are replaced with smaller plates once the wall has been plumbed. The excess bar is cut off flush with the nut and the assembly is stuccoed over and painted

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.

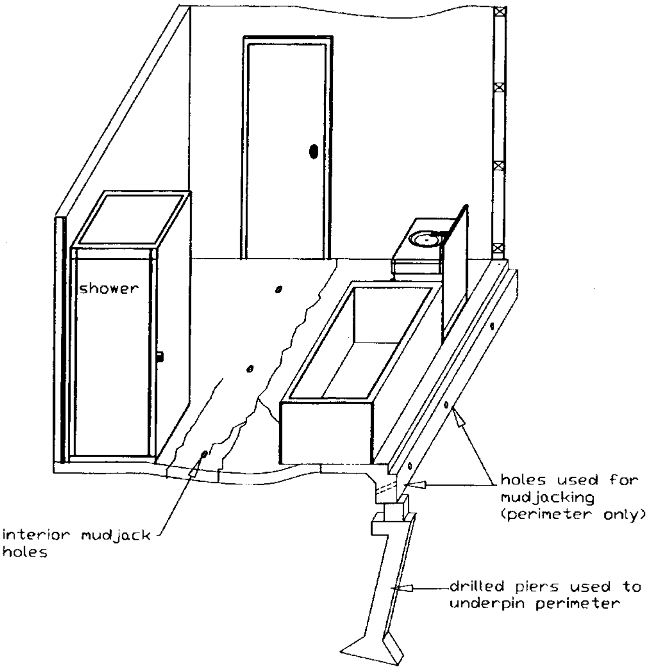
7B.7.8 Conclusions

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-

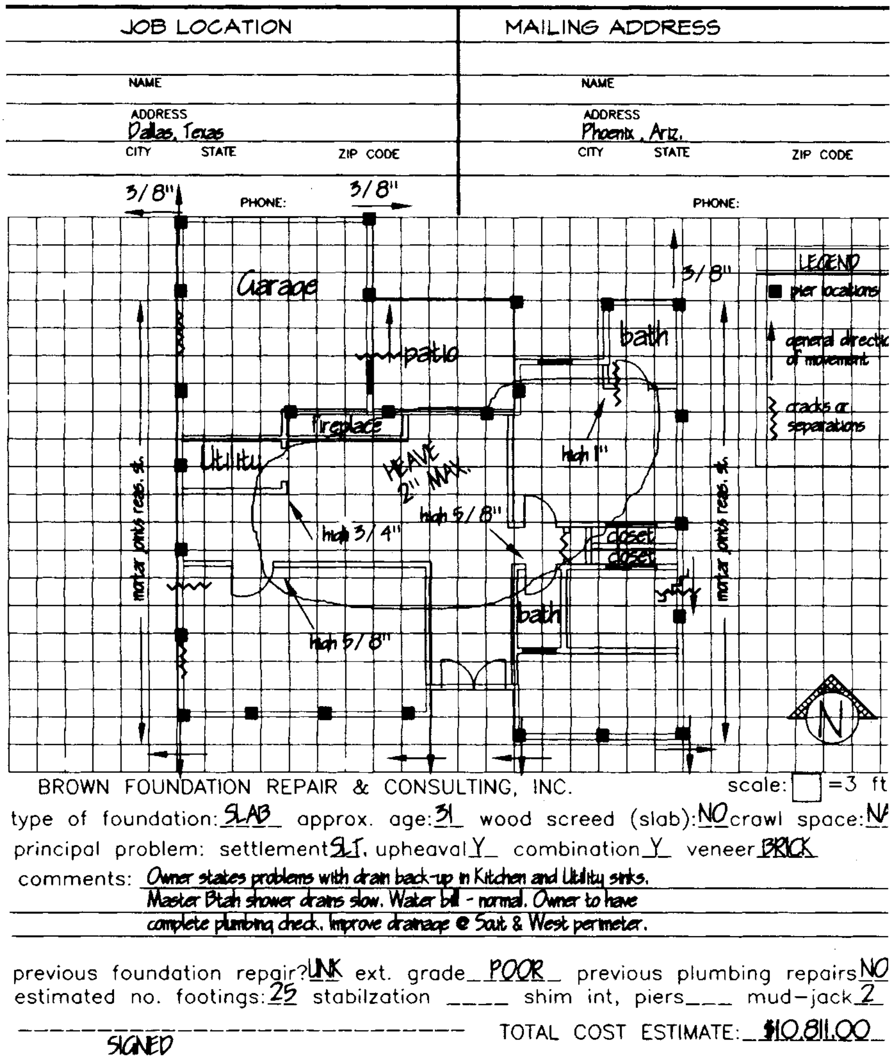


FIGURE 7B.8.1. Inspection report example.

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

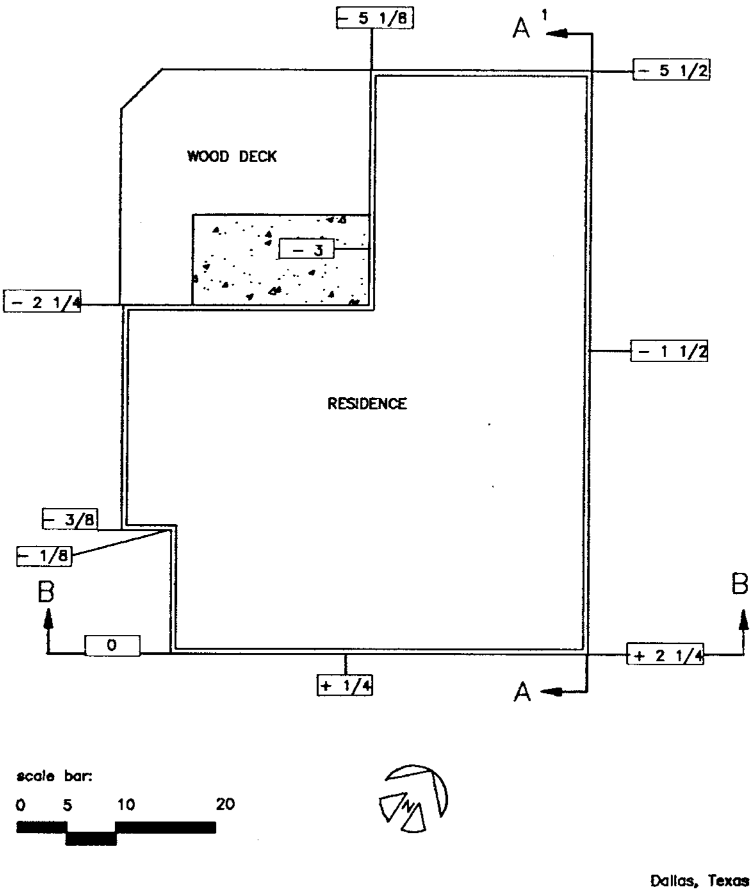


FIGURE 7B.8.2. Relative elevation survey.

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

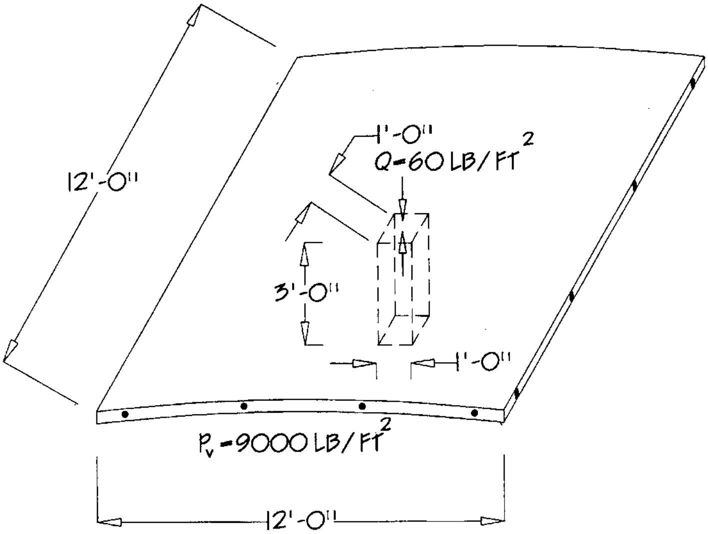
Some discussion has already been devoted to the issue of soil swell. Tucker and Davis presented data for a particular soil wherein a 4% increase in soil moisture was sufficient to cause a 1¼ in (3.2 cm) vertical rise in a type B, FHA slab foundation with a resulting potential swell force of 9000 lb/ft^2 (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]

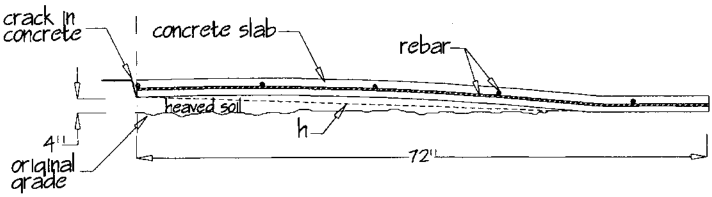
To pursue this train of thought, actually how much water is required to produce the 4% increase? Assume $W_S + W_w = 100$ lb/ft^3 (1602 kg/m^3); $W_S = 86$ lb/ft^3 (1376 kg/m^3); initial moisture, W% = 16, $W_w = 14$ lb/ft^3 (224 kg/m^3); final moisture, W% = 20, $W_w = 17$ lb/ft^3 (272 kg/m^3); where 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

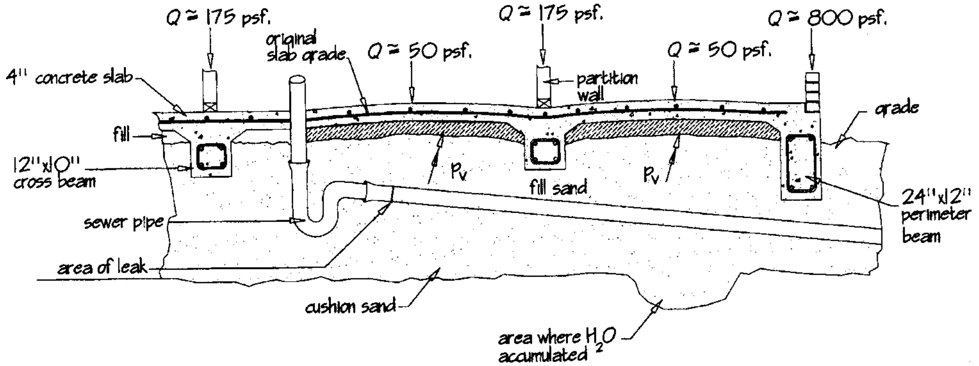


FIGURE 7B.8.4. Example of slab displacement due to upheaval resulting from soil swell. Note: the steel reinforcing and the cross-sectional cofiguration of the beams produce a resistance to stress far in excess of the structural loads indicated.

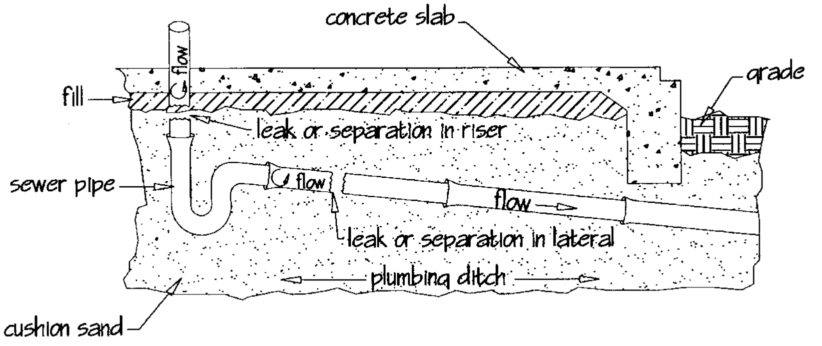


FIGURE 7B.8.5. Residential sewer.

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

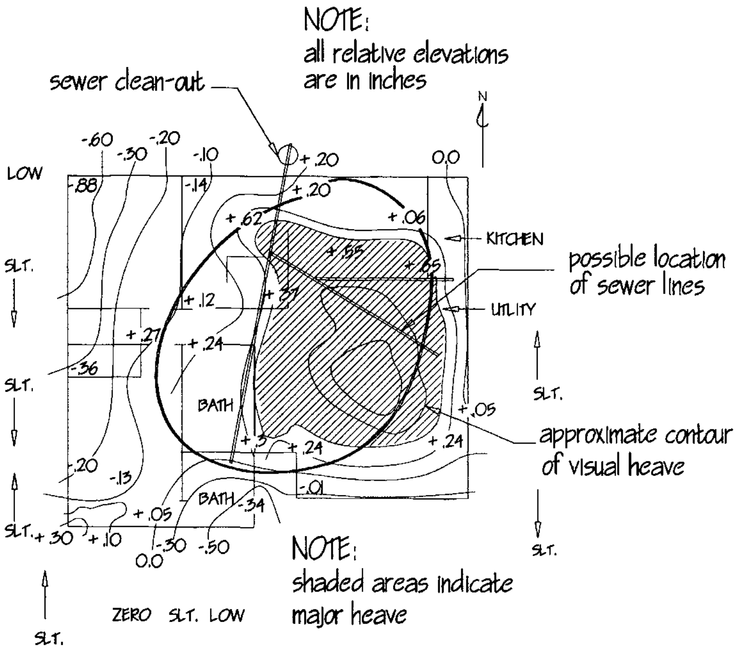


FIGURE 7B.8.6. Differential elevations in a slab foundation.

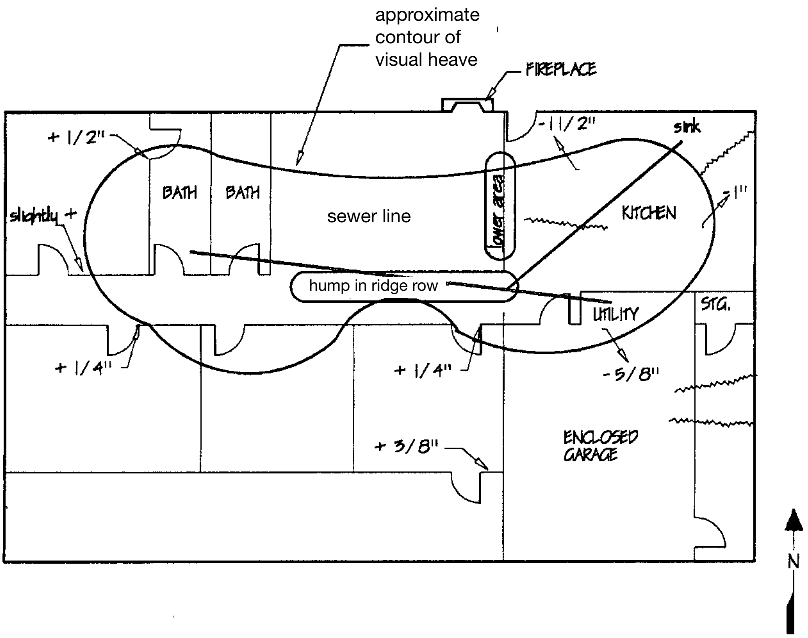
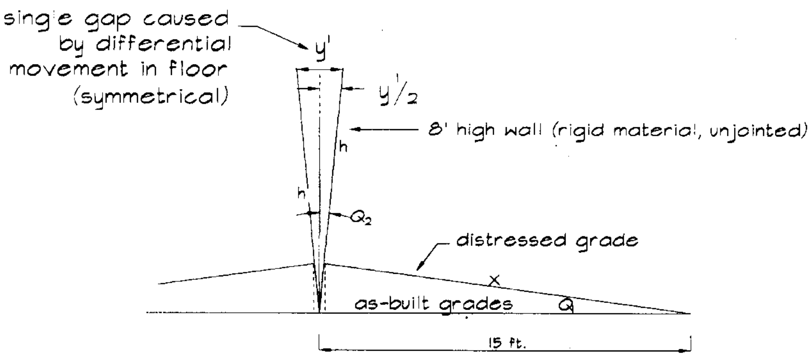


FIGURE 7B.8.7. Heave contour determined by field drawings.

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.

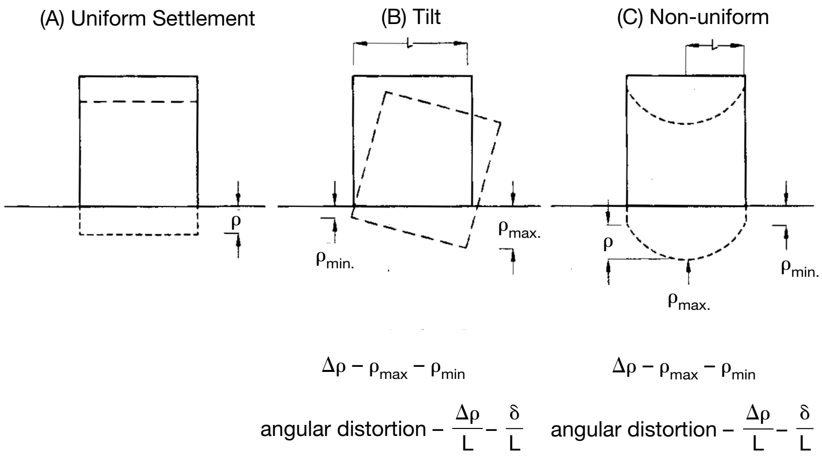


FIGURE 7B.8.9. Type of settlement.

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.

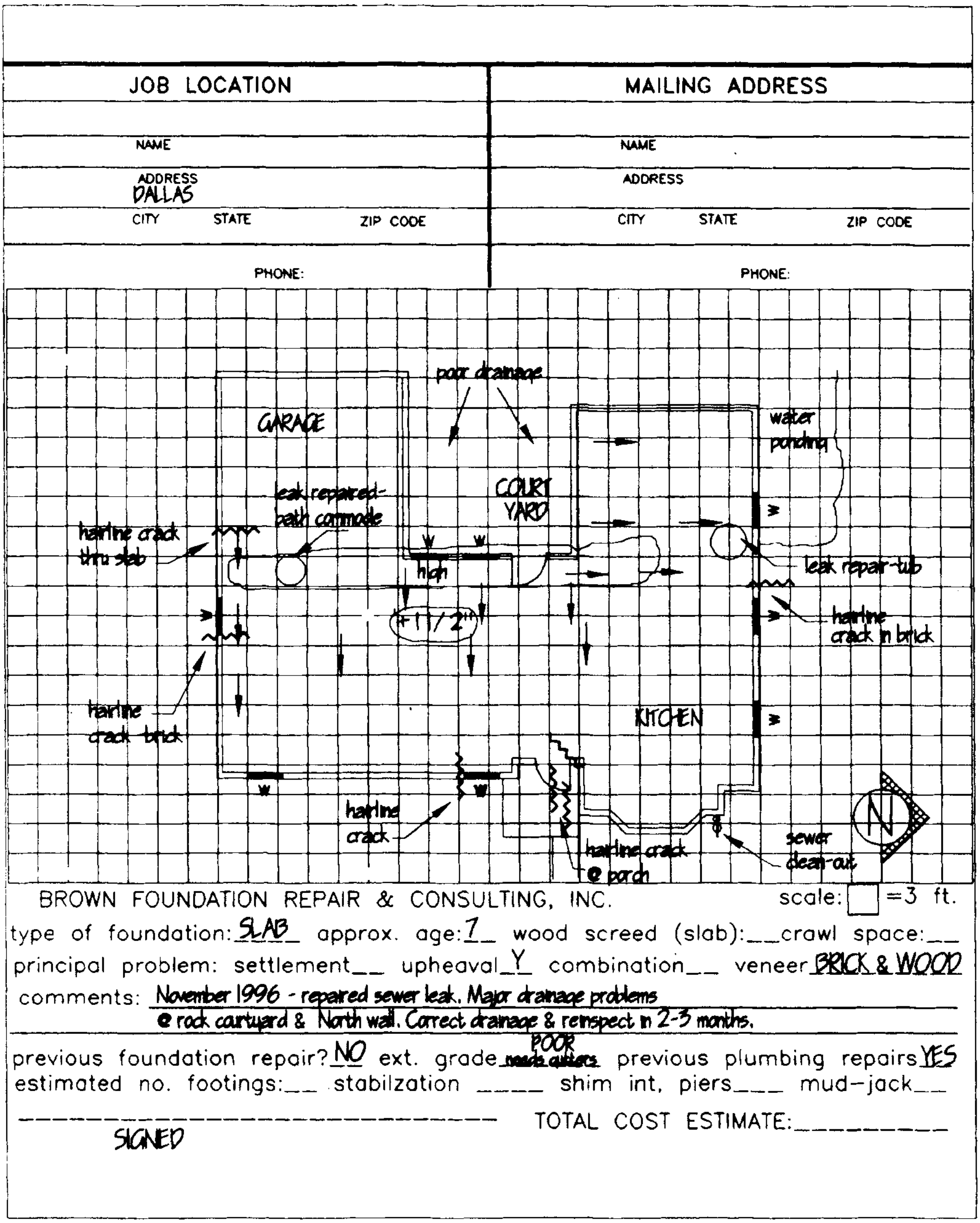
Evaluation:

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

NAME

ADDRESS
DALLAS
CITY STATE ZIP CODE

PHONE:

MAILING ADDRESS

NAME

ADDRESS

CITY STATE ZIP CODE

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__

SIGNED TOTAL COST ESTIMATE:________

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).

SECTION 7C
PREVENTIVE MAINTENANCE

ROBERT WADE BROWN

7C.1 INTRODUCTION

Preventing a problem always make more sense than curing one. Section 7A discussed the predominant causes of foundation failures. Sections 7B.1, 7B.2, and 7B.4 covered methods for correcting common foundation failures. Obviously, if the cause of the problem does not exist, neither will the problem. This chapter will focus on measures that encourage foundation stability, with a specific focus on those problems relative to expansive soils. Certain maintenance procedures can help prevent or arrest foundation problems if initiated at the proper time and carried out diligently. The following are specific suggestions on how this can be accomplished.

7C.2 WATERING

In dry periods, summer or winter, water the soil adjacent to the foundation to help maintain constant moisture. *Proper* watering is the key and will be discussed in the following paragraph. Also, be sure drainage is away from the foundation prior to watering. Remember, too much water can cause far more problems than too little.

When a separation appears between the soil and foundation perimeter, the soil moisture is low and watering is in order. Water should not be allowed to stand in pools against the foundation. Never attempt to water the foundation with a root feeder or by placing a running garden hose adjacent to the beam. Both represent uncontrolled watering. Sprinkler systems often create a sense of "false security" because the shrub heads, normally in close proximity to the perimeter beam, are generally set to spray away from the structure. The design can be altered to put water at the perimeter and thereby serve the purpose. This is done by replacing the sector heads with strip heads. However, in the end, the use of a soaker hose is still often the best solution. In either event, watering should be uniform and cover long areas at each setting, ideally 50 to 100 linear ft (15 to 30 m). From previous studies of water infiltration and runoff, it is evident that watering should be close to the foundation, within 6 to 18 inches (15 to 45 cm), and timed to prevent excessive watering.

Proper grading around the foundation will also serve to prevent unwanted accumulation of standing water. Sophisticated watering systems that utilize a subsurface weep hose with electrically activated control valves and automatic moisture monitoring and control devices are reportedly available. The multiple control devices are allegedly designed to afford adequate soil moisture control automatically and evenly around the foundation perimeter. Reportedly, the control can be set to limit moisture variations to plus or minus 1%. Within this tolerance, little if any differential foundation movement would be expected in even the most volatile or expansive clay soils. The key to this system is a true and proven ability to control water output and placement.[15–17*]

Avoid watering systems that make outlandish claims. They can often cause more problems than they cure. One example is the so-called water or hydro pier. One claim is that a weep hose with sections placed vertically into the soil on convenient centers (often 6 to 8 feet or 1.8 to 2.4 m) will develop a "pier," which will then support and stabilize the foundation. The "pier" is allegedly the product of expanded clay soil (see Figure 7B.4.16). Some contractors even claim the system will raise and "level" foundations. Outside the other obvious deficiencies, this procedure seems to lack controls capable of either equating water added to in situ moisture or monitoring total soil moisture. Lack of performance here introduces several problems when dealing with expansive soils.[16] First, moisture distribution within the soil is seldom, if ever, uniform. Second, excessive soil moisture approaching the liquid limit (LL) can cause a soil to actually *lose* strength (cohesion). Third, although *consistent* moisture content in the expansive soil will normally prevent differential deflection of the foundation, the method under discussion will not likely meet the requirements. And, fourth, water replenishment into an expansive soil will seldom, if ever, singularly accomplish any acceptable degree of "leveling." (Minor settlement, limited in scope, could be the exception.[16,26])

Where large plants or trees are located near the foundation, it could be advisable to conservatively water these, at least in areas with climatic (C_w) factors below about 25.[36] (These areas are generally classed as "semiarid."[36] Lower C_w values lean toward being more arid). As far as foundation stability is concerned, the trees or plants most likely to require additional water would be those that: (a) are immature, (b) develop root systems that tend to remove water from shallow soils, and (c) are situated within a few feet of the foundation. Refer also to Section 7B.2.

7C.3 DRAINAGE

It is important that the ground surface water drain away from the foundation. Proper moisture availability is the key. Excessive water is frequently detrimental. Proper drainage will help avoid excess water. Where grade improvement is required, the fill should be low clay or sandy loam soil. The slope of the fill need not be exaggerated but merely sufficient to cause water to flow outward from the structure. A 1% slope is equivalent to a drop of 1″ (2.5 cm) over approximately 8 ft (2.4 m). A satisfactory slope is often assumed to be 1–3%. Too great a slope encourages erosion. The surface of fill must be below the air vent for pier-and-beam foundations and below the brick ledge (weep holes) for slabs. Surface water, whether from rainfall or watering, should never be allowed to collect and stand in areas adjacent to the foundation wall.

Along with proper drainage, guttering and proper discharge of downspouts is quite important. Flowerbed curbing and planter boxes should drain freely and preclude trapped water at the perimeter. In essence, any procedure that controls and removes excess surface water is beneficial to foundation stability. Water accumulation in the crawl space of pier-and-beam foundation is also to be avoided. Low-profile pier-and-beam foundations can be particularly susceptible to this problem. Drainage control and adequate ventilation serve as the best preventive measures. As a "rule of thumb," vents should be provided on the ratio of one square foot per 150 square feet of floor space. Where construction design prevents an adequate number of vents, the desired ventilation can be implemented by the use of forced air blowers.

Domestic plumbing leaks (supply and sewer) can be another source for unwanted water.[24,26,51,79,83,86] Extra care should be taken to prevent and/or correct this problem. Water accumula-

*References are in Section 7E.

tion beneath a slab foundation accounts for a reported 70% of all slab repairs.[15–17] Sewer leaks are responsible for a very high percentage of these failures (see Section 7B.1).

7C.3.1 French Drains/Subsurface Water

French drains are required, upon occasion, when subsurface water migrates beneath the foundation. Figure 7C.1 is a drawing of a typical French drain. When the foundation is supported by a volatile (high-clay) soil, the intrusion of unwanted water must be stopped. The installation of a French drain to intercept and divert the water is a useful approach.[15–17,26,79] The drain consists of a suitable ditch cut to some depth below the level of the intruding water. The lowermost part of the ditch is filled with gravel surrounding a perforated pipe. The top of the gravel is continued to at least above the water access level and often to or near the surface.

Provisions are incorporated to remove the water from the drain either by a gravity pipe drain or a suitable pump system. Simply stated, the French drain creates a more permeable route for flow and carries the water to a safe disposal point. If the slope of the terrain is not sufficient to afford gravity drainage, the use of a catch basin/sump pump system is required. The subsurface water commonly handled by the French drain is perched ground water or lateral flow from "wet weather" springs or shallow aquifers.

Where the conditions warrant, the design of the drain can be modified to also drain excessive surface water. This is readily accomplished by adding surface drains (risers) connected to the French drain system. An alternative approach is to carry the gravel to the surface. (A proper drain intercepts and disposes of the water before it invades the foundation.) Water from downspouts should not be tied directly into the French drain; a separate pipe drain system is preferred. The second solid pipe could, however, be placed in the French drain trench.

A French drain is of little or no use in relieving water problems resulting from a spring within the confines of the foundation, as it is almost impossible to locate and tap a spring beneath a foundation. This condition is also rare, since a wet site causes real problems to the builder.

When distress problems exist prior to the installation of a French drain, foundation repairs are often required. In that event, these repairs should be delayed to give ample time for the French drain

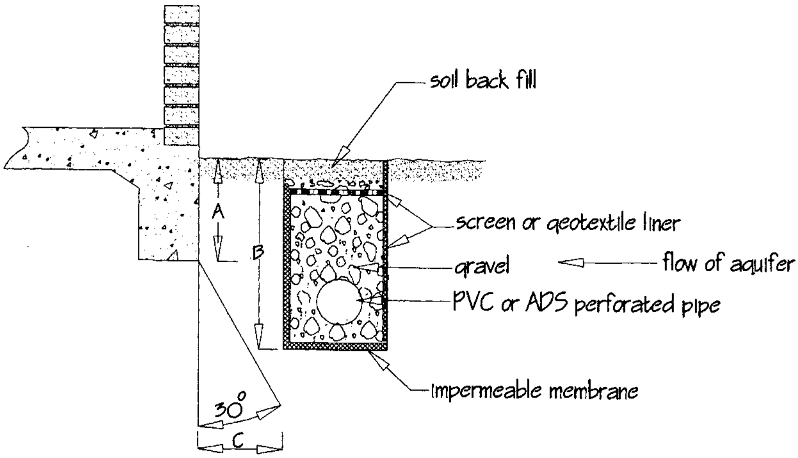


FIGURE 7C.1 Typical French drain. Generally, C is equal to or greater than A, and B is greater than A + 2 ft (0.6 cm). The drain should be located outside the load surcharge area.

to develop a condition of moisture equilibrium under the foundation. Otherwise, recurrent distress (repairs) can be anticipated due to the disturbance of soil moisture introduced by the drain.

7C.3.2 Water or Capillary Barriers

In an effort to maintain constant soil moisture, measures that impede the unwanted transfer of soil moisture can be considered. One such attempt has been the use of moisture barriers. The barriers may be either horizontal or vertical, permeable or impermeable. Refer to the following sections.

7C.3.2.1 Horizontal Barriers

Horizontal, *impermeable* barriers can be as simple as asphalt or concrete paving, or polyethylene film (see Figure 7C.2). These materials are used to cover the soil surface adjacent to the foundation and inhibit evaporation. Coincidentally, the covers could also restrict soil moisture loss to transpiration, since vegetation would neither grow nor be cultivated in the sheltered area.

Permeable horizontal barriers usually consist of little more than landscaping gravel or granular fill placed on the soil surface previously graded for drainage. Moisture within the porous material cannot develop a surface tension and no adhesive forces will exist, both of which are required to create a capillary (or pore) pressure. Gravity (drainage) then becomes the factor dictating free water movement.

7C.3.2.2 Vertical Barriers

The vertical *impermeable* capillary (or water) barrier (VICB) is intended to block the transfer of water laterally within the affected soil matrix. Figure 7C.3 shows typical vertical barriers. Placed adjacent to a foundation, the VICB will hopefully maintain the soil moisture at a constant level within the foundation soil encapsulated by the barrier.[1–4] As an added benefit, this approach will also prevent transpiration, since tree, plant, or shrub roots would be prevented from crossing the barrier. In some cases, the soil moisture within the intended confines of the VICB (prewetting) is increased to a percent or so above the soils' plastic limit prior to construction of the foundation. (This practice is generally exclusive to slab foundations.) This assumes that the soil is preswelled to a point that increased water is not likely to cause intolerable swell and, at the same time, the barrier will, hopefully, prevent a decrease in soil moisture beneath the foundation. Hence, a stable condition may be created.

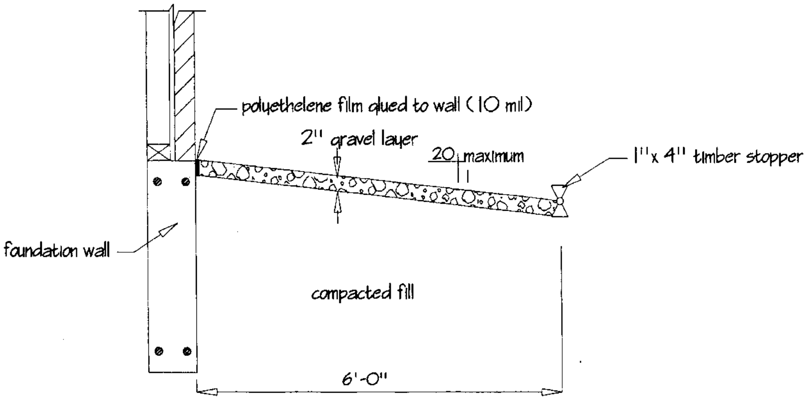


FIGURE 7C.2 Horizontal moisture barrier consisting of a polyethylene membrane overlain by a thin gravel layer (from Chen, ref. 26).

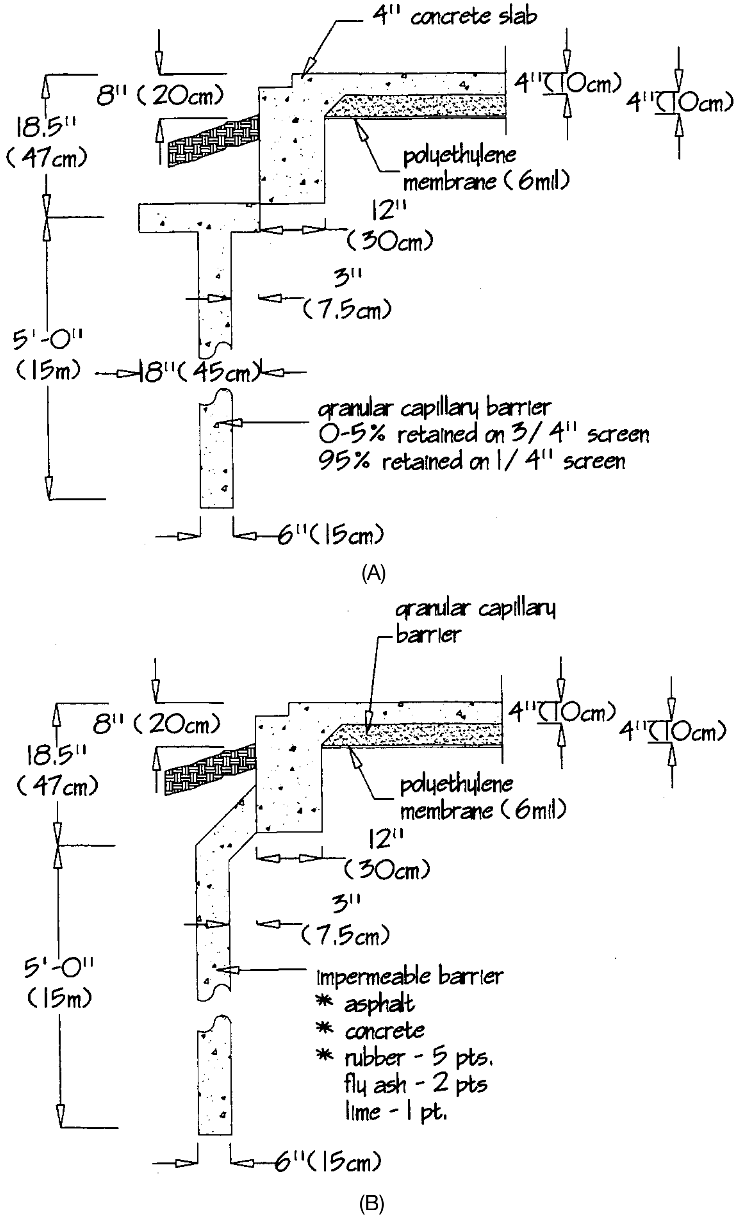


FIGURE 7C.3 Typical permeable (A) and impermeable (B) vertical moisture barriers.

Vertical *permeable* capillary barriers (VPCB) generally consist of a slit trench filled with a permeable material. The VPCB will accept water and distribute it into the permeable barrier. This also tends to block lateral capillary movement of water into the soil matrix itself. (Clay is far less permeable than the material within the barrier). The use of vertical capillary barriers has not met with appreciable success.[15–17,26,58,79] For example, Chen states "it is doubtful that the installation [of vertical capillary barriers] is a sufficient benefit to warrant the cost."[26]

7C.3.3 Conclusion

The use of water or capillary barriers offers possible benefits, but field data made public to date leaves much to be desired.[15–17,26,58,79] The installation of moisture controls (such as the French drain, vertical or horizontal moisture barriers), by intent alters the moisture profile within the foundation-bearing soils. The time period over which the results of this change becomes noticeable might vary from several days to several years. The amount and rate of moisture variation, the particular soil properties, and foundation design each influence the extent of soil volume change and ultimately any foundation movement.

The installation of French drains is frequently followed by "drying" of the foundation-bearing soils. This could ultimately result in a soil moisture regain if extraneous water becomes available. Unless this gain is uniform over the entire foundation area (which is usually true over the long term), some soil swell and resultant foundation upheaval could eventually occur.

As a matter of interest, the *horizontal permeability* (which translates to lateral water migration) in a highly expansive clay varies from something like 1 ft/yr (10^6 cm/sec) to 20 ft/yr (2×10^5 m/sec). (The *vertical* permeabilities are roughly 0.1 ft/yr or 10^7 cm/sec). For sands, the differences between horizontal and vertical permeabilities are much less, with the general readings being in the range of 1000 to 10 ft/yr (10^3 to 10^5 m/sec).[65]

7C.4 VEGETATION

Certain trees, such as the weeping willow, oak, cottonwood, and mesquite, have extensive shallow root systems that remove water from the soil. These plants can cause foundation (and sewer) problems even if located some distance from the structure. Many other plants and trees can cause different foundation problems if planted too close to the foundation. Plants with large, shallow root systems can grow under a shallow foundation and, as roots grow in diameter, produce an *upheaval* in the foundation beam. Construction most susceptible to this include flat work such as sidewalks, driveways, patios, as well as some pier-and-beam foundations. Pruning the trees and plants will limit the root development. Watering, as discussed earlier, will also help.

Plants and trees can also remove water from the foundation soil (transpiration) and cause a drying effect, which in turn can produce foundation *settlement.* [The FHA (now HUD) suggests that trees be planted no closer than their ultimate height. (There is no basis in fact that relates the lateral spread of roots to tree height.) In older properties, this is often not feasible, since the trees already exist. With proper care, the adverse effects to the foundation can however be minimized or circumvented.]

The principal moisture loss that would likely affect foundation stability occurs generally between the field capacity and the level of plant wilt.[16,17,69] Refer to Figure 7A.2. Dr. Don Smith, Professor of Botany at the University of North Texas, Denton, Texas, expresses the opinion that tree roots or other plant roots are not likely to grow beneath most foundations. This is due to several factors, the most important of which are:

1. Feeder roots tend to grow laterally within the top 24 inches (0.6 m).[49,50,98] The perimeter beam often extends to near that depth and would block root intrusion.

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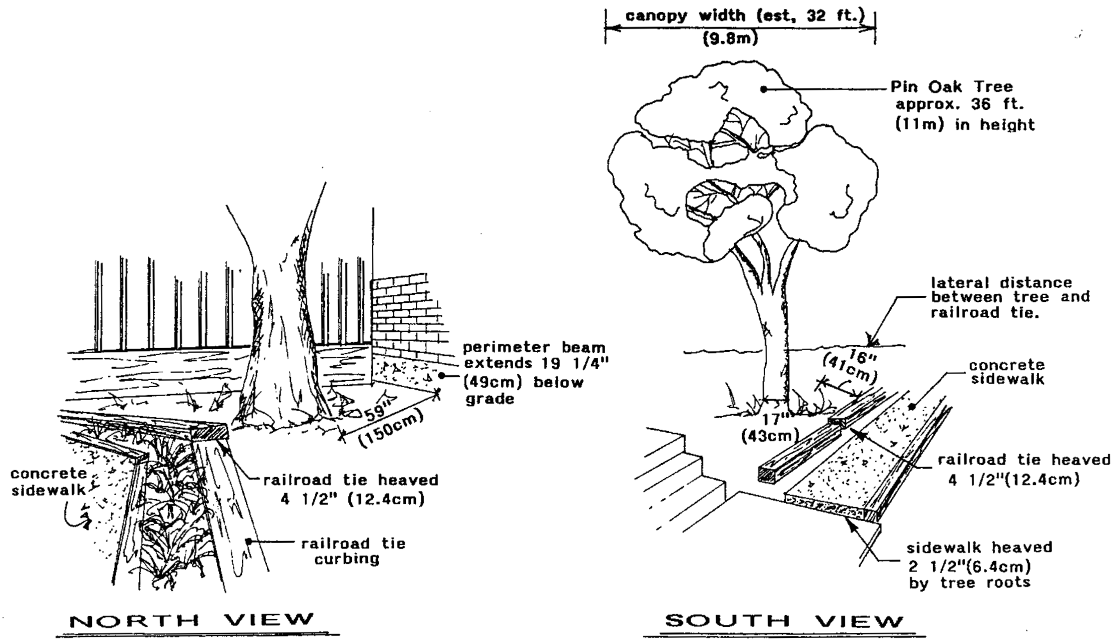
2. Roots prefer loosely compacted soil (low overburden).
3. Soil moisture (long range) and oxygen availability, both necessary for plant growth, are less abundant beneath the foundation.
4. These confined and sheltered areas have no normal access to a replenishing source for water. (Roots tend to "grow to water.")

For the foregoing reasons, it would appear that trees pose no real threat to foundation stability other than that noted in the first paragraph. Along these same lines, even in a semiarid area with highly expansive soil, it is seldom that a significant earth crack is noted beneath the tree canopy. This is particularly true with trees exhibiting low canopies. This suggests an actual conservation of soil moisture.107 Also, if trees pose the problems which some seem to believe, why don't *all* foundations with like trees in close proximity show the same relative distress. In literally thousands of instances where foundation repair is made without removal of trees, why doesn't the problem at least sometimes reoccur? Figure 7C.4 depicts an actual condition where trees are growing in close proximity to a slab foundation without mishap. This figure shows a "real-world" representation of the influence of trees on foundations. The tree shown is a pin oak that was planted at the time of construction. The tree is 17 in (43 cm) in diameter, approximately 36 ft (11 m) in height, with a canopy width of about 32 ft (9.8 m). The tree is located 59 in (150 cm) from the perimeter beam, which extends 15¼ in (49 cm) below grade. For the record, there are four pin oaks similarly planted along the west perimeter. The depth of the beam probably accounts for the lack of impact upon the foundation.

Removal of existing trees can create more problems than might originally exist.[15–17,26,42] Cutting (trimming) the roots can induce similar problems, though to a lesser extent.[53] Any extended differ-



(a)

FIGURE 7C.4 Tree in close proximity to foundation with no effect on foundation. (a) Site drawing.

(b)

(c)

FIGURE 7C.4 (*continued*). (b) Photograph looking north; note heave of railroad tie, left center. Tree in close proximity to foundation with no effect on foundation. (c) Photograph looking south; note heave of sidewalk and north end of rail road tie (in the grass).

ential in soil moisture can produce a corresponding movement in the foundation. If the differential movement is extensive, foundation failure will likely result.

Even with proper care, foundation problems can develop. However, consideration and implementation of the foregoing procedures will afford a large measure of protection. It is possible that adherence to proper maintenance could eliminate perhaps 40% of all serious foundation problems. Anyone who can grow a flower bed can handle the maintenance requirements!

Remember one of the basic laws of physics, nothing moves unless forced to do so. A foundation is no different. This has been emphasized in prior chapters. All foundation repair accomplishes is to restore the structural appearance. If the initial cause of the problem is not identified and eliminated, the problem is likely to reoccur.

SECTION 7D

FOUNDATION INSPECTION AND PROPERTY EVALUATION FOR THE RESIDENTIAL BUYER

ROBERT WADE BROWN

7D.1 INTRODUCTION

If you are house shopping in a geographical area with a known propensity for differential foundation movement, it is always wise to engage the assistance of a qualified foundation inspection service. This could involve an experienced foundation repair contractor or an equally qualified professional engineer (P.E.). The word *qualified* cannot be overemphasized. It is necessary to evaluate the existence of foundation-related problems, determine the cause of the problems, and, when such are found to exist, offer a repair procedure. The cause is particularly important. *Repairs will be futile, if the original cause of the distress is not recognized and eliminated.* The principal problem herein lies with the separation of settlement from upheaval (see Sections 7B.1, 7B.2, and 7B.4) . The checklist in Table 7D.1 presents several of the more obvious manifestations of distress relative to differential foundation movement. With only limited experience, one can learn to detect these signs. The problems of detection become more difficult when cosmetic attempts have been used to conceal the evidence. These activities commonly involve painting, patching, tuck-pointing, addition of trim, installation of wall cover, etc. The important issue in all cases is to decide whether the degree of distress is sufficient to demand foundation repair (see also Section 7B.8). This decision requires extensive experience and some degree of compromise. Several factors influence the judgement.

It is difficult, if not impossible, to properly evaluate the material set out in Table 7D-1 without extensive first-hand experience with actual foundation repairs. To quote Terzaghi, the founder of soil mechanics: "In our field [engineering and geology] theoretical reasoning alone does not suffice to solve the problems which we are called upon to tackle. As a matter of fact, it [engineering and geology] can even be misleading unless every drop of it is diluted by a pint of intelligently digested experience."[48] One good example of the importance of on-the-job experience is the proper determination of upheaval as opposed to settlement. If upheaval were evaluated as settlement, the existence of water beneath the foundation would likely be overlooked. In that case, future, more serious, conse-

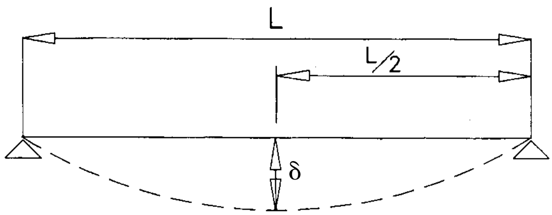
TABLE 7D.1 Inspection Checklist

1. *The extent of vertical and lateral deflection.* Does any structural threat exist or appear imminent? [Most authorities seem to accept a maximum differential movement of 0.3% (over 10 ft = 0.36 in; over 20 ft = 0.72 in) as tolerable for normal residential (frame) construction. The table below gives typical values for various angular distortions (δ) utilizing the "three point method."[7,73] In the real world of remedial concern, two point (δ′) is normally the "test" procedure utilized. This is due, in large part, to partition walls and other obstructions that prevent or hinder the three point method. The cited 0.3% equates to 1/333 or 1″/27.75 ft. The wall height has a direct bearing on crack width resulting from δ.[7]

   At the identical δ (or δ′) the crack width in a 16 ft wall will be twice that observed in an 8 ft wall, all other factors being equal.]
2. *Is the stress ongoing or arrested?*
3. *The age of the property.*
4. *The likelihood that the initiation of adequate maintenance would arrest continued movement.* [For example, 1) in cases of upheaval, elimination of the source for water will often arrest movement, and 2) where minor settlement is involved, proper watering may reverse or eliminate the problem.]
5. *Value of the property as compared to repair costs.* [Most foundation repair procedures require some degree of compromise. In order to arrive at a reasonable or practical cost, the usual primary concern is to render the foundation "stable" and the appearance "tolerable." In all cases, the primary goal should be a "cost-effective" solution. In virtually all repair, the cost to truly "level" a foundation, if it were possible, would be prohibitive. This is further complicated by the fact that "foundations are not generally built level."[15–17]
6. *Type and condition of the existing foundation.* [If the foundation is pier-and-beam, is adequate crawl space existent? If it is a slab, was it poured with proper thickness, beams and reinforcing steel?]
7. The possibility that, if the movement appears arrested, cosmetic approaches would produce an acceptable appearance.

A simple way to monitor future movement is to place a pencil mark on various doors, sheetrock cracks, and mortar separations, as illustrated by Figure 7D.1. Use a straight edge and sharp pencil. On cracks, place the marks such that horizontal as well as vertical displacements are monitored. Initial, date, and, where applicable, record the width of the crack. Repeat this process at random locations throughout the property. Future inspections will provide the answers to the on-going movement question. If any movement occurs, the lines will no longer line up. The measured crack widths previously recorded will also show movement. This method not only shows movement but provides a guide as to in which direction the movement occurs.

Angular Distortion, δ/L



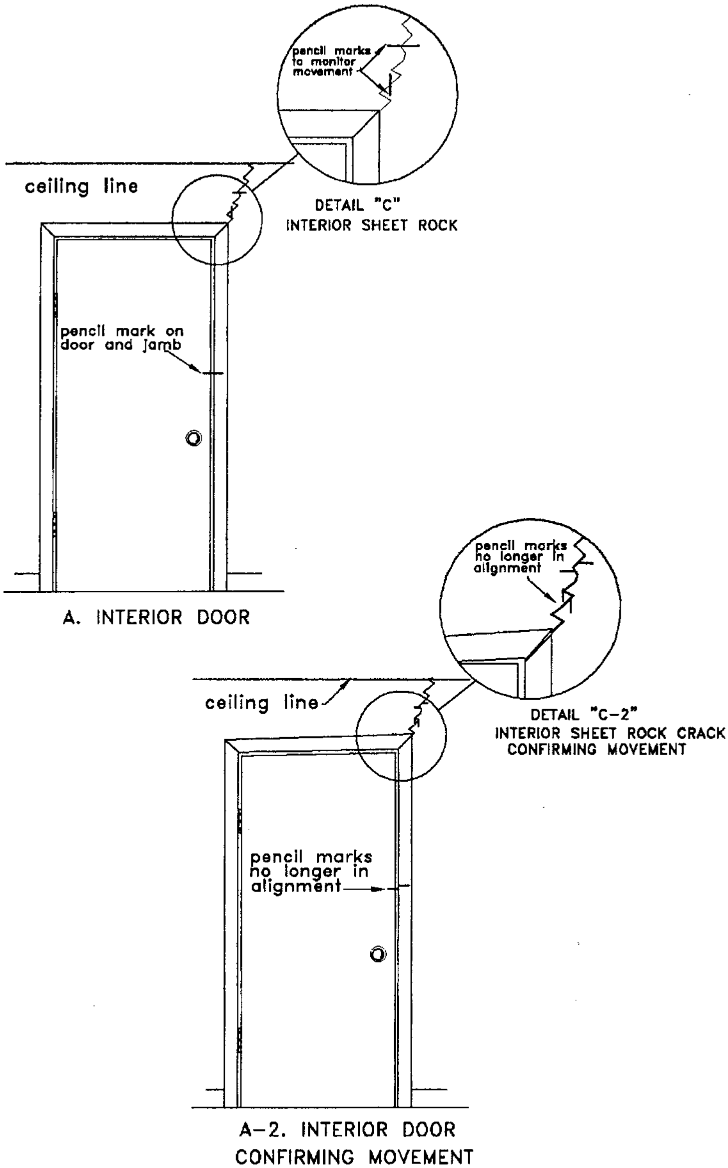
Two Point: = $\delta' = \delta/L/2 = 2\delta/L$

L	δ/L*	δ'/10 ft, in†	δ'/20 ft, in
12.5 ft (150 in)	1.5%	1.8 in	3.6 in
20.8 ft (250 in)	0.4%	0.48 in	0.96 in
25 ft (300 in)	0.33%	0.4 in	0.8 in
	0.31%	0.375 (3/8″)	0.6875 (11/16″)
33 ft (400 in)	0.25%	0.3 in	0.6 in
42 ft (500 in)	0.2%	0.24 in	0.48 in
50 ft (600 in)	0.167%	0.2 in	0.4 in

*δ is constant at 1 in and L varies as shown.
†Distance indicated (120 in or 240 in) times $2\delta/L$.
Source: After U.S. Dept. of the Navy, 1988, and Whitman and Lambe, 1979.
Example conversion: Assume: 1 in deflection (δ) over 25 ft (150 in).

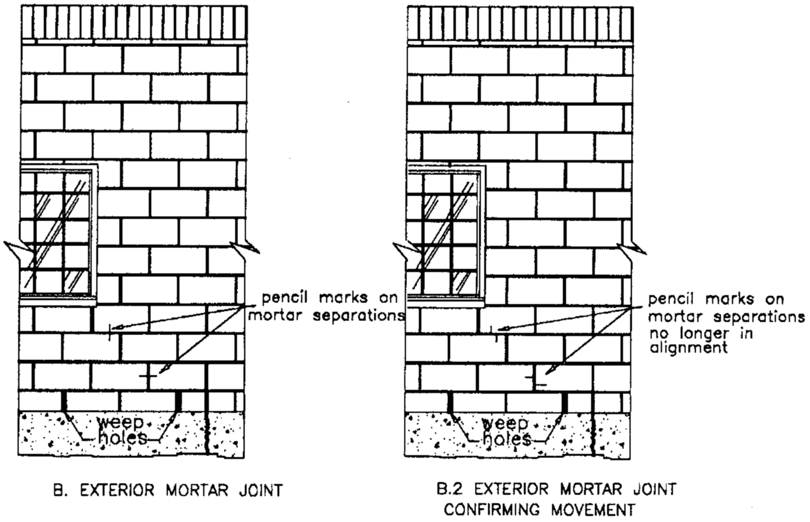
1″/25 = 0.33%, comparing to two point analysis:

$\delta'/10' = 0.33$ or $\delta' = (120\text{ in})(0.0033) = 0.4$ in.



(a)

FIGURE 7D.1 Monitoring ongoing movement.



(b)

FIGURE 7D.1 (*continued*). Monitoring ongoing movement.

quences would be nearly certain. Thankfully, there are usually no similar consequences in making the error of labeling settlement as upheaval. Essentially, this is true because first, all of foundation repair techniques are designed to raise a lowermost area to some higher evaluation, and second, there is no undisclosed cause for continual problems. In cases of settlement, one merely raises the distressed area to "as built." In instances of upheaval, it becomes necessary to raise the "as built" to approach the elevation of the distorted area. The latter is obviously far more difficult.

The existence of foundation problems need not be particularly distressing so long as the buyer is aware of potential problems before the purchase. As a rule, the costs for foundation repair are not relatively excessive, the results are most often satisfactory, and the net results are a stronger, more nearly stable foundation.3 It is the surprise that a buyer cannot afford.

7D.2 FOUNDATION INSPECTIONS/SELECTING AN INSPECTOR

The cost for a residential foundation inspection service is quite nominal, varying from about \$150.00 to \$500.00 (1998 dollars), depending on the time involved and the locale. A word of caution: be selective in choosing your inspector. Since the late 1980s many institutions such as lenders, insurers, and sophisticated buyers have begun the practice of requiring a structural inspection performed by a registered professional engineer (P.E.). The engineer should be registered as a structural or civil engineer. In some states (Texas, for example) an engineer registered in any discipline (mechanical, electrical, petroleum, aeronautical, industrial, etc.) can represent himself as qualified to make the inspection. More often than not, these people are not qualified by either experience or education. The engineer of choice should be independent, unbiased, and not associated (directly or indirectly) with any self-serving entity—foundation repair company, builder, insurer, etc. If possible, the engineer should have extensive hands-on experience in foundation repair. If not, he should request specific repair proposals from qualified repair companies. If the engineer accepts (or rejects) any bids proposed by qualified contractors, he should do so based on a comprehensive, conclusive mathematical analysis. This avoids both overkill and ensures a competent repair.

Experience has shown that often engineers not properly qualified or schooled in foundation problems tend to present defective or slanted reports. As an example, on separate occasions, the same "engineers" have specified *either:* 1) 12 in (30 cm) diameter piers, 6 ft (1.8 m) on centers and drilled to 10–12 ft (3 to 3.6 m) with caged 4 #5s and sometimes even belled to 24 in (60 cm), *or* 2) an 8″ diameter (20 cm) straight shaft with 3 #3s, 6 ft (1.8 m) on centers and drilled to 10–12 ft (3–3.6 m). The 12″ diameter option contains specifications that (compared to the 8″ diameter option) are either a gross overkill, or apparently intended to remove the 12″ (30 cm) diameter pier from price contention. (The ridiculous specifications increase the comparative cost for a 12″ diameter (0.3 m) pier by about $75.00.) A 12″ (0.3 m) pier to 10 ft depth, on 8 ft (2.4 m) centers reinforced with 3 #3s would represent a superior choice. By the way, in this specific example, none of the engineers were registered as either structural or civil. One was registered as a mining engineer and two others as mechanical engineers. This is offered as an observation, not a condemnation. This does not suggest that errors are not made by well-meaning civil or structural engineers. This happens frequently, usually as the result of lack of experience. Errors in judgment, intentional or otherwise, tend to produce a gross overcharge and/or ineffective repairs.

The engineer of choice should be unbiased and independent as well as competent. It is sometimes difficult for engineers (or anyone else for that matter) to be objective when a decision is contrary to the interest of the party paying their bill. This is particularly evident in the number of engineers who accept or recommend practices, procedures, or causes with obvious technical flaws.[15–17] Many of these people are either on the payroll of foundation repair companies who "push" a particular process, or retained by insurance companies with self-serving agendas.

Evaluate any disclaimers included in the engineer's report. If the engineer specifies the method of repair, he likely assumes legal responsibility for the outcome of the repair and future stability of the structure. This is probably true, despite any disclaimer included in his report. These disclaimers, often used as some attempt to shield the engineer from inexperience, probably would not survive an appropriate court of law. The repair contractor would likely be named as a third party defendant in any litigation resulting from repair; however, the ultimate responsibility would likely fall on the engineer who designed and specified the repairs. In this event, the consumer would be lucky if the defendant engineer carried insurance. Otherwise, the consumer is apt to be without recovery unless his state requires bond coverage for practicing engineers. This situation can be avoided (somewhat at least) by the engineer (or owner) soliciting proposals from qualified foundation repair companies who specify and, in turn, provide warranty for all work. The question now become "how can one determine the presence of a potential foundation problem."

7D.3 INDICATIONS OF STRUCTURAL DISTRESS

Following is a simple checklist for evaluating the stability of a foundation. If you have questions or uncertainty regarding any of the items, consult a qualified authority. One might also refer back to Section 7B.8.4 in order to evaluate the "severity" of any observation. Figure 7D.2 presents photographs showing several of the following concerns:

1. Check the exterior foundation and masonry surfaces for cracks, evidence of patching, irregularities in siding lines or brick mortar joints, separation of brick veneer from window and door frames, trim added along door jam or window frames, separation or gaps in cornice trim, spliced (extended) trim, separation of brick from frieze or fascia trim (look for original paint lines on brick), separation of chimney from outside wall, masonry fireplace distress on interior surfaces, etc.
2. Sight ridge rafter, roof line, and eaves for irregularities.

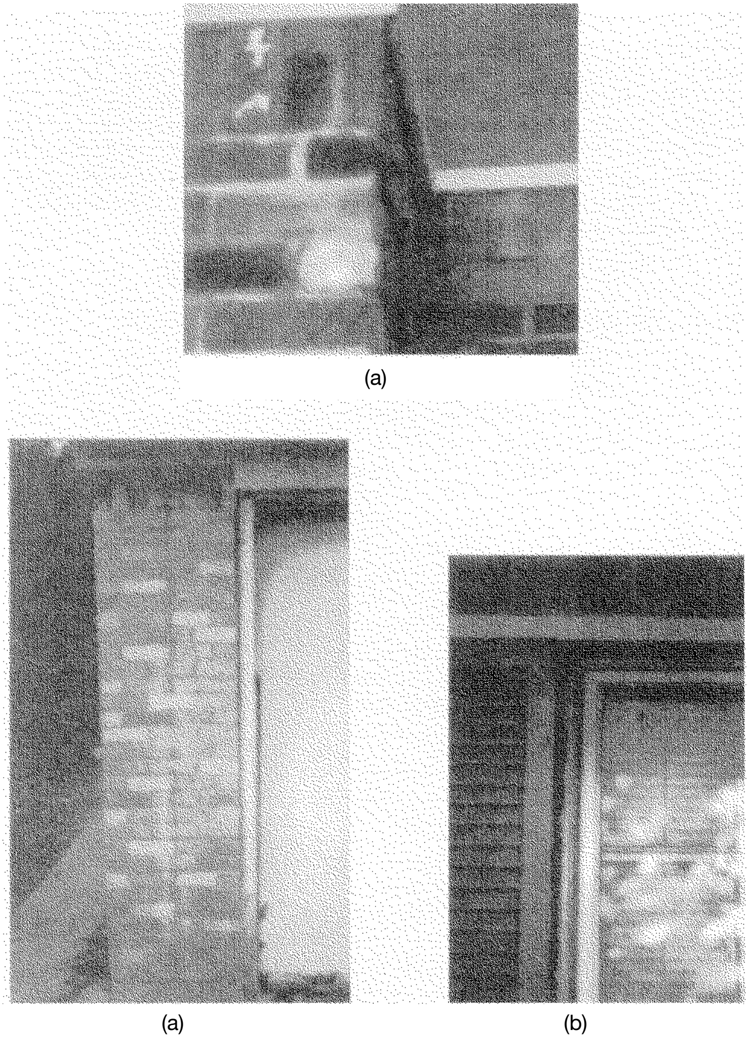
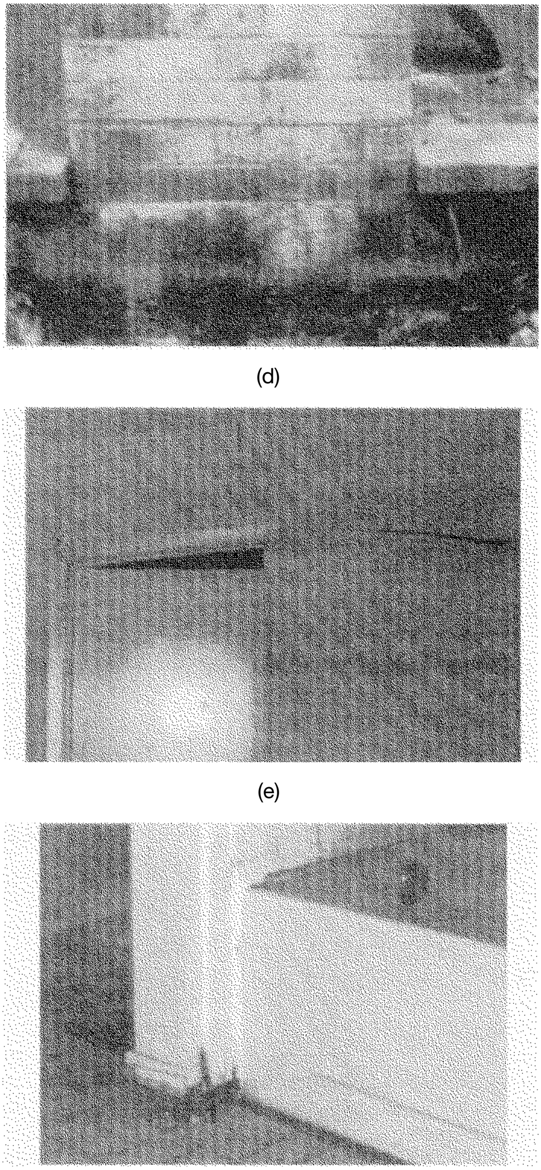


FIGURE 7D.2 Evidence of differential foundation movement. (a) Brick veneer tilted inward; often an indication of interior slab settlement. (b) Separation of brick veneer from garage door jamb. (c) Brick veneer separated from window frame.

3. Check interior doors for fit and operation. Check for evidence of prior repairs and adjustment such as shims behind hinges, latches or keepers relocated, tops of doors shaved.
4. Check the plumb and square of door and window frames. Are the doors square in the frames? Check to see if the strike plates have been adjusted to accommodate the strikers. A relocation might indicate movement. Measure the length of the door at the door knob side and the hinge side. A discrepancy suggests that the door may have been shaved. Also feel the top of the door

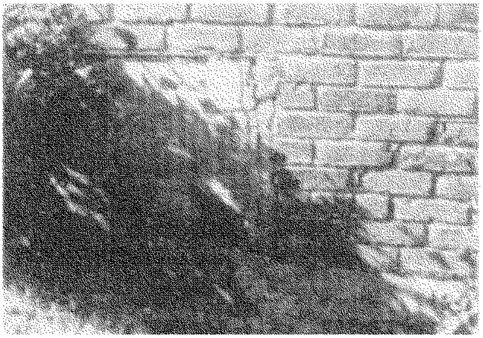


(d)

(e)

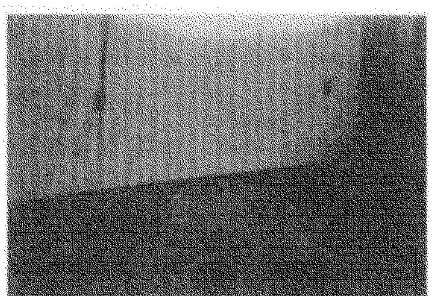
(f)

FIGURE 7D.2 (*continued*). Evidence of differential foundation movement. (d) Horizontal separation in brick mortar. (e) Interior door frame out of plumb, causing sheetrock cracks. This illustrates the effect of upheaval in an interior slab. The high point is to the right side of the door. (f) Interior slab settlement. This shows separation of interior slab from the wall partition. Note also that the bathtub has settled away from the ceramic tile.



(g)

(h)



(i)

FIGURE 7D.2 (*continued*). Evidence of differential foundation movement. (g) Crack in perimeter beam. This illustrates a major crack in the perimeter beam accompanied by secondary cracks in brick veneer and brick mortar. (h) Stair step separation in brick mortar. This shows an obvious crack in the brick mortar, but does it represent a problem concern? First, the crack width is less than ¼ in (0.6 cm). Second, the mortar joints are straight. Third, the cornice trim at the corner is tight. Fourth, no interior damage was noted. Thus, the damage is not one of concern. (i) Slab upheaval. This is a fairly typical representation of slab upheaval. Note the very obvious crown in the slab surface (approximately 4 in or 10 cm) accompanied by vertical separation in wallboard, crack in floor slab, and separation in show mold from slab.

above the door knob. If it is smooth, the door may have been sanded or shaved. If the door rubs slightly at the top, shimming the hinge plate might provide alignment without altering the door.

5. Note grade of floors. A simple method for checking the level of a floor (without carpet), window sill, counter top, etc. is to place a marble or small ball bearing on the surface and observe its behavior. A rolling action indicates a "down hill" grade. (A hard surface such as a board or book placed on the floor will allow the test to be made on carpet.)
6. Inspect wall and ceiling surfaces for cracks or evidence of patching. *Note:* Any cracking should be evaluated on the basis of both extent and cause. Most hard construction surfaces tend to

crack. Often, this can be the result of thermal or moisture changes and not foundation movement. However, if the cracks approach or exceed ¼ in (0.6 cm) in width, the problem is *possibly* structural. On the other hand, if a crack noticed is, for example, ⅛ in (0.3 cm) wide, is it a sign of impending problems? A simple check to determine if a crack is "growing" is to scribe a pencil mark at the apex of the existing crack and, using a straight edge, make two marks along the crack, one horizontal and one vertical. If the crack changes even slightly, one or more of the marks will no longer match in a straight line along the crack and/or the crack will extend past the apex mark. A slight variation of this technique is to mark a straight line across a door and matching door frame. If any movement occurs, the marks will no longer line up. Continued growth of the crack or displacement of the doormarks would be a strong indication of foundation movement.

7. On pier-and-beam foundations, check floors for firmness, inspect the crawl space for evidence of moisture, deficient or deteriorated framing or support, and ascertain adequate ventilation. The crawl space should be dry with adequate access. As a rule of thumb, one ft^2 (0.9 m^2) of vent is suggested for each 150 ft^2 (13.5 m^2) of floor space.
8. Check exterior drainage adjacent to foundation beams. Any surface water should quickly drain away from the foundation and not pond or pool within 8 to 10 ft (2.4 to 3 m). Give attention to planter boxes, flower bed curbing, and downspouts on gutter systems.
9. Look for trees that might be located too close to the foundation. Some authorities feel that the safe planting distance from the foundation is 1 or preferably 1.5 times the anticipated ultimate height of the tree. More correctly, the distance of concern should be perhaps 1 to 1.5 times the canopy width. Consideration should be given to the type of tree. Also, remember that the detrimental influence of roots on foundation behavior is grossly overrated (see Sections 7A, 7B.2, and 9A).
10. Are exposed concrete surfaces cracked? Hairline cracks can be expected in areas with expansive soils. However, larger cracks approaching or exceeding ¼ in (0.6 cm) in width warrant closer consideration.[17,61]

Absence of the foregoing evidence of structural distress suggests one of two conclusions: 1) either there has been no differential movement (hence no need for repairs), or 2) serious cosmetic work has been performed. In the latter case, the evidence will still be present, only much more difficult to detect. Look for 1) tapered or "out of place" trim strips, 2) newly paneled walls or walls covered with a fabric, 3) patches that do not match the rest of the surface in either texture or color, 4) striker or keeper plates adjusted, 5) doors cut or sanded (feel the top edge to see if it has been cut or sanded and measure both sides of the door to determine whether or how much the door has been cut), 6) differences in widths of mortar joints, and also 7) sight mortar joints to assure that joints are all straight and level. Refer also to Section 7D.3. In instances where no distress has occurred, foundation leveling is neither required nor advised and such attempts could create serious damage. Foundations are often constructed out of level. In fact, the tolerance often acceptable for residential and light commercial construction often exceeds 1″ (2.5 cm) over 20 linear ft (6 m). The home owner is not normally aware of this "out-of-level" because the framing carpenters compensate by adjusting the floor plates. Hence, the finished product boasts plumb and level door frames with generally square corners and level cabinets. Grade elevations subsequently taken on this foundation might still show "diselevations" up to 5 in (12.5 cm) over 100 ft (30 m). However, without differential movement (Figure 7D.3), no leveling would be practical or possible.

The differences between grade elevation, deflection, and differential movement are, for some reason, very confusing to many people. For repair purposes, the primary issue of concern is differential movement. Perhaps the following analogy will help bring clarity to the confusion.

In Figure 7D.3, Door A won't latch and clearly does not properly fit frame, an example of differential movement. Note also the sheetrock cracks. This represents a potential for foundation "leveling." Door B fits and latches properly although floor is obviously not level. This door was framed square on an unlevel floor. This situation does not represent a potential for foundation repair.

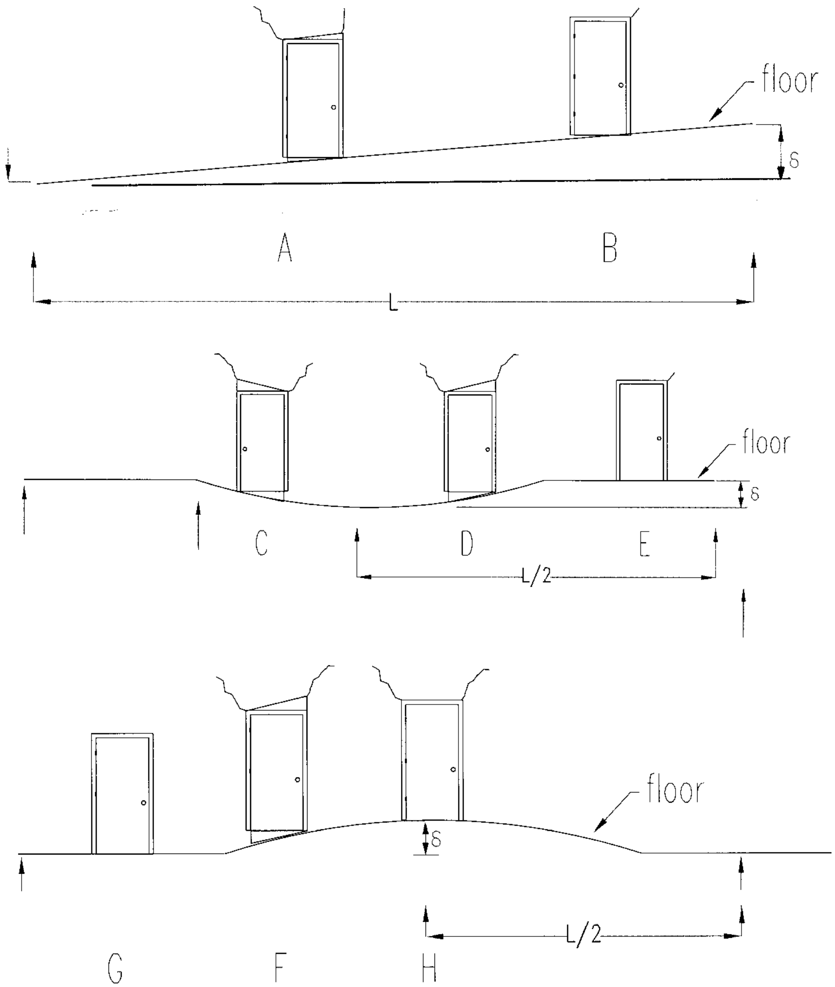


FIGURE 7D3. Evaluating door misalignment.

Doors C and D won't latch and are low on the hinge side. The sheetrock is badly cracked. The center slab has settled and is a strong candidate for foundation repair. In the case of a concrete slab foundation, the likely repair would involve mudjacking. If the foundation is concrete pier-and-beam with concrete interior pier caps, the logical repair procedure would be shim the interior floors or existing pier caps. Refer to Sections 8B.1 and 7B.2

Door E shows no evidence of differential movement, since it is located outside the area of differential movement.

Door F won't latch, clearly does not fit the frame, and is high on the inside (door knob). The sheetrock is severely cracked. This distress needs repair.

Door G shows no distress, since it also is not the subject of differential movement.

Door H shows minor distress, which on its own probably would not suggest repair. However, this door is situated along the peak of the heave. Doors perpendicular to and on a plane with F may show similar movement. However, all doors parallel with the slope of heave will appear similar to door F. This foundation is in need of serious foundation repairs. The procedure would require underpinning the perimeter with the hopes of raising the (more or less stable) lower areas to meet or minimize the heave (δ) Refer to Section 7B.4. As one might expect from the described repair procedure, upheavel is the most difficult of all foundation problems to correct.[13,18]

In fact, repairs are frequently a matter of compromise, " level to extent practical from a cost-results balance and stabilize." Refer also to Section 7A.3. The interior would then be supported and/or leveled as outlined for settlement in the prior example. The foregoing analysis should be used to evaluate or interpret any grade elevation developed from the property inspection performed by either the engineer, house inspector or foundation contractor, as the case my be. Refer also to Section 7B.7.2.

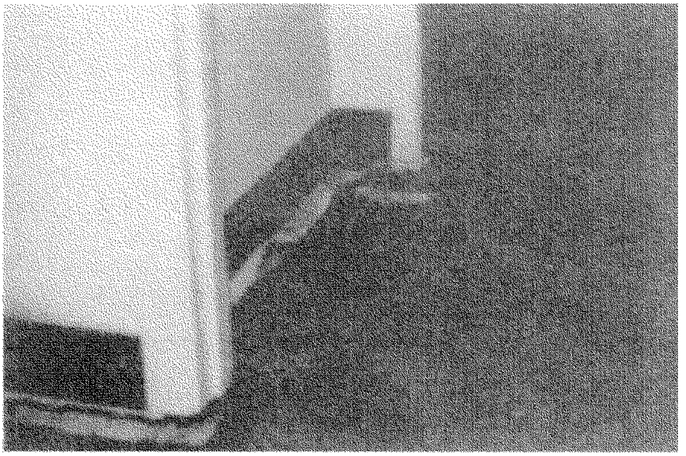
7D.4 IMPACT OF FOUNDATION REPAIR ON PROPERTY EVALUATION

Do foundation repairs impact resale values? This is a question often heard, particularly from appraisers and attorneys. Generally speaking competent foundation repairs should produce an end product at least equivalent to and, more often, superior to the original. Figure 7D.4 shows before and after photos of actual foundation repair. In all instances, the damage was completely reversed. (This is generally, but not always, the case.) The repaired foundation will have no increased susceptibility to future problems. In fact, the properly repaired foundation might be considerably more resistant to future problems. These facts are due, at least in part, to such factors as:

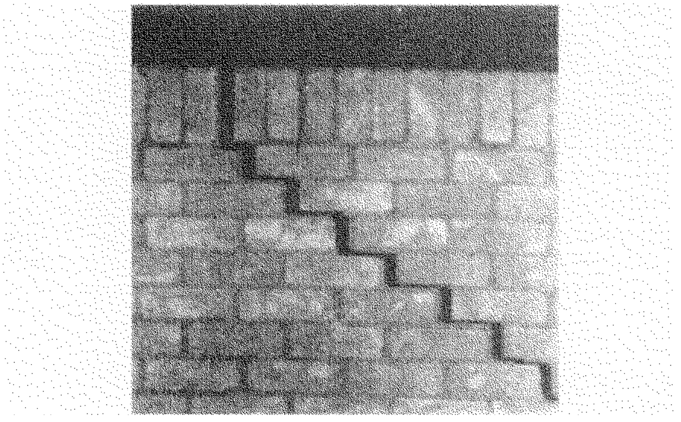
1. Underpinning and mudjacking provide support to the foundation that either did not preexist or had been rendered ineffective.
2. The cause of the foundation failure was hopefully identified and corrected prior to the repair. [With the new (unrepaired) foundations, some problem might exist but not be detected until a visible foundation problem actually develops.]
3. Certain repair procedures (such as mudjacking or chemical stabilization) actually help stabilize the soil moisture, inhibit moisture transfer within the soil, and increase soil resistant to shear. (The grout placed by mudjacking tends to serve somewhat as horizontal and vertical capillary barriers.)
4. A foundation properly repaired by a reputable contractor will be covered by a warranty of some duration. The legitimate warranties generally vary in length from 1 year to 10 years, with restrictions or limitations in coverage after the first year or so. Read any so-called "lifetime" or extended-term warranty carefully. If it sounds too good to be true, you can be sure *it is.*

A foundation is no different than any other inanimate object—it refuses to move unless forced to do otherwise. Eliminate or prevent this force and no movement occurs. In the case of foundations built on expansive soils, the primary force is provided by variations in soil moisture. Add water and the soil swells. Remove water and the soil shrinks. Do neither and no movement occurs. It makes no difference whether the foundation is new, old, or previously repaired.

Proper maintenance and alert observation to the early warning signs of impending distress will prevent or minimize all foundation movement. Drainage and proper water content can be easily controlled by common sense. The "unknown" in the formula involves the accumulation of water beneath slab foundations due to some form of utility leak. The most serious source is some form of sewer leak. As a rule, the latter is usually detected after the fact from the various signs of differential foundation movement as manifested by sheetrock cracks, ill-fitted doors, windows, cabi-



(a)

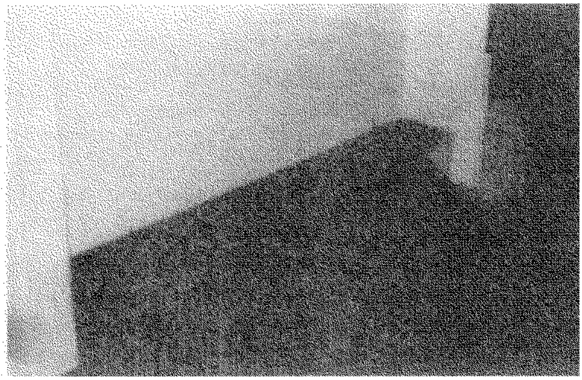


(b)

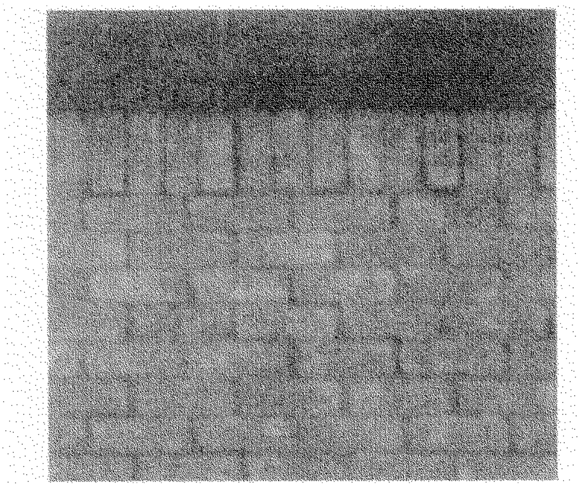
FIGURE 7D.4 Foundation leveling. (a) The floor is separated from the wall partition by about 4 in (10 cm). (b) The separation in brick mortar is an excess of 2 in (5 cm).

nets, etc., or deviated floors. Figure 7B.8.3A shows a simplistic slab behavior as it is being heaved. Figure 7B.8.3b shows what happens to the rebar after the slab is heaved. Catch the problem sufficiently early, eliminate the cause and, in some cases, avoid the need for foundation repair. Notice that no differentiation is given as to age or history of the subject foundation. It makes little difference.

The principal foundation concern in purchasing a property should be whether the foundation was properly designed for the site conditions, competently constructed, and properly maintained. The occurrence of foundation repair would have no negative impact, providing the repairs were properly executed.



(c)



(d)

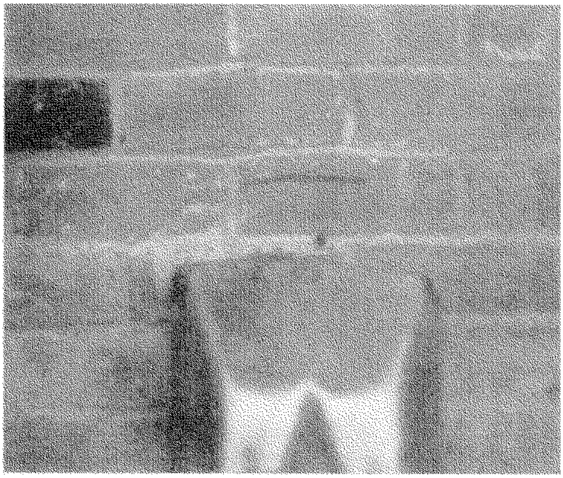
FIGURE 7D.4 (*continued*). (c) and (d) Results from foundation levelings of (a) and (b), respectively. The separations are closed. The end results of foundation levelings are not always so impressive.

7D.5 WARRANTIES AND SELECTING A REPAIR CONTRACTOR

Once the method of repair has been resolved, the selection of a contractor has been narrowed considerably. A next point to consider might be the contractor's warranty.

7D.5.1 Warranty

A foundation will not move unless forced to do so. The force causing foundation movement (in expansive soils) is generally water—too much or too little. The contractor has no control over the availability of water to the foundation. Therefore, most contractor's warranties limit liability for up-



(e)

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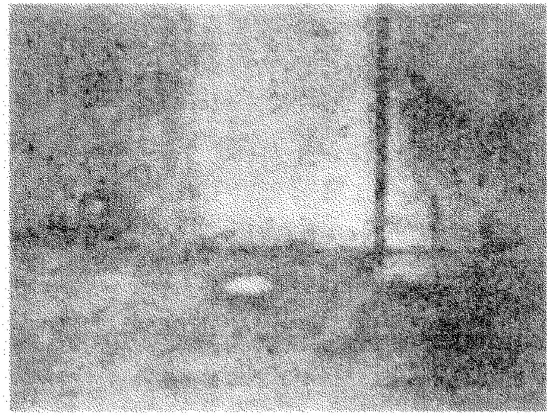

(f)

FIGURE 7D.4 (*continued*). Foundation leveling. (e) Mudjacking in progress. The shovel and scribed marks monitor the raise of slab. At this point, the raise has been more than the width of a brick. (3½ in or 9 cm). This is shown by the distance between the shovel point and the original scribed mark (the shovel handle is on a stationary surface.) (f) In this example, the shovel point is on the stationary surface. The floor slab is down about 4 in (10 cm). (g) The floor is restored to it original grade.

heaval, whether caused by deficient drainage (ponding), excessive watering, or domestic sources. Excluding these events, the foundation is not likely to experience significant movement anyway. The quality contractor's standard limited warranty (usually 1 to 2 years) will frequently: 1) waive the upheaval restriction on a limited basis, 2) provide transfer to new owners, and 3) cover differential movement in excess of ¼″ to ⅜″ (0.6 to 0.9 cm). (Lower ranges of movement are allowed since virtually all foundations on expansive soils will move. In fact, slab foundations were originally designed to accommodate soil movement (often 0.3%). Hence, the term "floating slab" (refer to the Introduction and Section 7B.8.4). Many contractors will also offer a limited *extended* warranty to cover the repaired foundation for an extended period of time, usually 3 to 10 years. Use of the "extended coverage" would cost the consumer a reduced amount, often about one-half the normal charges. The "extended coverage" is frequently free of extra cost to the consumer unless, of course, work is actually performed.

Be wary of "lifetime" warranties or other ridiculous warranty claims. The verbiage of the lifetime warranty often includes several or all of the following caveats:

1. Covers *initial owner only* or reduces to 10 year warranty if property changes hands within 10 years.
2. Does *not cover* foundation movement due to *heave.* (Over 70% of all slab foundation repairs are precipitated by upheaval.)
3. Specifically excludes settlement (or movement) of interior floors.
4. Covers materials only.
5. Limits coverage to replace or rework *material* installed by contractor. Additional material or "new" installations subject to normal charge.
6. Limits coverage to *vertical settlement.*
7. Requires differential deflection of 0.4% (1 in over 240 in), as opposed to the "normal" 0.3% (1 in over 360 in), to define failure. Refer to Section 7B.8.4.
8. Exempt all "consequential" damage such as landscaping, floor covering, sheet rock, brick, masonry or concrete, utilities, etc. (This is basic to most, if not all, warranties.)

The terms *lifetime* and *warranty* are somewhat deceptive, to say the least. Aside from the foregoing, for a lifetime warranty to be taken seriously it would need to be backed by a *substantial,* irrevocable, escrow account. The balance for such a fund should start at perhaps $500,000.00 (U.S.) and be increased annually as the company increases exposure. A half million dollars would assure full dollar coverage on a mere 30 to 100 jobs, dependant, of course, on the size of the composite projects and assuming that neither punitive nor treble damages become involved. (A major foundation repair company might perform upwards of 40 jobs per month.) Many warranties, particularly the so-called lifetime warranties, are equivalent of insuring swimming pools against theft. The rates are quite cheap and the coverage can be lifetime; however, the protection is meaningless.

When all is said and done, the best and most consumer-protective warranty is probably represented by the "normal" 2 Year Standard Limited Warranty. In effect, these warranties state "Contractor warrants work performed by the contractor against *settlement* in excess of ¼ in for a period of 24 months after completion of initial work." Note particularly the reference to settlement. Most warranties (and contractors) exclude upheaval because this problem is caused by water accumulation and is beyond the control of the contractor.

Most recurrent problems, which are generally the responsibility of the contractor, occur within the first year. Even then, the problems are likely to be quite minor. The principal exception to this represent those circumstances where: 1) a slab foundation is underpinned without the prerequisite mudjacking, or 2) defective methods are used to underpin the perimeter and proper mudjacking is not performed. Sometimes, more than two years are required for these problems to become apparent to the consumer. However, the Extended Limited Warranty (5 to 10 years) will adequately cover any of these contingencies.

Upheaval represents the largest cause for recurrent foundation failure of slab foundations on expansive soils. This problem is exempted from most, if not all, warranties. However, some companies will accept this as a covered item *strictly* as a public relations concession. Where the concession is made, the coverage is provided by the Standard Limited Warranty.

The foregoing analysis is understood when one remembers the principal of physics—"Nothing moves unless *forced* to do so." With slab foundations or expansive soils, this force is generally water and, most often, excess water. This occurrence is beyond the control of even the most conscientious contractor.

In other cases, recurrent problems can result that are not related to expansive soil behavior. Generally, these are also exempted by the contractor and might include such instances as sliding or embankment failure, erosion, consolidation of fill due to decay of organic content or collapse of soil structure, thaw of permafrost, etc. (see Section 7A).

If you have a warranty concern, consult your attorney for an opinion. Be more concerned about what the warranty covers and less about the duration.

7D.5.2 Selecting a Contractor

Next in the process to find the preferred contractor is to consider individual credentials. To help ensure the selection of a competent contractor, consider the following criteria:

1. The contractor has at *least* five years experience (preferably ten) in your locale dealing with problems similar to your own. Request proof of experience. Contact the *Yellow Pages* to determine the contractors' continuous years therein. There are companies in the *Yellow Pages* who advertise "X" years experience. In reality, a number of different employees may each have a few years experience, which totals "X."
2. The contractor should be fully insured and licensed, where applicable.
3. The contractor should be financially stable. A bankrupt contractor offers little or no protection or recourse for complaints.
4. Check with the Better Business Bureau, Attorney General, and County and District Courts to evaluate litigation and/or unresolved disputes. Any contractor in business for an extended period of time is subject to complaints. The concern is how the disputes were resolved.
5. Does the contractor have adequate technical competency? Repairs are neither repetitious nor sometimes even similar. The contractor must have the ability to adapt.
6. Ascertain that the contractor owns all the proper equipment with which to do your job. Use of unfamiliar equipment has been known to result in serious property damage. The contractor should do his own work with company personnel. In some cases, *limited* subcontracting has proven acceptable.
7. Hire a contractor who pays his sales force a salary. Many contractors compensate sales on a per pier sold basis (often $10 to $15). This gives rise to an overabundance of piers with corresponding neglect to other areas.

With careful reliance on the foregoing paragraphs, an honest, capable, and economical foundation repair should be the reward. As stated in Part 8, the principal goal is to identify and eliminate the cause of the problem. This done (and assuming competent repairs), recurrent movement is unlikely unless a new cause develops.

7D.6 BUILDING CODES

New construction is governed, to a large extent, by municipal or national codes.[29,36,90,93] Most major cities have written codes allegedly based on the Council of American Building Officials (CABO) publication for one and two family dwellings, 1989 Edition. This information does not provide any real assistance, as far as foundation repair is concerned. In fact, the author is not aware of any significant industrial, municipal, national or governmental code covering foundation repair, although some maybe in the offing.

Many municipalities in the United States have established city policies intended to control at least certain aspects of foundation repair. Generally, this is through the requirement of building permits. At the end of the day, about the only benefit from this policy is to the city coffers, at the expense of the citizens. The municipality offers no guidelines as to proper repair procedures. The code merely requires the seal of a P.E. In Texas, the P. E. might be schooled in such areas as aeronautical, sanitary, industrial, electrical, mechanical, agricultural, petroleum, mining, or, if the citizen is extremely lucky, civil or structural engineering. In many cases, this "engineer" has neither leveled a foundation in his entire career nor does he provide any mathematical analysis that would verify his design or support his credibility. Refer also to Section 9A.

7D.7 LEVELS OF EVALUATION (PROPOSAL)

Due to the high number of complaints (and resulting litigation) against engineers performing design or evaluation of residential foundations, the industry has been under intense pressure to provide a commentary or policy that would minimize that legal exposure.

One such entity, the Texas Board of Professional Engineers, made some attempt in this direction. The Residential Foundation Committee (RFC) produced the Policy Advisor (PA), 09-98-A, in late 1998.* The purpose of this PA was stated as two-fold:

A. Provide recommendations to various non-engineering entities on how to minimize the probability that residential foundation problems, currently encountered by home owners, will occur.
B. Provide practicing licensed professional engineers with guidance in the preparation of design and evaluations of residential foundations to minimize the probability that problems, currently encountered by home owners, will occur.

(Readers interested in design should obtain a copy of both the PA and the RFC documents from the Texas Board of Professional Engineers, P.O. Drawer 18329, Austin, Texas 78760-8329.)

Inspection/evaluation/repair was approached from a cursory review. The essential points that *might* be of benefit to these practitioners could be summarized as follows:

If a the development of a repair plan or forensic report is the purpose for the evaluation, the engineer must establish the minimum level of evaluation required to adequately accomplish that purpose. Inherrent in this rule is the notion that an engineer is to provide an optimized, cost-effective design (repair).

The board recommends the use of the following three levels of evaluation design.

1. *Level A*. This level of evaluation will be clearly identified as a report of first impression conclusions and/or recommendations and will not imply that any higher level of evaluation has been performed. Level A evaluations will typically:

a. Define the scope, expectations, exclusions, and other available options
b. Interview the home owner and/or client if possible
c. Document visual observations personally made by the engineer during a physical walk-through
d. Describe the analysis process used to arrive at any performance conclusion
e. Provide a report containing one or more of the following: observations, opinions, performance conclusions, and recommendations based on the engineer's first impressions of the condition of the foundation

2. *Level B*. This level builds upon the elements found in a Level A evaluation. In addition to the items included in Level A, a Level B evaluation will typically:

a. Request and review available documents such as geotechnical reports, construction drawings, field reports, prior additions to the foundation and frame structure, etc.

*This policy was removed from the Texas State Board web page and at least temporarily shelved in early 2000. It is awaiting more review.

b. Determine relative foundation elevations to assess levelness at the time of evaluation and to establish a datum

c. If appropriate, perform noninvasive plumbing tests, recognizing that additional invasive testing is also available

d. Document the analysis process, data, and observations

e. Provide conclusions and/or recommendations

f. Document the process with references to pertinent data, research, literature, and the engineer's relevant experience

3. *Level C.* This level builds upon the elements found in the Level B evaluation. In addition to the items included in Levels A and B, a Level C evaluation will typically:

a. Conduct noninvasive and invasive plumbing tests as required by the engineer;
b. Conduct site-specific geotechnical investigations as required by the engineer
c. Conduct materials tests as required by the engineer to reach a conclusion
d. Obtain other data and perform analyses as required by the engineer
e. Document the analysis processes, data and observations
f. Provide conclusions and/or recommendations

The stated levels of evaluation will likely provide some shield against litigation to those engineers responsible for foundation evaluation reports. At least this wording might both limit the engineer's exposure and better communicate to the consumer the extent of the evaluation.

However, PA seems to lack provisions that would either improve the quality of the foundation evaluation or the competency of the engineer to thus afford the consumer a degree of confidence in the engineering community. Refer also to Section 9A.

Another aspect, obvious by its absence, is any attempt to standardize or monitor repair procedures. Refer to Section 7B.4. Any efforts along these lines would be a tremendous benefit to the consumer. Before any substantial movements can be made in this direction, some group or individual must devote the time (and weather the criticism) necessary to produce the meaningful data. Courage is required to stand up and tell a contractor or engineer that his repair method is inappropriate, too expensive, or both.

If competent foundation evaluation followed by competent repair could be established, both the consumer and the engineering community would truly benefit.

On the brighter side, anything done to improve and standardize foundation design will favorably impact foundation repair/evaluation procedures.

7D.8 CONCLUSIONS

Regardless of the foundation repair contractor's skills or methods, he can generally locate an engineer who is willing to sign off on any proposal and/or drawing(s). This secures a building permit. Next, the city requires inspections with the intent of ascertaining that the work performed is according to the engineers' approved design, not with any regard as to the *merit* of the repairs. Generally, this provides little service to the consumer, unless he has hired a less than honest contractor. The city inspectors can, in fact, threaten the success of a job, principally by delaying the pouring of concrete. Drilled pier shafts should be poured as quickly after drilling as possible. An overnight delay can be detrimental to the function of the drilled pier/haunch.

A reasonable and effective code covering foundation repair would be a substantial asset to the owners and repair contractors alike. However, *serious* thought must be given to establish a valid, workable, reasonable code (see Sections 7B.1 through 7B.6). So much “hocus pocus” is written about foundation failure and repair that it has become increasingly imperative that fiction needs to be separated from fact. Writers have attempted to base a *trend* on one or two questionable case histories. In other cases, repair procedures have not been subjected to a *thorough* mathematical analysis to evaluate such characteristics as component resistance to: 1) shear, tensile, lateral, or compressive stress, 2) moments that induce misalignment or cause failure, 3) potential heave, 4) chemical attack or deterioration, and 5) load attributed to the weight of the structure.

The specific design of the foundation, the site conditions and the bearing soil characteristics are examples of other factors to be considered when adopting repair procedures.

Proper forethought and the input of knowledgeable, experienced, unbiased contractors in concert with equally capable engineers, geotechnicians, and/or perhaps architects could well develop a usable foundation repair code. Actually, the need would require a series of codes applying to variable conditions. Refer also to Section 9A.

SECTION 7E

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P • A • R • T • 8

FOUNDATION FAILURES AND REPAIR: HIGH-RISE AND HEAVY CONSTRUCTION

SECTION 8

FOUNDATION FAILURES AND REPAIR: HIGH-RISE AND HEAVY CONSTRUCTION

DOV KAMINETZKY

8.1 INTRODUCTION

8.1.1 High-Rise and Heavy Construction

Foundation failures of high-rise and heavy construction generally do not differ from those of low-rise structures and single residences. Foundation failures are all caused by human errors and are the result of either one or a combination of these three types of errors:

1. Errors of knowledge (ignorance)
2. Errors of performance (carelessness and negligence)
3. Errors of intent (greed)

This section will help to minimize future failures resulting from the first two types of errors. This famous quote says it well: "A wise man learns from the mistakes of others. Nobody lives long enough to make them all himself."

The consequences of high-rise failures are often very serious and most devastating because of the large scale and size of such structures. It is also self-evident that the potential for high loss of life is also great.

The pattern of foundation failures in high-rise structures is unique and differs from patterns of failures caused by forces such as lateral wind pressures, earthquakes, or punching shear of slabs. Since the foundations transfer the structural loads such as vertical dead and live load and lateral wind load to the ground at the bottom of the structural frame, foundation failures will typically *telegraph* their effect *upward* for the full height of the structure. Such failure also may affect the stability of the high-rise structure and often may result in a noticeable tilt of the structure. This is especially so in cases of unequal support. (See Figures 8.1 and 8.2.)

8.1.2 Foundation Failures

The foundation is the structural element providing support for the various loads acting on the structure. It is the link between the structure and its eventual support, which is the soil itself. The actual transfer of load may be by direct bearing on soil or rock or by intermediary elements such as piles or caissons.

When we speak about foundation failures, we often refer to both the failure of the structural elements of the foundation such as footings or piles and the failure of the soil itself. Whereas the first type of failure may be the result of overloads on the foundation or its understrength, the second type results from overconfidence in the borings or other subsurface information or loss of bearing value because of adjacent work. Foundation failures resulting from failure of the footing itself is a rare occurrence. One such case was the fracture of the post-tensioned concrete foundation mat in Washington, D.C., which was caused by a design error. (See Figures 8.49 to 8.51).

The foundation is usually a human-made structural element, whereas the soil is a material found in its natural state, disturbed or undisturbed by humans. Artificially made soils are also used in construction (these are mechanically compacted soils or soils made totally from nonnatural materials). The construction of a foundation introduces new conditions into the soil. This happens as the subgrade is exposed and, therefore, unloaded, as changes in the internal friction of the soil are effected by blasting or by the densification of soil by pile intrusion.

Ground conditions often vary considerably from one location to another within the confines of a single construction site. Sometimes the variation is so great that, when two borings do show' identical soil layers, the validity of all borings will be questioned. Rock exposures may range from fine gray syenite, hard seamy limestones, granites, to schists of various hardnesses. These vary to such an extent that they are ringing hard or so soft that a pipe pile will penetrate over 10 ft (3.0 m) before indicating a 30-ton (267-kN) resistance. In some areas even seams of serpentine and asbestos are found.

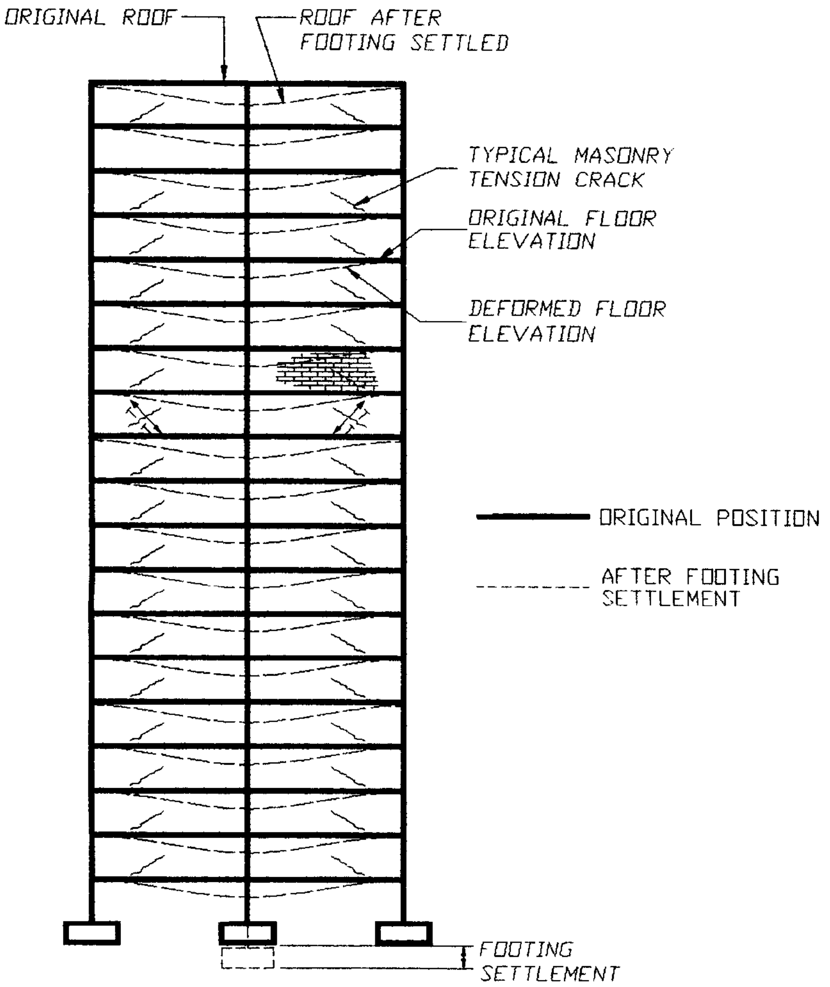


FIGURE 8.1 Settlement of center column—high rise—showing effects of settlement of center column telegraphing upwards of the roof of the high-rise structure.

However, granular materials will range from hardpans to uniformly sized, running fine sands. Soils also often contain clay deposits and organic silts. And, of course, there are the exposed and buried mud deposits with organic layers intermixed in a series of lenses to considerable depths. Because natural soils are often so varied and inconsistent, difficulties in foundation design and construction are expected and are encountered, often with associated trouble.

Actual *water* level in rock excavations sometimes has no relation to the level indicated by borings or to conditions next door. A typical example was the completed work for the New York Lincoln Center underground parking facilities and mechanical plant, where groundwater was first encountered at quite a high level. This water later disappeared after some anchorage drill holes were completed; it was later discovered almost 20 ft (6.1 m) lower than the groundwater encountered in the Philharmonic Hall, adjacent to it.

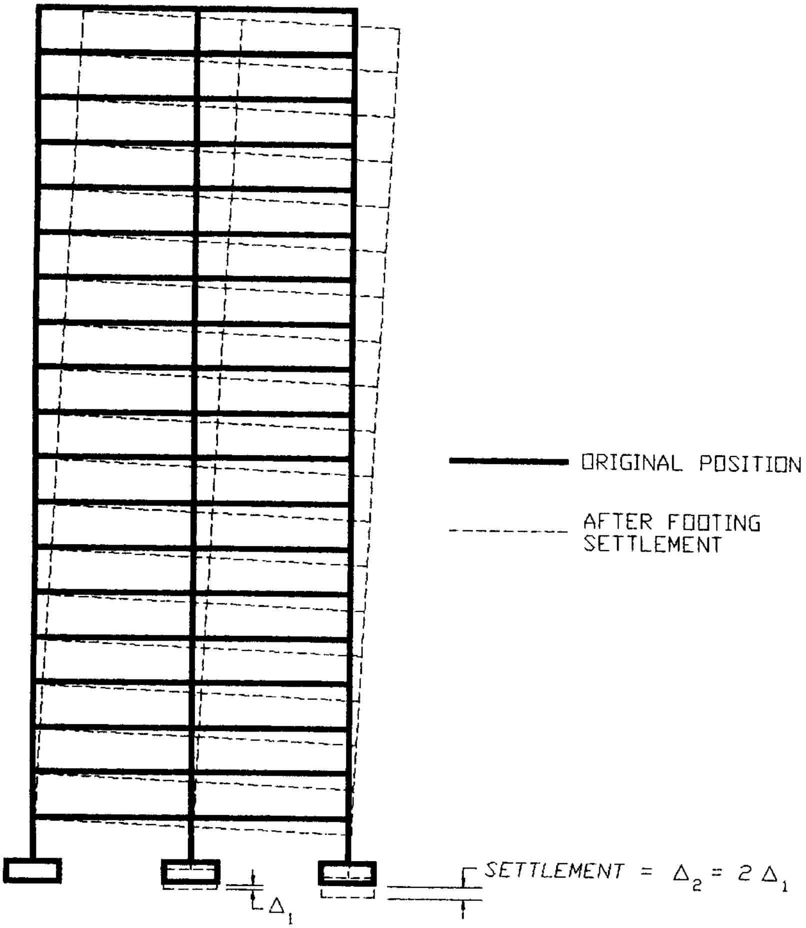


FIGURE 8.2 Structural tilt—a result of footings settlement—showing high-rise structure tilted sideways as a result of proportional footings settlement.

Bed rock surfaces are often very erratic and irregular. During the excavation for the Sixth Avenue subway in New York City during the 1930s, it was found that a strip only 100 ft (30 m) wide was 40 ft (12 m) lower on one side than on the other.

During the construction of the new Bellevue Hospital in Manhattan in 1960, the main hospital building was supported on piles as long as 100 ft (30 m). The adjacent parking facility, which was located no farther than 50 ft (15 m) away, was founded on concrete piers bearing directly on rock.

It is not uncommon to find large vertical mud holes or wide gaps filled with silt within a rock structure.

With such erratic rock strata, rock blasting has a serious effect on the behavior of rock. Rock seams must be found, traced along their length with the seams cleaned and packed and tied across with grouted dowels and rock anchors. Rock as a supporting subgrade can be made safe and sound, but only with careful planning and special work.

Granular soils are found in different varieties. There are some very good and some not-so-good granular soils. Some dense consolidated glacial sandy gravel deposits can safely sustain 10 tons per square foot (957 kPa), whereas uniformly fine grained and very loose silty sands are treacherous and extremely difficult to control. Glacial deposits also vary. Sometimes old sand fills are taken to be natural deposits. Standard borings contribute little information to distinguish the man-made from the natural geological sand layers. Only actual load tests are effective indicators, which often will show surprising looseness of these fills.

Layers of varied *silts and clays* cover many rock troughs and old shore lines. These layers are quite stable when left alone but become liquid when disturbed. Dewatering must be done slowly, to permit the seams to drain out, otherwise "boils" and collapse of braced sheeting excavation will result. Pile driving also generates local liquid conditions that flow readily. The plastic flow of the shore lines continuously pull the river structures outward. Even structures supported on piles carried to rock are affected and rotate with time.

Buried layers of *peat* are sometimes interspersed with silt or sand deposits and are found at great depths. Where piles receive their support at soil layers overlying peat deposits, serious settlements are common. Also, where pumping occurs in areas where footings are bearing above peat layers, settlements are common.

Pumping of water out of weak soils is known to cause consolidation of these soils and result in settlement damage to existing structures. Several precautionary methods are presently being employed to avoid this damage. Water-tight sheeting enclosures with deep cutoffs and internal pumping has been often found to be an adequate solution.

It is well recognized that all loads must be transferred to the underlying soils so that the resulting settlements can be tolerated by the structure without distress. At the same time, stability must be maintained over the life of the structure.

The ten most common categories of foundation failures are:

1. Undermining of safe support
2. Load transfer failure
3. Lateral movement
4. Unequal support
5. Drag down and heave
6. Design error
7. Construction error
8. Floatation and water-level change
9. Vibration effects
10. Earthquake effects

A foundation failure obviously is a serious event, since such a failure may trigger the collapse of the entire structure. This is critically so because most structures are based on structural systems whereby the support of the upper levels always depends on the structural integrity of the lower elements. Ironically, some foundation failures have been economical successes. One such example is the Leaning Tower of Pisa in Italy (Figure 8.3). A close look at this tower will reveal that the tower started to lean during its construction. The reason for this conclusion is a slight change in the slope of the tower, near its midpoint. It is an indication that the builders attempted to correct the foundation-settling problem (Figure 8.4).

7.2. UNDERMINING OF SAFE SUPPORT

Robert Frost in his poem "Mending Wall" (published in *North of Boston*) describes the troubles with stone walls in New England and concludes, "Before I built a wall I'd ask to know what I was walling

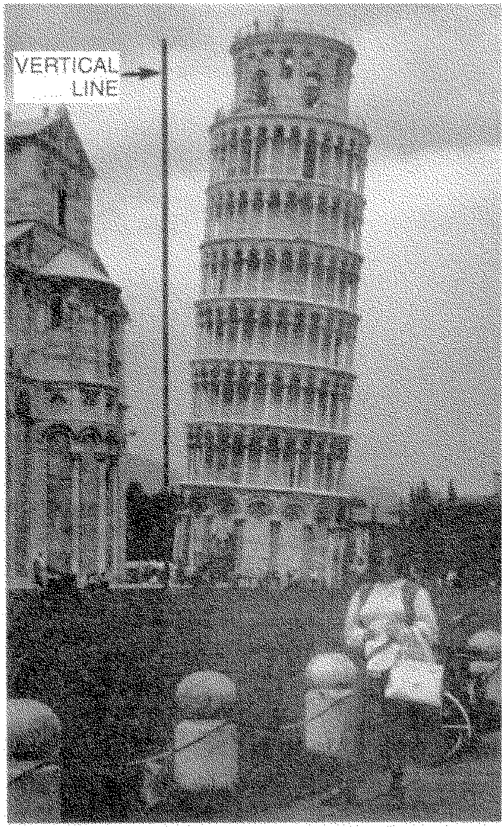


FIGURE 8.3 Leaning Tower of Pisa.

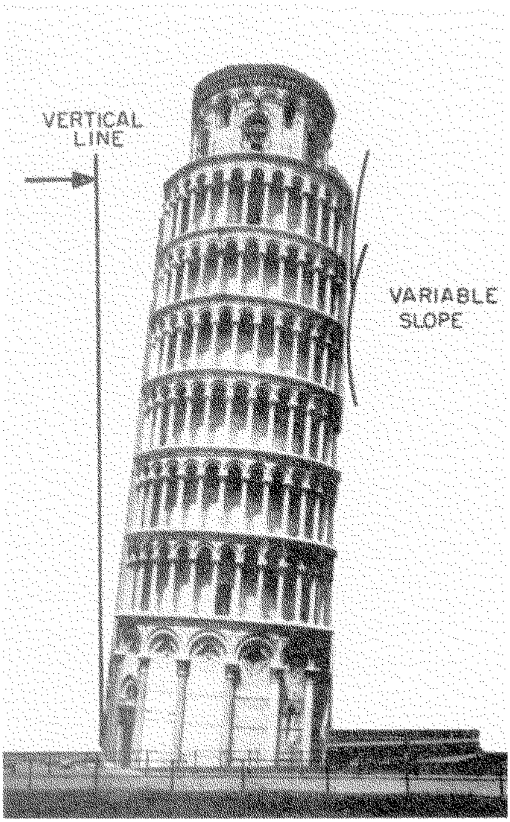


FIGURE 8.4 Leaning Tower of Pisa—variable tilt.

in or walling out."[1] To paraphrase this warning, before a foundation is designed, one should know *what loads are to be carried and what soils are expected to carry the loads.*

The need for a thorough soil investigation prior to the undertaking of a construction project cannot be overemphasized. In addition to the careful study of the soil strata directly below the proposed structure, existing adjacent structures must be reviewed with care. The need for temporary supports must be evaluated. A well-designed bracing and shoring system is often needed to prevent a lateral shift. A permanent support structure such as underpinning should be installed where the new construction will undermine an existing present support system. Where these provisions are ignored or entirely omitted, serious distress follows (Figure 8.5), and sometimes tragic consequences result (Figure 8.6). Another example is the total collapse of a five-story building on 34th Street in New York City that occurred when the vertical support was lost as a result of loss of lateral restraint. This happened when the excavation for a new high-rise building came too close to the foundation of the existing old building. Fortunately, the collapsed building had been evacuated a short time earlier because of unsanitary conditions. As a result of the collapse, the rubble filled the cellar completely and piled up almost to the second floor level.

Excavations for new sewer trenches adjacent to existing buildings have frequently caused undermining of footings resulting in distress and even total collapse. In the President Street collapse in Brooklyn, and similar other collapses, several occupants of existing buildings were killed or injured

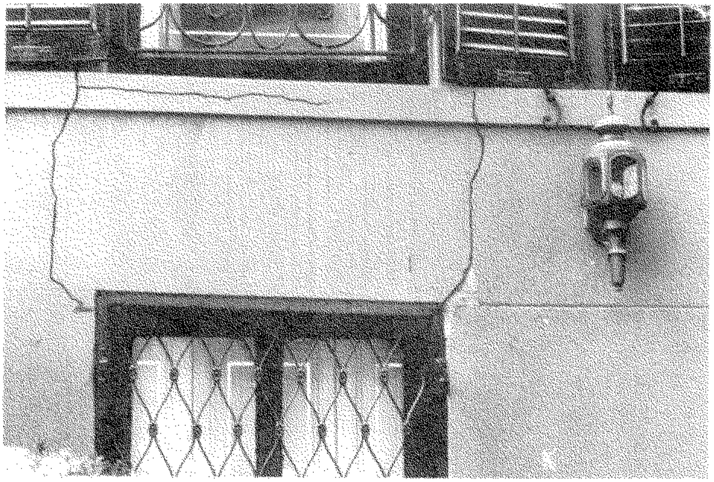


FIGURE 7.5 Structural damage caused by adjacent excavation.

FIGURE 7.6 Building collapse caused by adjacent excavation.

when their buildings were undermined. When these excavations were close to the existing footings, and within the influence line (Figure 8.7), the footings were undermined, often with disastrous results.

8.2.1 New York Suburb Sewers

Construction for a new sewer main project in Brooklyn required the excavation of deep cuts adjacent to existing old residential buildings. The sewer lines were designed to run too close to existing footings. Not only were the new cuts within the influence lines of existing footings, but the contractor used vibratory pile drivers to install the piles. There was no monitoring of the vibration levels during pile driving. As a result, many nearby buildings settled, cracked, tilted, and shifted laterally. After the damage occurred, diagonal braces were added for lateral support (Figure 8.8).

One building collapsed, killing one of the tenants (Figure 8.9). The investigation revealed that the building was constructed using timber footings, a construction method common at the turn of century. The footings were found to be in good condition but had been disturbed by the sewer activity. Once the excavation was within the influence line, vertical support was lost, causing footing set-

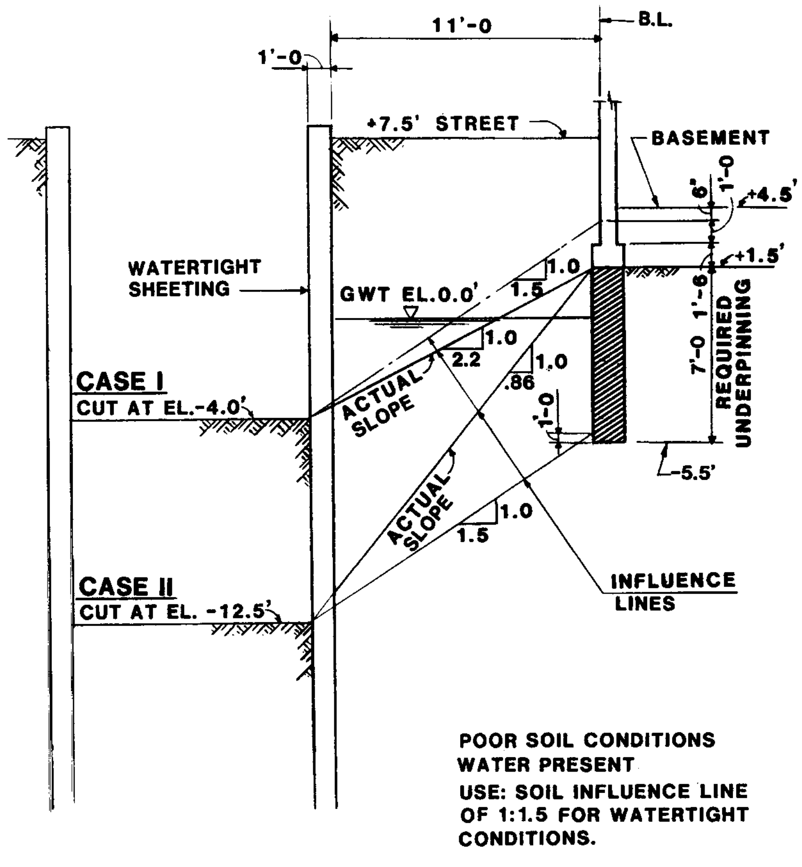


FIGURE 8.7 Bracing structures for lateral support.

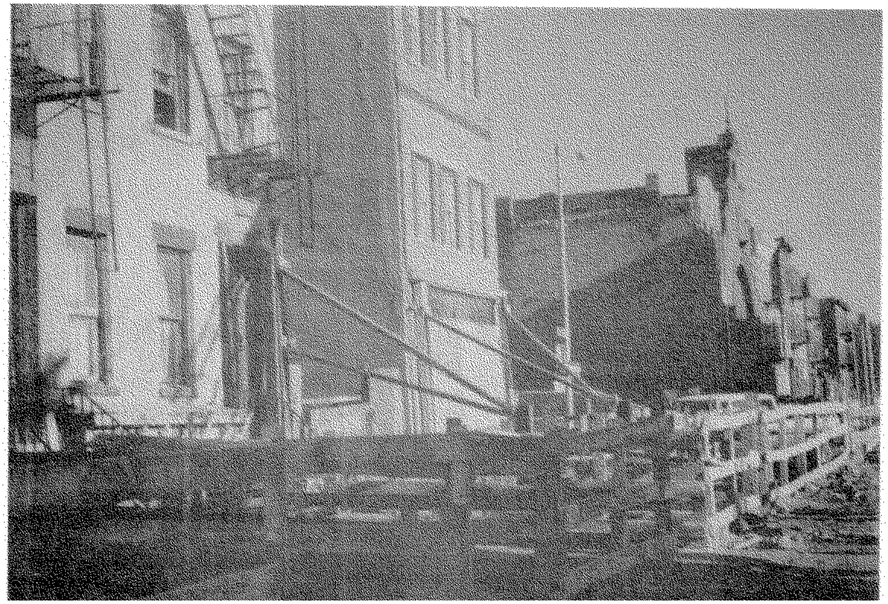


FIGURE 8.8 Bracing structures for lateral support.

FIGURE 8.9 Building collapse—ineffective bracing.

tlements. The high-grade steel bracing system (Figure 8.10) could not save this building, because soil support was lost *below* the footing level.

To restore the integrity of the vertical support it was initially decided to underpin the buildings. The bid prices obtained for underpinning were so high, however, that it was more cost effective to buy the damaged buildings and demolish them.

Lessons to be learned:

1. Careful *preconstruction* study to determine the need for protection and underpinning of adjacent structures is required. This study should review existing plans, taking sufficient soil borings and evaluating the condition and strength of existing buildings and other structures.
2. Excavations for sewers or other utilities should be moved away from existing buildings and so located that the excavation cut is outside footing influence lines.
3. Use of vibratory equipment for pile driving should be avoided in loose sands and similar soils.
4. Vibration readings, using seismographs, should be taken as often as needed to establish the suitability and proper energy of the driving hammers.

8.2.2 Electronics Plant, Upstate New York

An electronics plant was under construction when the entire building started to sink vertically and slide laterally toward the low adjacent valley. Instrument survey readings confirmed these visual observations. The footing movements, as is usual in these cases, were accompanied by distress in the building in the form of cracking of slabs and concrete grade beams (Figure 8.11).

The main question that had to be answered was what was the effect of large movements of the order of 4 in (102 mm) on the structural steel frame and its connections (steel bolts)? To answer this question, we had to determine the level of stress in the loaded and deformed steel beams and the ac-

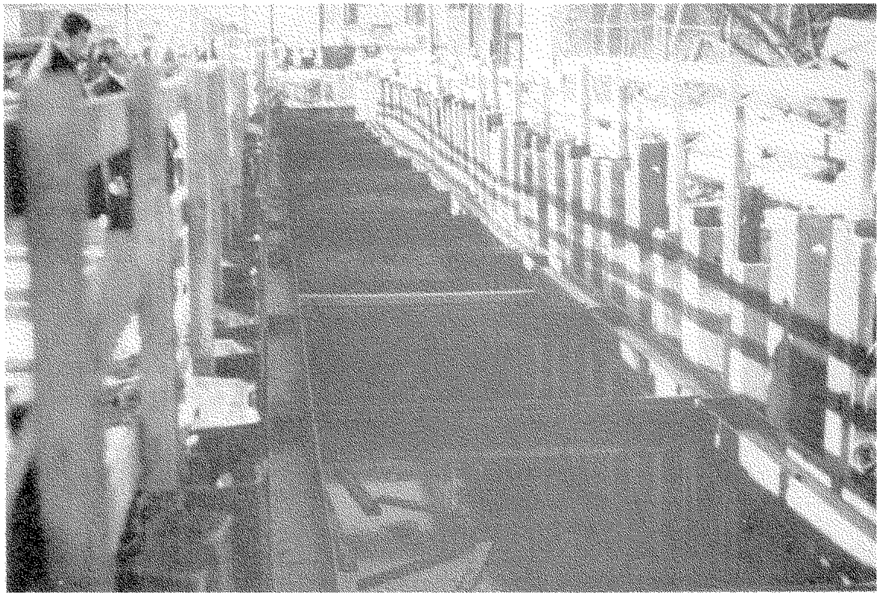


FIGURE 8.10 Bracing excavation cut.

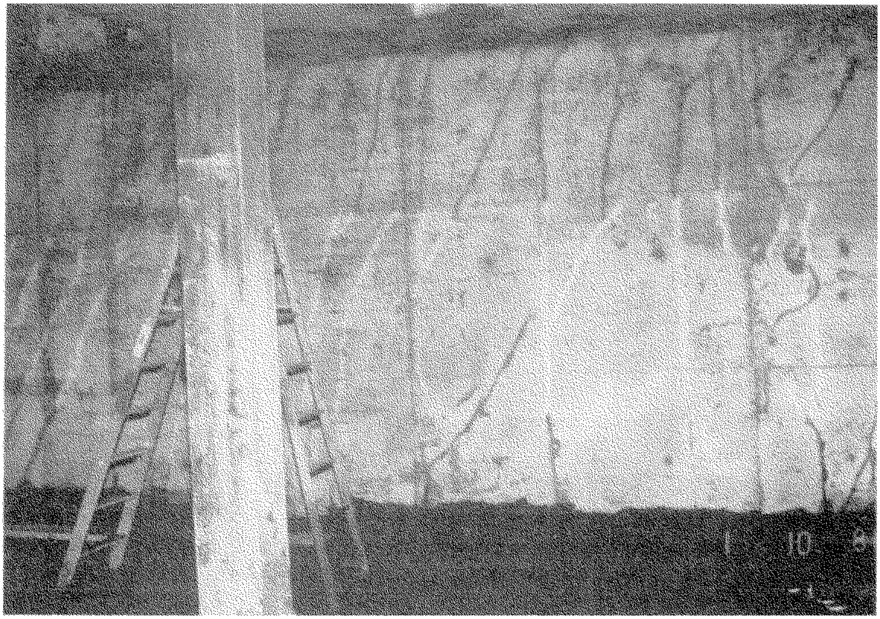


FIGURE 8.11 Structural failures (cracking) in deep-grade beams.

tual loads in the connection bolts. For that purpose we used the innovative stress-relief method (Figure 8.12). This method, which our firm pioneered, proved that the level of stress actually present in the steel and its connections was low and acceptable.

This procedure requires the attachment of electrical strain gages to the steel flanges of the beams. Through each strain gage, a hole is then drilled directly into the steel. The reduction of stress at the hole is measured by the strain gage connected to a Wheatstone Bridge. The relieved stress thus measured gives the approximate stress level then existing in the loaded steel beam. The only structural corrective measure that was executed was to enlarge the existing footings, so as to enable the structure to support the additional live loads without considerable settlements.

The method of repair is shown in Figures 8.13 and 8.14.

Lessons to be learned:

1. High footings adjacent to severe soil slopes should be designed using low allowable bearing pressures to reduce possible settlements.
2. In extreme cases, the use of retaining walls should be considered.
3. The actual stress level in a deformed structure is the true yardstick for evaluating the adequacy of a distressed structure.
4. The stress-relief method proved to be an effective tool in strength evaluation of distressed structures.

8.2.3 East Side Hospital, New York City

During excavation for a new high-rise hospital structure, an alarm went out to everybody connected with the project. It was discovered that, as a result of rock removal by the foundation contractor, the adjacent high-rise apartment building to the west was precariously sitting on a “sliver” of rock (Fig

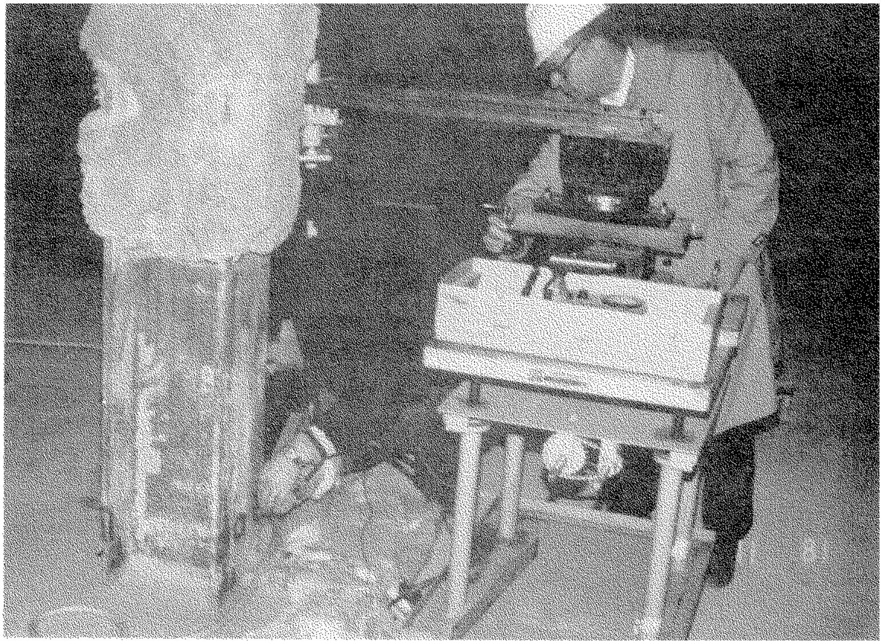


FIGURE 8.12 Stress-relief test.

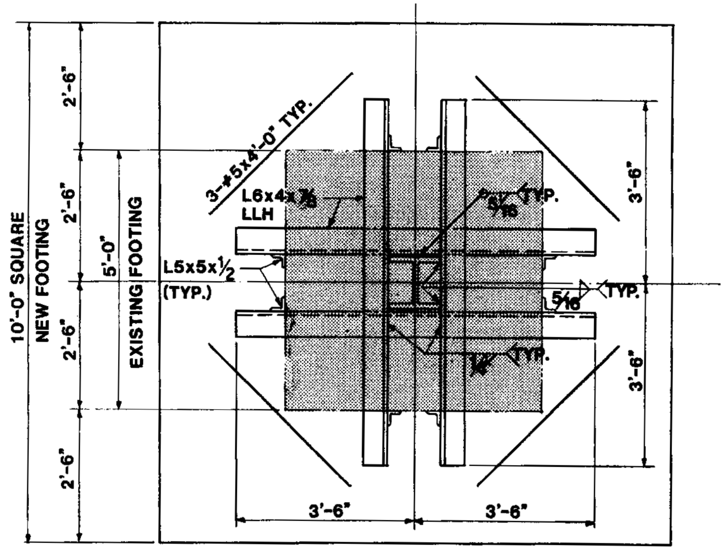


FIGURE 8.13 Increased footing size to accommodate live loads.

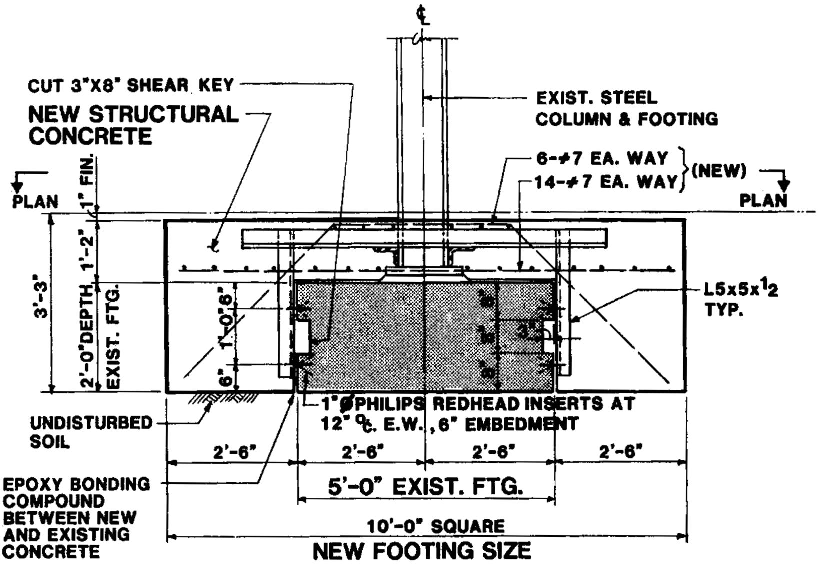


FIGURE 8.14 Cross-sectional view of Figure 8.13.

8.15). Construction work came to a halt, and serious consideration was given to evacuating the fully occupied apartment building.

Probes into the wall revealed the following conditions (Figure 8.16):

1. The width of the rock "sliver," of unknown strength, was approximately 14 in (355 mm). Incredibly, it was the only structural element supporting the high-rise building.
2. Even more remarkably, the footings called for in the original design for the existing apartment building were not built at all.
3. The easterly wall of the building cellar was not a standard concrete cellar wall but was actually a rock face with stucco finish.
4. The plans that had been filed with the Department of Buildings did not show the "as-built" conditions.

An emergency plan for underpinning was immediately launched. This plan included new concrete underpinning piers (braced laterally) cut into the rock "sliver" and bearing on the rock below the existing cellar level (Figure 8.17). The plan was executed perfectly, and the alarm was turned off.

As usual, the responsibility for the unexpected additional cost was in dispute. The questions asked by everybody were:

1. Was the original 1964 construction, which deviated from the design, bearing the apartment building columns on the neighbor's rock, legal?
2. Did it follow accepted good construction practice?
3. What responsibility, if any, does the Structural Engineer of Record have for the contractor's variation from the plans?

Regarding the first question, our research into the building code showed that the code was completely silent on this issue. Regardless, we found no requirement forbidding the practice. Our opinion was that it is poor construction practice for one building owner to utilize a neighbor's rock for

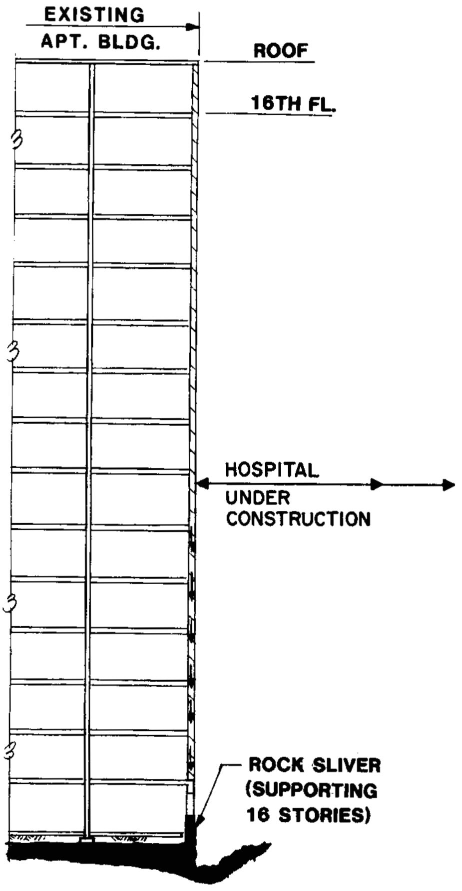


FIGURE 8.15 Rock sliver precariously supports 16-story brick wall.

bearing for the owner's building, especially since he or she has no control over nor any right to limit the neighbor's construction activity.

The Engineer of Record had no "Controlled Inspection" responsibility for this project, neither was he required to perform such an inspection of the construction according to the then-pertinent code.

According to many building codes, when neighboring construction goes below the bottom of an existing footing, it is the responsibility of the neighbor doing the new construction to underpin the existing structure, when his or her footings are more than 10 ft (3.0 m) below curb level.

The New York City Building Code, for example, states[2]:

> (b) Support of adjoining structures.—
>
> (1) Excavation Depth More Than 10 Ft. (3.0 m)—When an excavation is carried to depth more than 10 ft. (3.0 m) below the legally established curb level, the person who causes such excavation to be made shall preserve and protect from injury any adjoining structures. . . .

Lessons to be learned:

1. Do not use the rock outside your property for support of your structure, regardless of the apparent immediate savings.

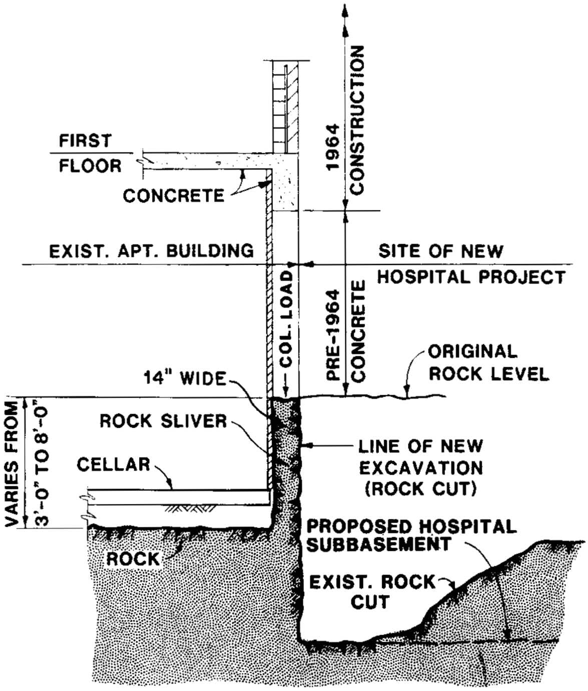


FIGURE 8.16 Cross section of Figure 7.15.

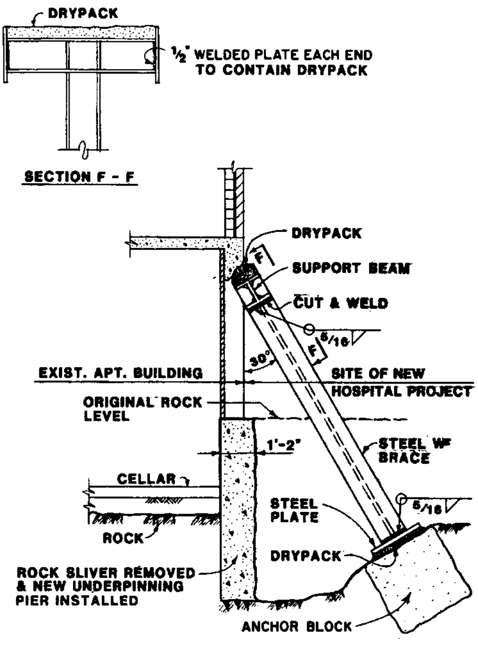


FIGURE 8.17 Emergency plan for underpinning existing building to create proper bearing.

2. You may support your structure only on the rock totally within your own side of the property line.
3. Rock "slivers" should not be relied upon for foundation support.

8.2.4 Manhattan Hospital Complex, New York City

When the structures adjacent to a deep excavation begin cracking, it slowly starts to sink in that something is wrong. An excavation extending an entire site was progressing toward a total depth of 45 ft (14 m) below grade. The existing structures contained only single-level basements. Initially, it was expected that much of the excavation would be into sound rock, which was identified as gneiss. The ground-water level within the buildings was controlled by a sump pump located below an existing basement slab.

During the design phase it was obvious that the adjacent structures would have to be underpinned. The New York City Building Code requires that underpinning will be designed by a professional engineer. This requirement was met, and a design using underpinning pits with post-tensioning cables to resist lateral earth pressures was devised.

As excavation for the pits was proceeding, movement was detected in the building directly east of the site. This movement was recorded by the tell-tales (movement gages) which were properly installed for monitoring such movements. As water was found several feet below the pits, attempts were made to dewater the soil surrounding the pit excavation. But, as recorded by the inspection engineer, ". . . this proved futile because of the impermeable nature of the soil existing at this location."

Dewatering by well points was then tried. This also proved ineffective, because the silts in the soil clogged the well point screens. A third alternative using jack-piles was then considered. This method of pushing pipe segments into the ground requires the use of the building weight to act as a reaction for the jacking forces. In this project, unfortunately, it readily became apparent that the footings of the existing building, which consisted of decomposed rubble, did not offer sufficient resistance to the jacking loads.

Even an attempt to use a steel beam to jack against failed, and the scheme was then abandoned. While these efforts were going on, a further adverse movement was detected in the adjacent building. At this time the safety of the structure was questioned, and temporary rakers (sloped braces) were installed. For all practical purposes, construction operations came to a stop.

The two buildings on the east exhibited serious cracking throughout (Figures 8.18 and 8.19). The one nearest the excavation settled, causing the entire building to pivot about the foundation walls. This pivoting effect in turn caused the entire party wall to move westward, leaving a 6 in (152 mm) gap at the top (Figure 8.20). This movement caused enormous distress on the interior of the building. Floors settled, ceilings and walls cracked, and even the elevator got stuck as its shaft deformed and would not permit free movement of the cab. At the fourth floor there was actually a separation between the wood joists and the party wall, causing the floor to settle by as much as $1\frac{1}{8}$ in (28 mm) at one point.

The foundation contractor, realizing the danger, immediately installed vertical shores under the west end of the wood joists, from the first floor all the way up to the roof.

It was at this point that our firm was retained. Subsequently it was recommended that the contractor take weekly movement readings using surveying instruments and continue doing so until the settlement stopped. The contractor was also instructed not to proceed with any restoration work until all settlement had stopped. *The New York Times* of July 3,1977, printed the following article regarding this project under the headline "When the Earth Opens and Walls Move[3]":

> George N. and Barbara K. don't live here any more. Their former apartment, in a brownstone in the Gramercy Park area, once looked like the home of many a professional couple—antique furniture, luxuriant plants and exposed wooden beams, lots of beams.
>
> . . . Their apartment is one of the occasional casualties of excavation work, the bulldoz-

FIGURE 8.18 Damage resulting from excavation.

FIGURE 8.19 Cracking of partition wall and settlement of stairs due to footing movement.

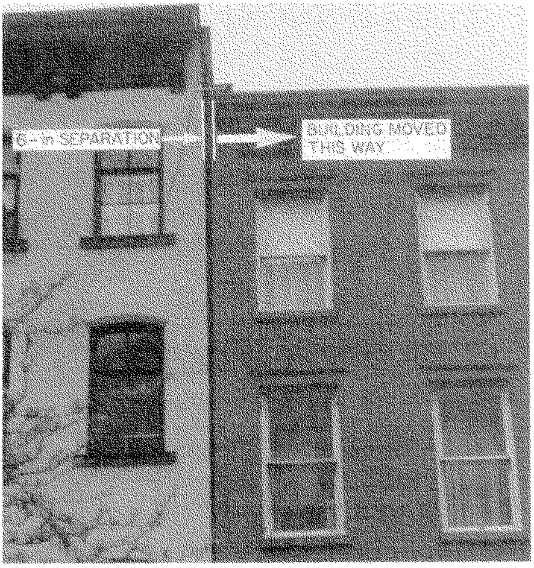


FIGURE 8.20 Lateral displacement of existing building toward excavation.

> ing, blasting and drilling that create the cavernous holes meant to hold a new building's foundation.
>
> Though New Yorkers frequently sue or arrange out-of-court settlements with excavators (claims range from apartment damage to sexual inadequacy caused by noise), George and Barbara just decided not to renew their lease.
>
> . . . The whole experience was very unnerving, "Mr. N. recalls. "I was really concerned how much physical damage would be done to my home by the end of a day. It started out as a hair-line crack and got bigger every day (author's italics). I saw the baseboard move further away from its original place on the floor. Then we had to take down a shelf in the closet because it was no longer wide enough. *The wall had moved four inches (102 mm) toward New Jersey.*" (author's italics).
>
> . . . A side of the ceiling was now without support and, just as Mr. N. had feared, it soon gave way. Fortunately, their apartment was saved by beams, which had been installed by workmen a few days earlier. [The vertical shores referred to above (author's comment).]
>
> . . . Dr. F., a physician, and her husband, resident-owners of the building next to the excavation, never contemplated selling their property.
>
> Although the roar of the machinery can reduce telephone conversations to the F. residence to frustrating shouts, Dr. F. says that somehow her practice at home (psychiatry) "has not been disturbed."
>
> . . . Mr. R., project engineer for the excavation company, says that the unknown factor in that site was an unexpectedly high underground water elevation. . . . "We had what you call test borings. . . . As it turned out, these test borings were greatly different from what we encountered."

When all efforts to support the building failed, the method of last resort was used, very costly soil solidification by freezing the soil. The saturated soil proved to be an advantage for the freezing process. This process with all its pipes and equipment took about 3 weeks to install (Figure 8.21). It

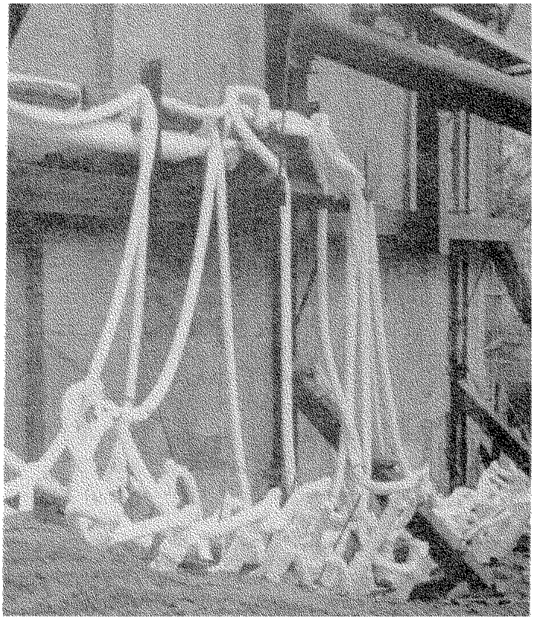


FIGURE 8.21 Soil solidification by freezing.

proved to be a total success, as practically all movements within the adjacent "ailing" structure stopped. Once the soil under the building was consolidated, the original conventional underpinning design was implemented. Just prior to freezing the soil, a cross-lot bracing system was installed (Figure 8.22). The purpose of this system was to eliminate any further movement in the building on the east and to stabilize it during the subsequent construction operations. The giant cross-lot braces that spanned the entire lot permitted construction to proceed without interruption. After completion of the underpinning, the adjacent buildings were restored.

FIGURE 8.22 Cross-view of Figure 8.21.

Lessons to be learned:

1. Accurate preconstruction subsoil study is essential to the success of foundation work. Inadequate test data can cause serious delays, cost overruns, damage, and even collapses. In this case, had accurate information been obtained, design changes and soil solidification could have been planned prior to start of construction.
2. Adjacent structures must be carefully monitored during underpinning operations, so if and when movements are detected, changes in methods or procedures can be implemented.

8.3 LOAD-TRANSFER FAILURE

A well-designed and constructed rigid-frame structure will tolerate even substantial foundation movements. When an assembly of walls, floors, frame, and partitions is rigidly connected, the system will adequately adjust itself to differential foundation movements. The load transfer is made by frame action through the support offered by the foundation. When this interconnected rigidity is absent, the load transfer will be through a single support so that the load will go directly to the soil vertically. Where this single support in the soil is missing, the structure will fail, unless the structural rigidity will transfer it horizontally to other footings and then, in turn, to the soil.

Where the adjacent rigidity is present but lacks sufficient strength, the adjacent structure will fail. Figure 8.23 shows a diagrammatic description of this action. Four men are carrying a log, its weight uniformly distributed. When A steps into a ditch (an inadequate foundation), his portion of the load is suddenly transferred to B, who may be unable to support the additional load.

One such typical example was a 16-story warehouse building in lower Manhattan, supported by four continuous pile caps and timber piles. Pumping operations at a neighboring 20 ft (6 m) deep excavation for a depressed roadway lowered the water table and caused one line of piles to settle and pull the pile cap down.

This released the exterior support so that the load transfer to the interior line of columns overloaded their steel beam grillages. The 24 in (610 mm) beams collapsed down to a 12 in (305 mm) depth with consequent 12 in (305 mm) settlement of every floor in the building. The floor beams

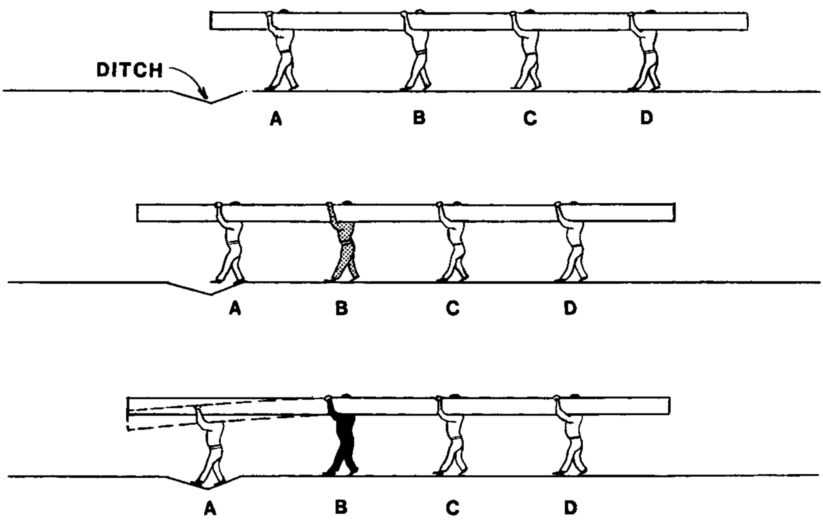


FIGURE 8.23 Diagram of log deflection.

resting on steel girders between columns had only "soft" support so that they took the new trough shape without failure.

The entire building was shored, and new shallow grillages were installed to replace the failed ones. Rather than jacking the entire structure, which entailed definite risks, especially for an old structure, each floor was releveled, and the building was put back into use.

8.4 LATERAL MOVEMENT

It is well known that 1 inch (25 mm) of lateral movement of a foundation causes more damage than 1 in (25 mm) of vertical settlement. Lateral movements are caused from either the elimination of existing lateral resistances or from the addition of active lateral pressures and loads. The changes in the active pressures and passive resistances as a result of variable water conditions must be considered. Saturation of the soil often increases the active pressures and reduces the passive resistances. Many collapses have occurred as a result of demolition of adjacent buildings. In these instances the backfilling of adjoining cellar space generated lateral soil pressures on the old cellar partitions. These walls, in turn, failed because they were not designed and constructed as retaining walls.

Lateral flow of soil under buildings is also known to cause collapses of buildings. In Sao Paulo, Brazil, such lateral soil flow caused the total collapse of a 24-story office building in 1943.

A similar case occurred in 1957 in Rio de Janeiro, also in Brazil, where an 11-story residential building collapsed completely after the excavation of an adjoining wall removed the lateral restraint of the building piers. The attempted underpinning, which was started too late, could not save the rotating structure.

There also are numerous cases of wall failures from broken drains alongside the footings with washout of the soil during heavy storms.

Change in pressure intensity against walls often causes failure, especially in the unreinforced concrete basement walls for residential buildings. These walls are unfortunately seldom investigated for high soil pressures. Surcharging soil on land adjacent to structures often causes large lateral pressures. Mounds of debris from demolitions are frequently piled adjacent to basement walls that had not been designed to resist such loads. These bowed basement walls often cave in and cause the total collapse of the structure.

The stability of retaining walls is often similarly affected, resulting in failure by overturning, either because of inadequate base width or because of high soil pressure caused by defective drainage or adjacent load surcharge.

Failure of buried sanitary structures are also common, especially when they are emptied for cleaning or repairs.

8.4.1 New Jersey Postal Facility—Cracked Pile Caps

Shortly after completion of the construction of this mail bag repair shop, the entire structure started to shift sideways, causing many of the piles to bend and the pile caps to crack (Figure 8.24). In addition, an interior block wall parallel to the direction of the shift also cracked (Figure 8.25).

Our firm was called to investigate and to determine the structural adequacy of the entire facility. What was immediately evident was the very heavy vertical gravity loads imposed on the piles. Figure 8.26 shows heavy bales of paper and cloth sitting directly on the already shifted and distressed piles.

The area directly east of the plant was found to be surcharged with mounds of heavy garbage dumping (Figure 8.27). It quickly became clear that the soft clay soil directly underneath this extreme garbage weight was surcharging the soil, causing lateral pressures on the long, unbraced steel piles. Previous attempts to "hold the line" by welding steel cross-ties had been unsuccessful, and rather than stopping the lateral shift, they only transferred the lateral loads to the adjacent piles, which eventually failed, too. The additional lateral cross ties may be seen in Figure 8.28.

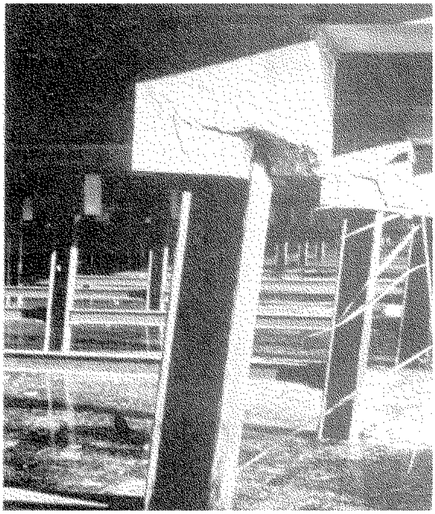


FIGURE 8.24 Lateral failure of H piles and concrete caps.

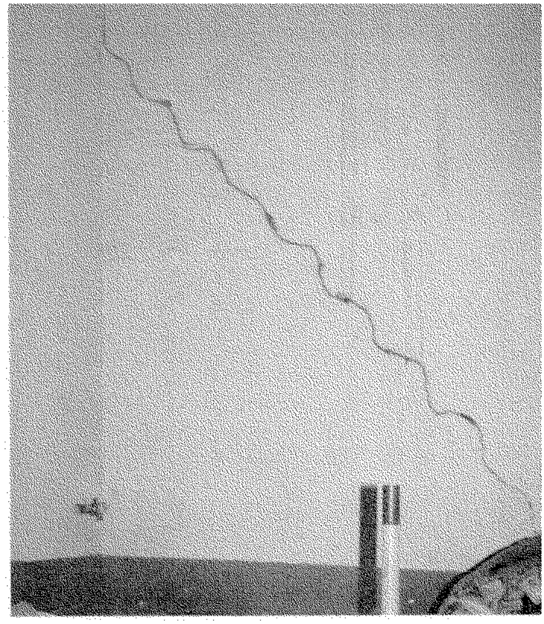


FIGURE 8.25 Diagonal masonry cracks due to lateral and downward settlement.

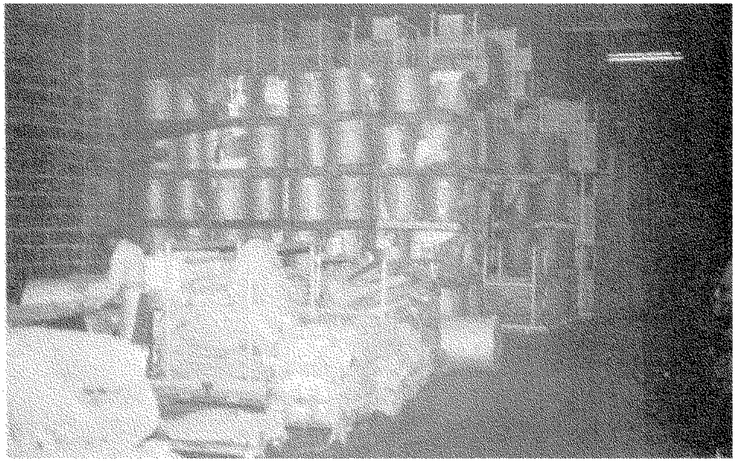


FIGURE 8.26 Unsafe loading on damaged piles (Figure 8.24).

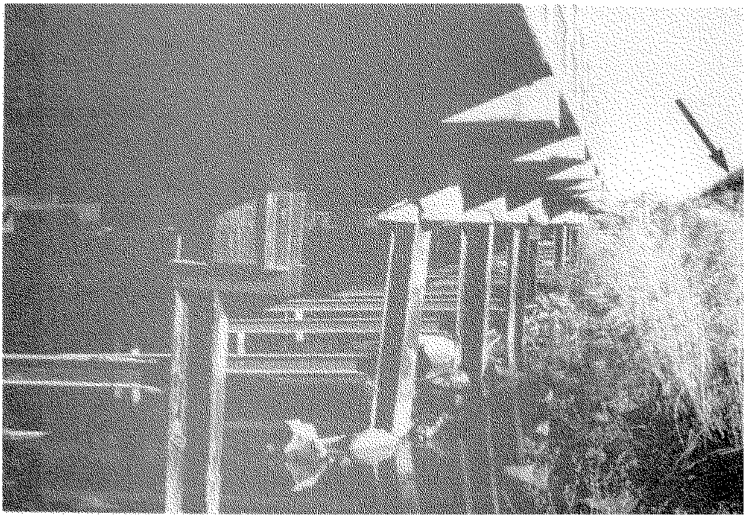


FIGURE 8.27 Lateral soil shift causes pile failure.

We immediately recommended that the owners reduce the actual live loads on the piles, by moving the heavy bales to unaffected areas. A complete underpinning program involving drilling through the existing slabs and jackpiling down through the soil, was then recommended. The owners dragged their feet for a while, but finally a program similar to one proposed by our firm was implemented several years later.

Lesson to be learned:

The placement of high surcharge adjacent to structures generates high lateral loads that can cause shifting of piles and bowing of foundation walls, and should be avoided.

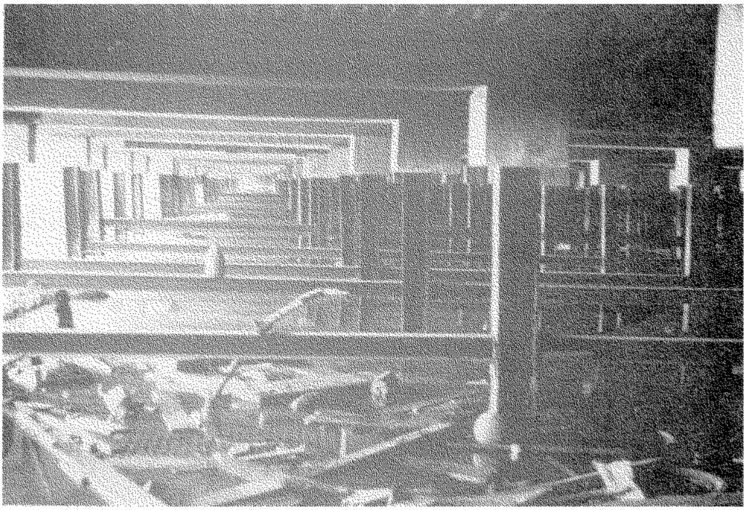


FIGURE 8.28 Steel cross-ties stabilize piles.

8.4.2 The Collapse of the Hartford, Connecticut, Foundation Walls

This large housing development is located on a sloping terrain, using concrete foundation walls enclosing the basements. The walls were unreinforced, common for low-rise residential buildings. Rather than using controlled granular fill, the contractor used clay backfill during a dry period of weather. This clay backfill shrank away from the concrete walls, leaving narrow gaps along the full height of the walls. After a heavy rainstorm, the runoff filled these gaps, causing very high hydrostatic lateral pressures. Some of the walls which were not yet braced at the top toppled over, and some bent, cracked, and completely failed [Figures 8.29(a), 8.29(b), and 8.30].
Lessons to be learned:

1. Foundation walls must be braced at top and bottom prior to any backfilling.
2. Only well-controlled and drainable, porous backfill should be placed against foundations and other retaining walls to avoid shrinking soils and subsequent development of high lateral soil and water pressures.

8.4.3 Long Island Water Pollution Control Plant Outfall

During 1972 and 1973, precast concrete sections were installed for the Long Island Water Pollution Control Plant Outfall. Later, in April of 1974, during a dye test conducted to check the continuity of the ocean outfall, it was discovered that there were open joints along the bay length. Further investigation disclosed that 2300 ft (710 m) of pipe sections out of the overall length of 22,000 ft (6706 m) were either displaced, damaged, or completely separated.

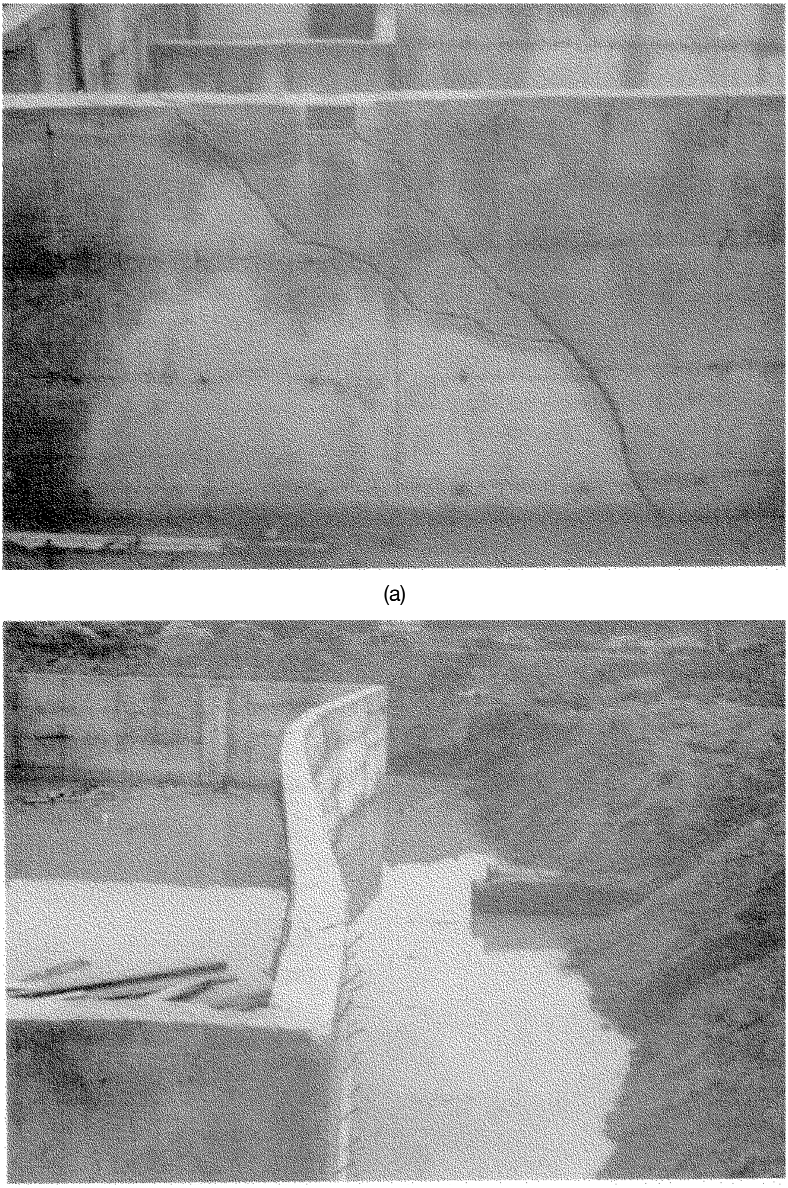
The outfall consisted of an 84 in diameter, prestressed concrete pipeline pipe sections, 20 ft (6.1 m) long, which were generally assembled into 100 ft (30.5 m) strings and installed on pile supports spaced at intervals of 20 ft (6.1 m). The outfall consisted of three sections: a 17,000 ft (5182 m) bay section which was essentially horizontal; a 5000 ft (1524 m) section sloping at a 3% grade; and the remaining horizontal section. The failure occurred in the sloping portion, which deformed into a lateral S-shaped form.

This S shape was consistent with a buckling failure of a compression member under high stress. These types of failures may occur under several basic conditions:

1. Structural continuity
2. Axial compressive force
3. End restraints
4. A slight eccentricity with/or a lateral load adequate to trigger the initiation of the failure

Our review of all the data indicated that all the above-listed conditions were met in the summer of 1973.

1. The pipeline was completely closed, and all the sections were assembled and laid into the water under winter conditions.
2. The change in water temperature within and without the pipeline, which was estimated to be at least 30°F (–1°C) between February and August, induced the expansion of the concrete pipe.
3. End restraint was provided by the pile caps.
4. A small lateral soil pressure was considered to be the trigger most likely to have started the movement. The S bending shape was governed by lateral loading conditions. Movement downward was resisted by the pile support, and movement upward was counteracted by gravitational force. Therefore lateral movement was met with the least resistance. Additionally, uneven backfill will result in horizontal pressure component resulting in an unbalance sufficient to trigger the movement.



(a)

(b)

FIGURE 8.29 Total collapse of foundation walls [Figure 8.29(a) and (b)].

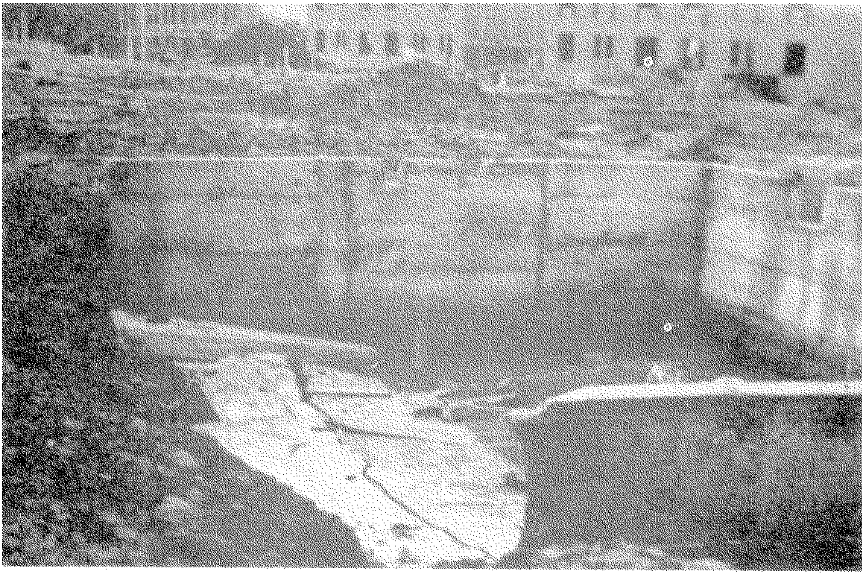


FIGURE 8.30 Total collapse of foundation walls [Figure 8.29(a) and (b)].

Lessons to be learned.

1. Special care should be taken to provide balanced lateral pressure by providing continuous quality control of backfill operations at underwater structures.
2. Although underwater structures are subjected to a rather small temperature differential, some relief must be provided to permit linear expansions.

8.5 UNEQUAL SUPPORT

A basic rule of engineering is that there is no load transfer without deformation. When loads are transferred to the soil through the foundation, this transfer is associated with a deformation of the soil. In other words, all footings settle when they are loaded. The amount of settlement is equal for different footings when soil resistances are identical and load distributions are equal. When they are not, differential settlements occur, and load transfer or tipping of the structure will result. The portion of the structure founded on the weaker soil will tip away. Where the framework is not continuous, the brittle masonry enclosure will crack (usually in a diagonal pattern) during the shear transfer. There are a great number of such structures, with foundations partly on rock and partly on soil, partly on rock and partly on friction piles, partly on stiff soil and partly on soft soil. There are no doubt many structures with footings bearing on different soils with different and unequal soil bearing resistances. The results are invariably distress and severe cracking (Figure 8.31).

All these soil support deficiencies may be corrected and are often so rectified. However, these corrections are usually quite costly and involve underpinning the weaker soil or the weaker support. The use of additional piles or the installation of jack piles are some of the most commonly used and most successful methods. The famous Transcona grain elevator in Manitoba, Canada (Figure 8.32) showed unequal settlement and tipped. This structure consisting of 65 concrete bins 93 ft (28 m) high on a concrete mat 77 × 175 ft (24 ×53 m), settled 12 in (305 mm) and then tipped 27° when

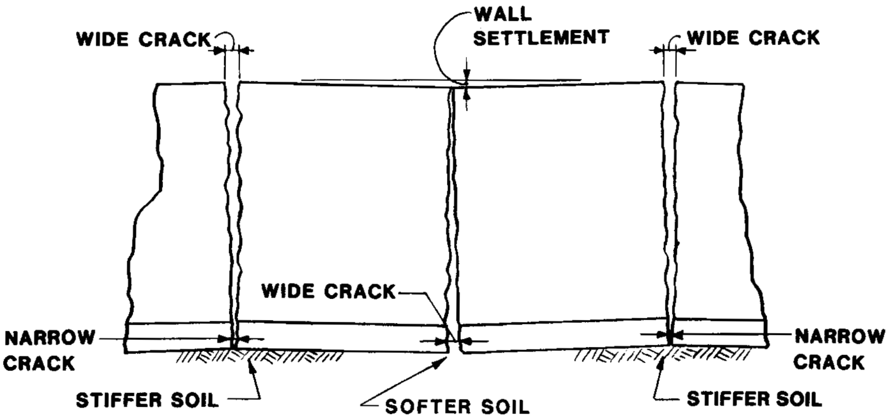


FIGURE 8.31 Cracks indicate rotation from vertical (heave) or settlement movement.

about 85% full of grain. The structure weighed 20,000 tons (178 MN) and contained at the time grain weighing 22,000 tons (195 MN). Already when repair work was begun in 1914 the massive structure, which was 34 ft (10 m) out of level, was jacked back to use with underpinning to rock—a remarkable engineering feat.

Sometimes, other methods of soil stabilization by cement or chemical injection are used. One other correction procedure utilizes the addition of a subsurface enclosure (usually a tight sheetpile cell) that increases the bearing value of the soil. This was done on a grain elevator in Portland, Ore-

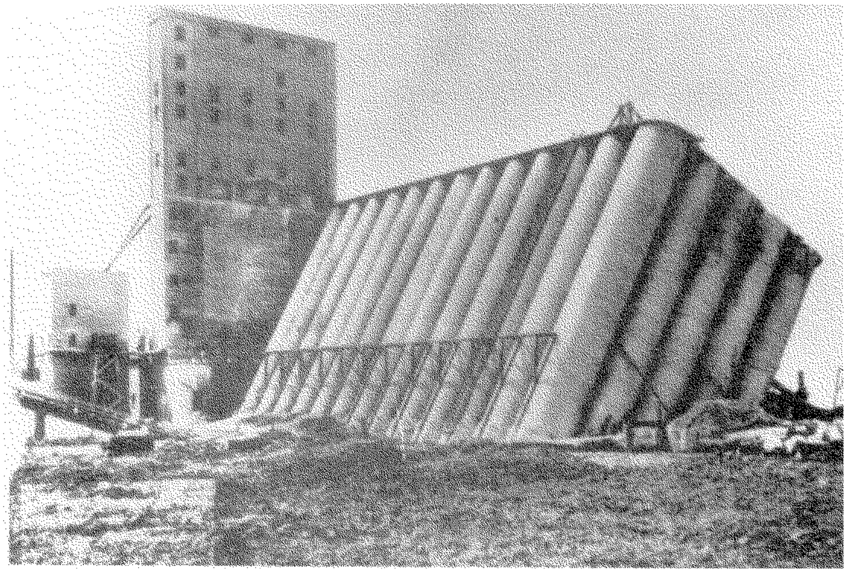


FIGURE 8.32 Tilt in Canadian grain elevator.

gon, in 1919, where 2600 pinch piles were added to enclose the foundations after the structure showed progressive unequal settlement and was tipping.

Not so fortunate was a smaller grain elevator in Fargo, North Dakota, which tilted in 1955 when it was only 1 year old. This elevator, which contained 20 bins 120 ft (37 m) high, was supported on a concrete mat measuring 52 × 216 feet (16 × 66 m). As the mat tilted, the bins crumbled and the structure was wrecked and had to be abandoned.

There are many classical examples of tilting buildings as a result of unequal settlements. The Tower of Pisa is probably the best known and researched example and is still in use, as is the Palace of Fine Arts in Mexico City (Figure 8.33) with its settlements measured in meters. The Guadalupe Shrine and Cathedral, also in Mexico City (Figure 8.34), is so distressed, cracked, and tilted that it makes visiting tourists nervous.

Buildings partially on earth and partially on rock have too often been designed on the incorrect assumption that the mere use of local building code allowable bearing values will ensure equal settlement. Where such variable bearing conditions exist, a separation joint must be provided so that each section of the building can then act as an individual separate structure. These buildings, especially when they utilize bearing walls, often crack almost immediately where such a joint is missing. Thus, nature provides the omitted joint (Figure 8.35). Underpinning and other repair costs together with the additional economic loss caused by delayed or vacated use far exceed the cost of properly designed and constructed separated foundations.

Our firm has also investigated several instances where soil shrinkage due to local dewatering triggered differential settlements even though the original construction was proper and followed faultless design.

8.6 DRAG-DOWN AND HEAVE

As a footing is loaded, the supporting soil reacts by yielding and compressing to provide resistance. The actual compressing of the soil takes place rapidly in the case of granular soils but much more

FIGURE 8.33 Settlement of the Palace of Fine Arts, Mexico City.

FIGURE 8.34 Foundation failure of Guadalupe shrine.

slowly for clays. Once this compression occurs, the structure remains stable, since the foundation no longer settles. This stability depends on fringe areas as well as the soil directly below the footing, or, in the case of piles, on the soil near the pile tip. If the soil below the footing is removed or disturbed, settlement or lateral movement is induced. Similarly, if the fringe area is removed, the soil reaction pattern must change. Even if sufficient resistance still remains without measurable additional settlement, the center of the resistance is changed, thus affecting the stability of the footing. When the

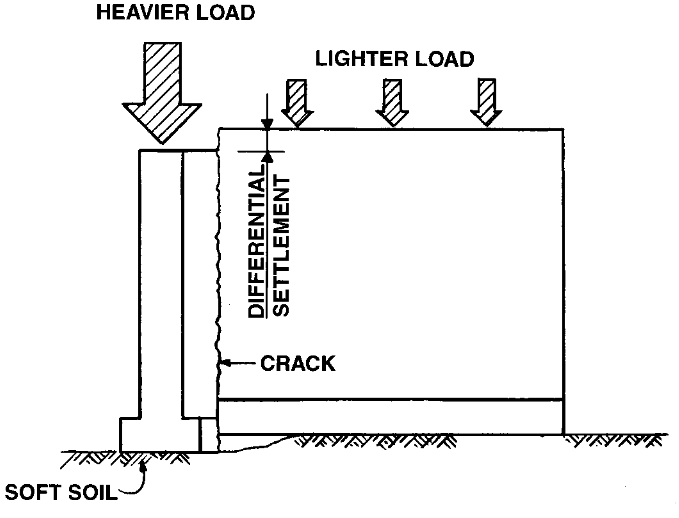


FIGURE 8.35 Differential settlement.

fringe area is loaded by a new structure, causing new compression in the affected soil volume, the old area can be required to carry some of the new load by internal shear strength. In such a case, there will be an additional unexpected new settlement of the previously stable building. In the event the new building is not separated from the existing construction, the settlement caused by the new load will cause a partial load transfer to the existing wall, with possible overloading of the previously stable footing.

In plastic soils, these new settlements often are accompanied by upward movements and heaves (Figure 8.36) some distance away. The liquid in the soils cannot change volume, and every new settlement must produce an equal volume heave.

When an entire structure settles at a slow rate for a long period, these settlements may be acceptable as long as they are uniform. Large differential settlements will result in damage.

To disregard such elementary considerations is often the cause of much litigation stemming from cases where new foundation construction in urban areas suddenly places neighboring existing construction in danger of serious damage or even complete failure. Under several building codes (The New York City Building Code is one such example), if new construction requires excavation more than 10 ft (3.0 m) in depth adjacent to an existing foundation, all precautions to avoid damage to the existing conditions are the obligation of the new project. If the new excavation depth is 10 ft (3.0 m) or less, the obligation for protection falls on the owner of the existing building should its foundations be higher than the new footings.

In medium- to well-compacted sands, pile driving demands heavy power, and the vibration impulses can shake adjacent structures to the point of serious damage.

Drag-down from soil shrinkage when water tables recede or from desiccation by tree growth, with consequent differential settlements, is a phenomenon observed in many localities. The failure of a theater wall in London in 1942 was correlated with the growth of a line of poplar trees. In Kansas City, Missouri, 65% of the homes in one residential area were found to be affected by soil desiccation from vegetation growth in foundation plantings.

Piles embedded in soil layers that will consolidate from dewatering or from load surcharge are subject to overloading when the greater soil density increases surface friction and the new soil loads "hang" on the piles. This phenomenon is sometimes referred to as negative friction. The added load may cause considerable increase in settlement and may even pull the pile out of the pile cap or pull the pile cap free of the column or wall.

Dewatering operations are commonly caused by new construction or pumping by water supply companies.

In some instances, as consolidation of organic clay strata takes place, piles continue to sink with additional settlement of the structure. This was the case at the Queens Apartments in New York.

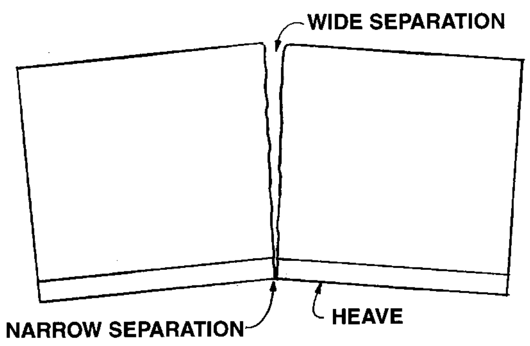


FIGURE 8.36 Upheaval.

8.6.1 Queens Apartments, New York

Settlements and masonry cracks developed in many levels of the buildings of this project (Figure 8.37). The cracks appeared in both interior partitions and the exterior brick walls.

These structures, located in Flushing, New York, were constructed over marsh land and were supported by timber piles. An earlier subsurface investigation of this project indicated that it was likely that at least some of the piles in the three-pile group supporting the distressed corner of the building had broken. This fact could not be confirmed without excavation or probes. The breakage may have occurred when the piles were driven or subsequently when the loads resulting from consolidation of the organic clay stratum were imposed on them.

In an effort to assess the potential for additional settlement and their likely magnitude, a laboratory and a field program were performed. In the laboratory, the rate of settlement due to a secondary compression of samples of the organic clay was measured. In the field, a sensitive settlement plate (Figure 8.38) was monitored for movement every 2 weeks for about 6 months, using a depth caliper accurate to 0.001 in (0.025 mm).

The area selected for installation of the plate was the interior corner directly outside of the laundry room. This area was chosen due to its close proximity to settlement cracks; it also provided sufficient overhead clearance for drilling.

A 1 in (25 mm) diameter heavy-duty pipe was driven into the lower sand stratum to provide a fixed reference point. A 2 in (51 mm) diameter outer casing protected this pipe from down-drag effects from the overlying fill and clay.

Analysis of laboratory data has suggested that settlement of the organic clay, as a result of the fill above it, can be expected to be about 1 in (25 mm) every 10 years. The field measurements from the settlement plate indicated even greater settlement.

Figure 8.39 presents the data obtained from the settlement plate device, with settlement rates of up to about ¼ in (6.4 mm) per year. Thus, the field data suggested settlements about twice those indicated by the laboratory data.

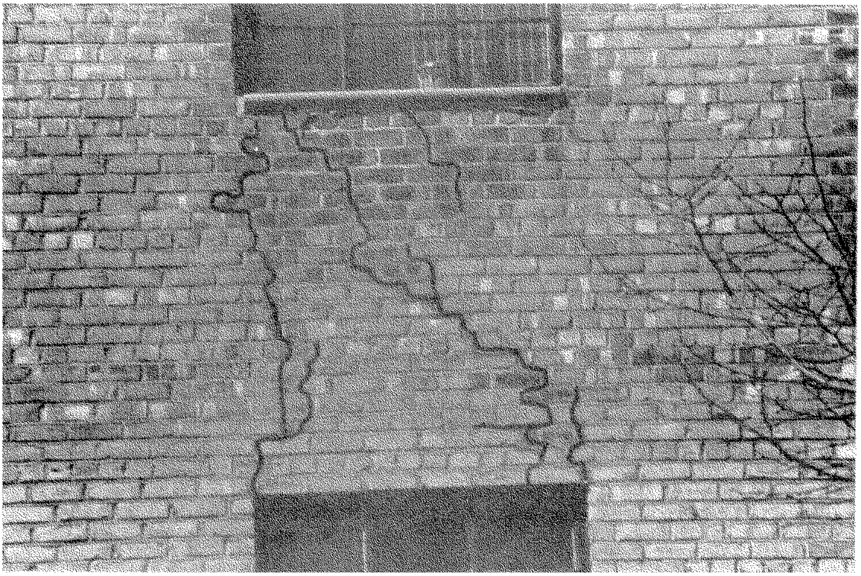


FIGURE 8.37 Masonry cracks due to settlement.

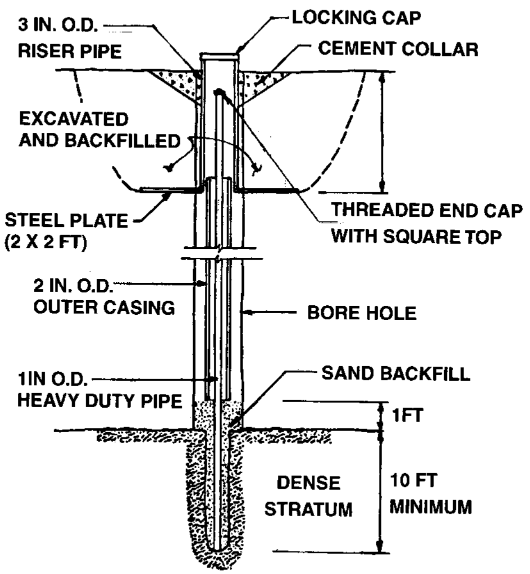


FIGURE 8.38 Instrumentation for measuring soil settlement.

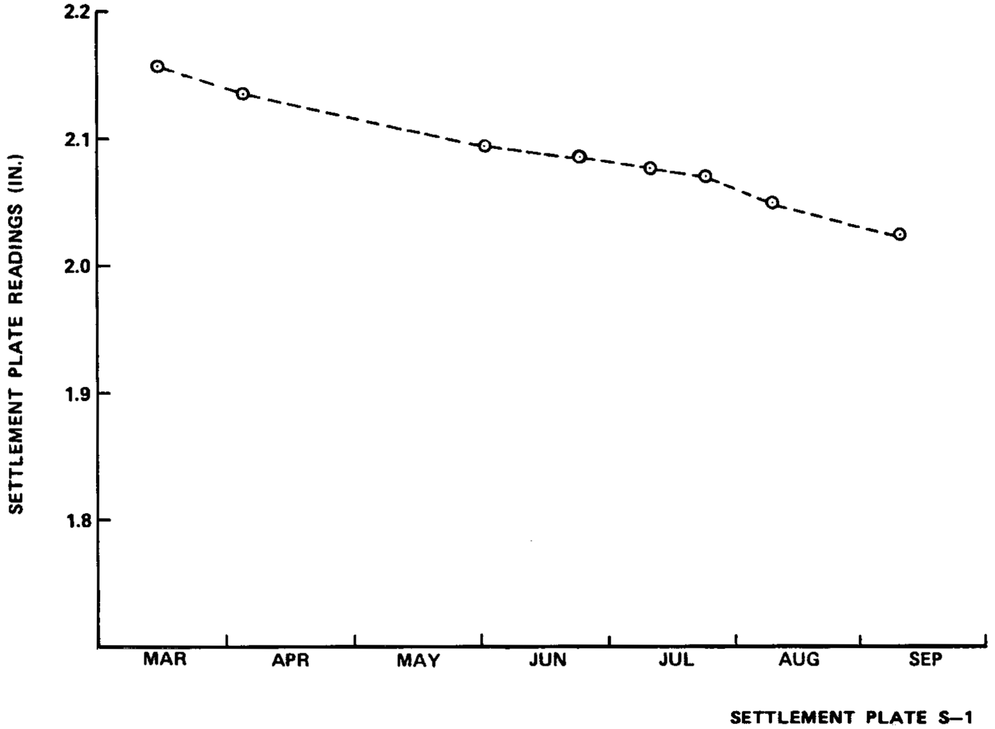


FIGURE 8.39 Chart of measured settlement.

Lessons to be learned:

1. Do not guess whether a structure that has settled will continue to do so. Instrument the foundations and the soil to assess potential future settlements.
2. Placing a structure on piles bearing in marsh land always involves risks of settlements.

8.6.2 Edgewater, New Jersey, High Rise

The piles supporting the first-floor slabs of this complex settled and pulled the slab downward. This caused settlements and cracks of the slabs and tilting and cracking of partitions (Figure 8.40). The first signs of distress were "sticking" doors and windows.

An in-depth structural investigation was carried out, revealing that although the main supports for the columns were bearing on steel piles driven to resistance in rock, the piles supporting the slabs were only wooden friction piles. The pattern of slab deflection was confirmed by instrument survey and the corresponding contour map (Figure 8.41).

Probes below the slab showed that the backfill had settled approximately 3 in (76 mm) (Figure 8.42). The negative friction effect of the downward settlement of the backfill and consolidation of the soil pulled the piles down. Consequently, the concrete slab attached at the top of the piles was dragged down.

The repair required cutting into existing slabs and adding new reinforced-concrete beams that transferred the slab loads into the reliable column piles (Figure 8.43).

Lesson to be learned:

When piles are driven into compressible soils, negative friction effects should be taken into consideration. Additional soil surcharge will increase the drag-down forces.

Heaving of footing contact surface is known to neutralize soil bearing by frost action. We have investigated several cases where during extremely cold winter, frost penetrated below slabs on grade

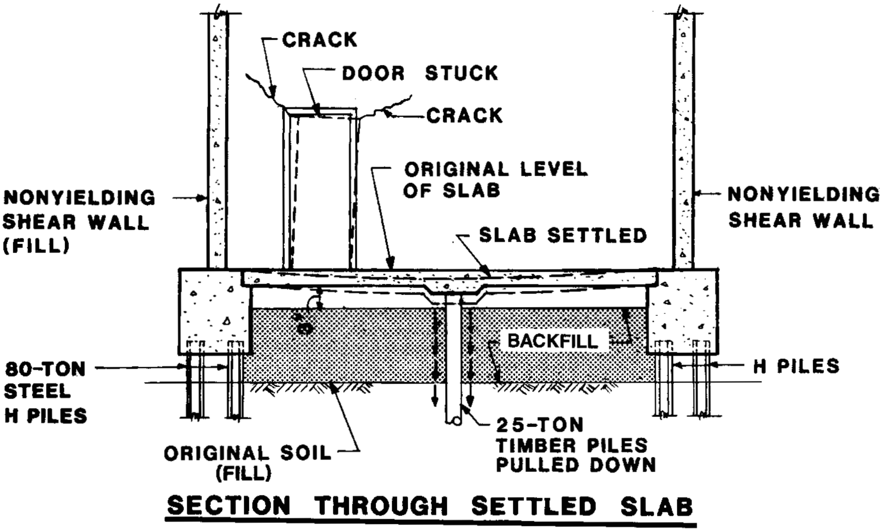


FIGURE 8.40 Structural distress relative to differential slab deflections.

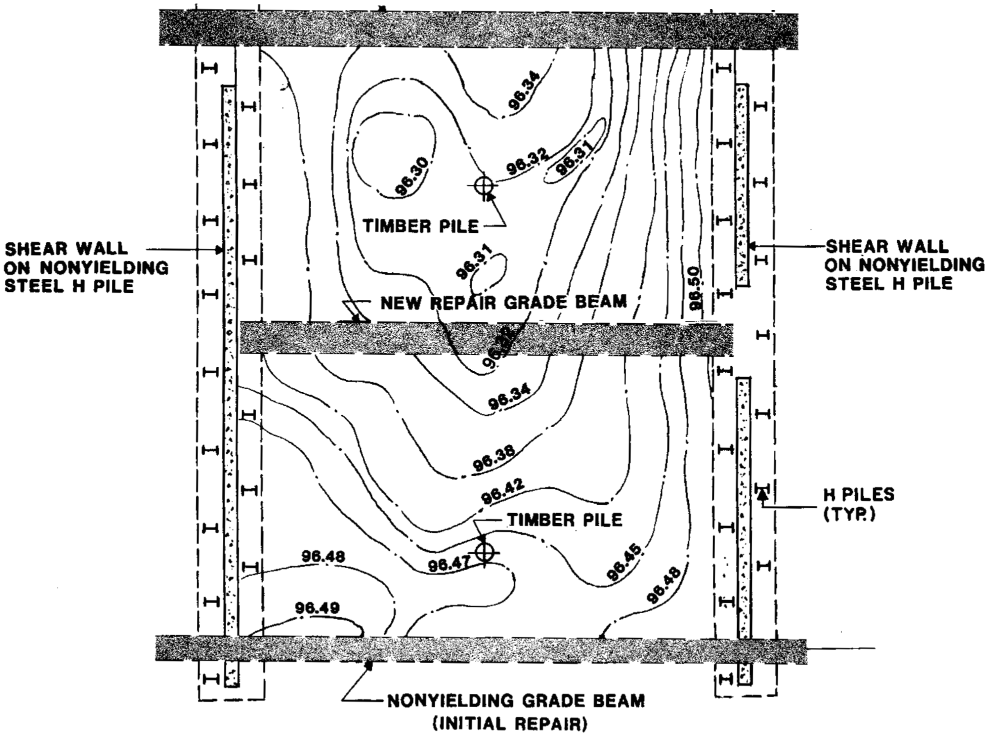


FIGURE 8.41 Slab elevations suggesting settlement of up to 2½ in (64 mm).

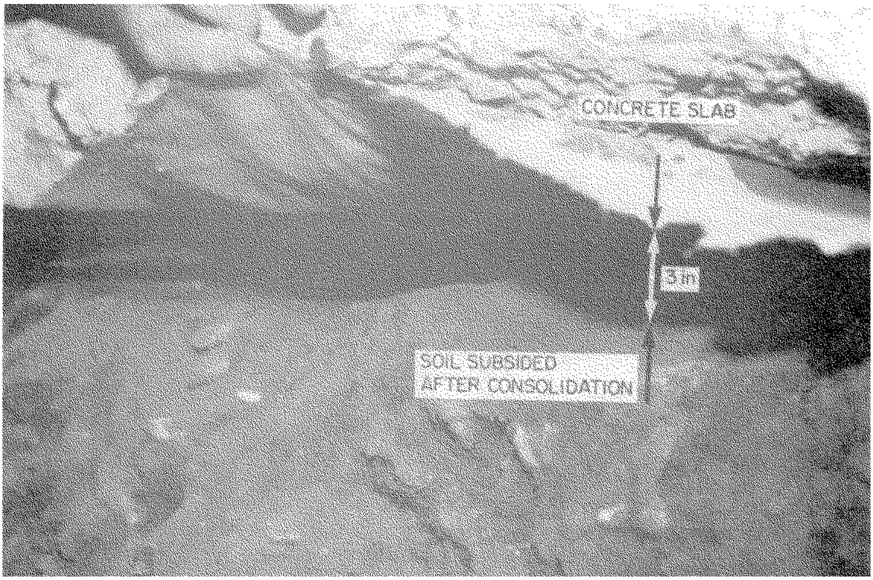


FIGURE 8.42 Settlement of fill leaving 3 in (76 mm) void.

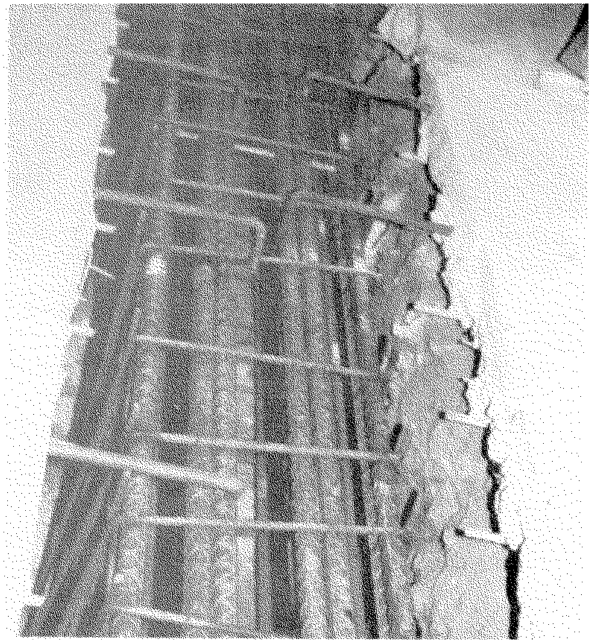


FIGURE 8.43 New concrete beams shift slab loads to steel plates.

and cracked buried water and sprinkler lines, causing ice buildup which heaved the structure above and even sheared some of the supporting columns.

8.7 DESIGN ERROR

Unfortunately, many foundations are designed with insufficient prior subsurface investigation or with disregard of the true soil conditions. This results in inadequate support for the structure, often requiring later expensive corrective work.

Standard structural design procedures for foundations will result in an adequate factor of safety; most errors are in judgment, leading to assumptions or decisions that are not consistent with the actual behavior of the soil later.

A common design error is often made, usually in an effort to save initial construction costs. In these cases the designers provide for pile support for the walls and the roof, while the main floor is placed on "more or less" compacted sand over fills, rubbish, peals, organic silts, and other compressible layers. Our own firm has seen many industrial plants where the roof was supported on piles while the floor supporting expensive equipment was placed over soil fill. It seems more logical to support expensive equipment on piles and let the roof settle. Heavy factory machinery generally requires installation on precisely level floors, and small differential settlements can become cause for concern.

Even in warehouse use, floor deformations become objectionable, making efficient use of forklifts impractical. Placing floor slabs on inadequate soil fills obviously is a false economy. True cost efficiency is measured by the total of the initial and in-life service costs.

8.7.1 The Alaska Pier Collapse

This unusual failure was caused by a design error resulting from inadequate preinvestigation study of local data and conditions. The formation of huge ice blocks atop the pier pilings was totally unexpected. The pier designers had not taken into account loads of such magnitude being imposed on the structure. The total collapse of the pier occurred at low tide when the total weight of the ice was suddenly suspended from the sloped piles (Figure 8.44).

There were unique conditions that contributed to this collapse. These were the extremely large variations between low and high tide, which were approximately 20 ft (6.1 m). The blocks of ice that had formed during the winter suddenly lost their support when the thaw came in the spring. The enormous weight of the ice was shifted to the piles, which bent, causing fracture of the concrete pile caps. The thawing of the ice at the contact surface with the piles also caused the ice blocks to slide down diagonally while being wedged between the piles (Figure 8.45). The bending stresses at the ends caused by the wedging forces (see force and moment diagrams shown in Figures 8.46 and 8.47) severely cracked the concrete pile caps (Figure 8.48) leading to the eventual collapse of the pier.

The author's files show that the partially completed pier was supported on pile bents spaced 20 ft (6.1 m) on center. The bents consisted of both vertical and diagonal piles. Twenty-inch octagonal prestressed vertical piles and 20 in (508 mm) diameter batter steel piles were arranged in pairs forming quasi A-frames. The length of the piles was between 70 and 80 ft (21 to 24 m). The piles were driven into 15 ft (5.0 m) of silt, then 55 ft (16.8 m) of silty gravel and sand underlain by layers of clay. By midwinter, huge blocks of ice, approximately 20 ft thick (6.1 m), had formed on the piles. The blocks weighing 50 tons (445 kN) or more slid down the sloped piles, damaging the piles and pile caps. The recorded telephone conversation between the design engineers on the project went like this:

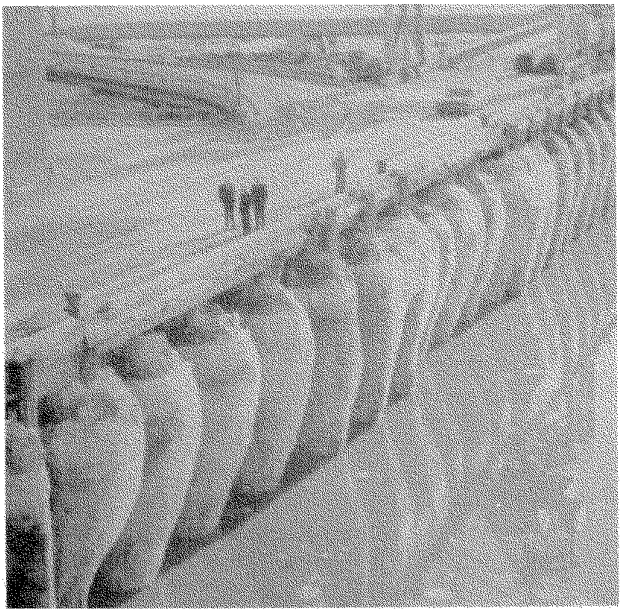


FIGURE 8.44 Ice-encrusted piles.

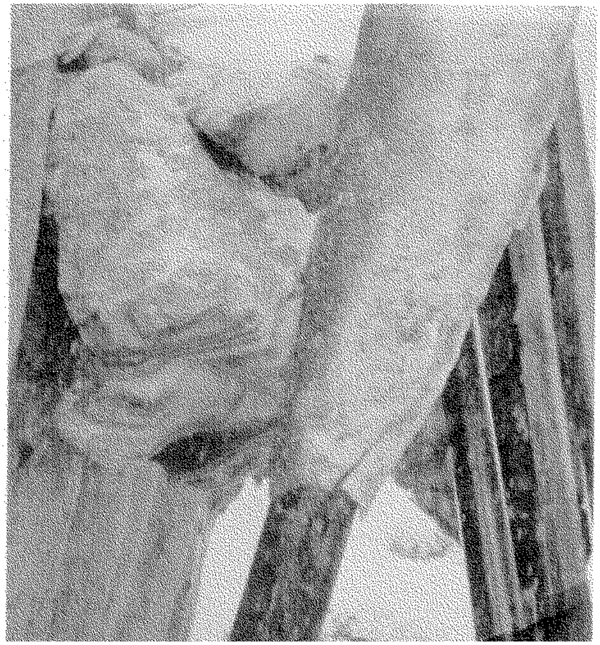


FIGURE 8.45 Ice melts, and blocks become wedged between piles (see Figure 7.45).

D: We've got great big problems up here . . . I went down the dock . . . a good half of our pile caps where the brace pile . . . are cracked.
They're cracked bad. They've got an inch and a half (38 mm) crack in some of these. We know it's because of the tidal action and the ice.

A: It looks like. the weight of the ice is causing such a moment up there that it is cracking the heck out of it. We're going to have to take the panels off, go in there and remove the caps and redesign the whole thing. . . .

D: We've got a good connection up there . . . it's very, very, rigid, but it's bending that's doing it. Would have saved us I think, if we'd had a lot more of those stirrups in there, but there are only No. 5 bars at about 18 in (457 mm) on center and it's just not enough.

While this failure was considered by many investigators to be caused principally by a design error, it was also established that damage was caused to the vertical piles during installation with overjetting and pounding with the diesel hammer. Insufficient penetration of the batter piles was also confirmed in several cases.

Lessons to be learned.

1. The effect of ice formation and resulting forces must be considered in the design of structures in northern exposures.
2. Piles subjected to lateral loading and bending must be sufficiently embedded into the concrete pile caps.
3. The consequences of extreme variations of tidal rise and fall should always be taken into consideration in design of marine structures.

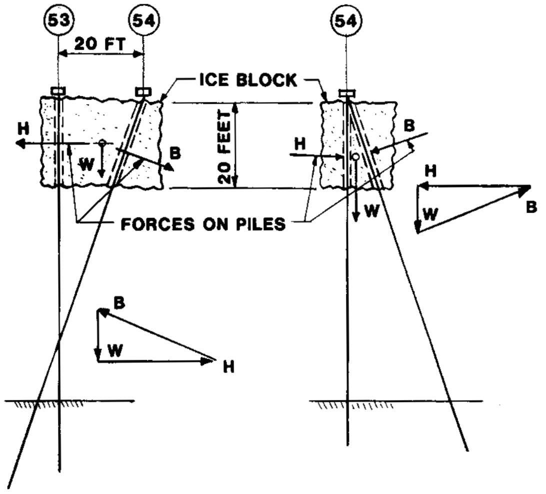


FIGURE 8.46 Lateral forces on battered and vertical piles due to ice action (see Figure 8.45).

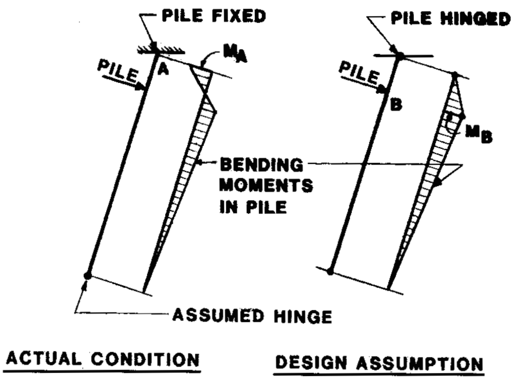


FIGURE 8.47 Lateral forces on battered piles due to ice melting (see Figure 8.46).

FIGURE 8.48 Failure of concrete pile caps due to ice action.

8.7.2 Washington, D.C., Foundation Mat

A 3 ft thick (0.92 m) foundation mat was cast with draped tendons (Figure 8.49) in accordance with the design requirements, and shortly afterward was post-tensioned. Unfortunately, as soon as the high prestress loads were applied, the slab lifted upward and cracked (Figure 8.50).

The error was discovered immediately because it was so obvious. It was amazing that the incorrect procedure eluded everybody involved with the project. The fact that the designer of the mat was a prestigious engineer only proves that no one is immune to mistakes.

As can be seen in Figure 8.51, there could be no equilibrium of forces in the foundation mat without the presence of the building columns. Realistically, the design should have planned for the prestressing to be executed in stages, with the prestress forces increasing as more and more weight of the upper levels of the structure was imposed. This procedure was not followed, and the failure ensued.

The repair was carried out using rock anchors to provide the necessary vertical reactions to the prestressing forces (Figure 8.52). The mat was repaired, and the prestressing was successful the second time around.

Lessons to be learned:

1. Posttension sequence must be carefully reviewed on a step-by-step basis.
2. Total equilibrium of forces must be ensured.

8.7.3 Boston Housing Slab Settlement

Where slabs on grade are expected to be at the same level as adjacent structurally supported slabs, total reliance on well-compacted backfill is merely wishful thinking (see Figure 8.53). Unless a de-

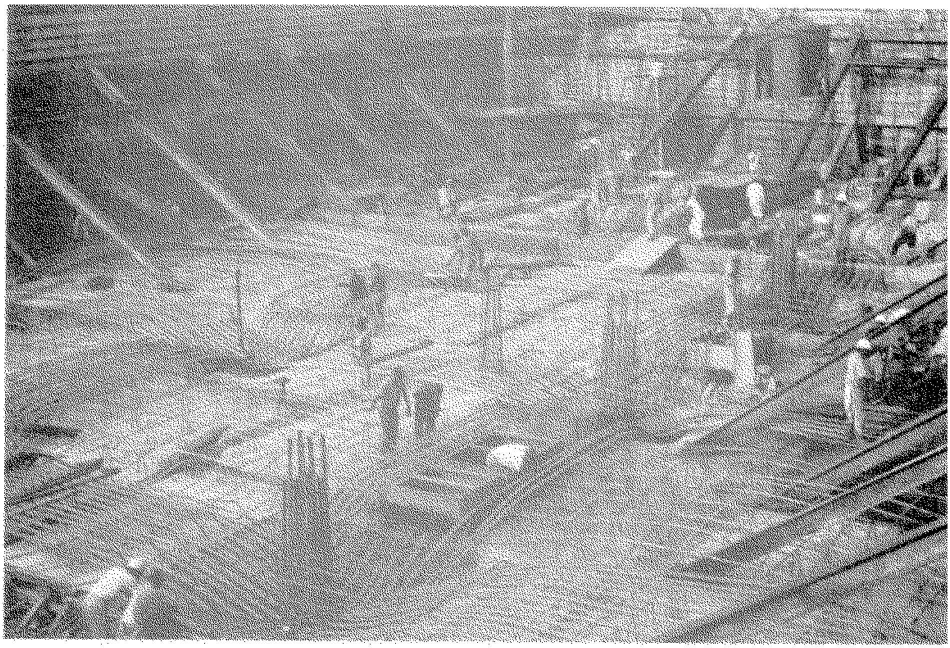


FIGURE 8.49 Concrete reinforcing prior to placement of concrete.

FIGURE 8.50 Prestress loads cause slab uplift.

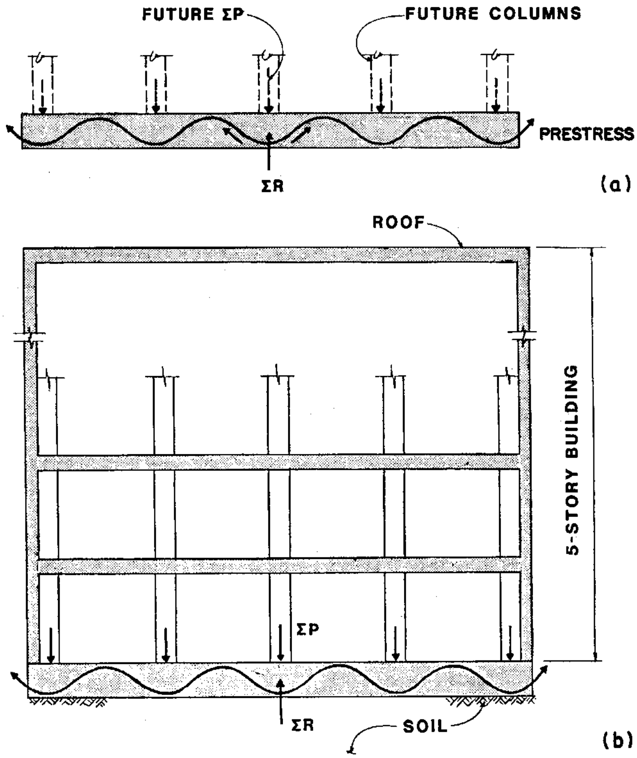


FIGURE 7.51 (a) Distress caused by design error in assumption of prestressing forces. (b) Lack of equilibrium of prestress forces is obvious.

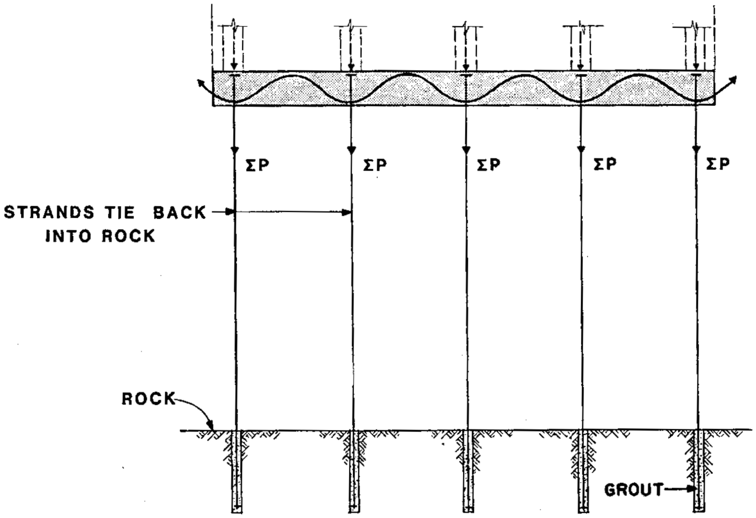


FIGURE 8.52 Rock anchors provide vertical reactions to prestressing forces (see Figure 8.49(a) and (b).

tailed compaction control program is set in advance of construction, an undesirable step will ensue as a result of differential settlement.

At the Boston project, the entrance to the 10-story building resulted in a hazardous step [painted red (Figure 8.54, see arrows)]. The proper design detail (Figure 8.53) will support the slab on grade (or fill) directly on a seat prepared in the foundation wall, thus preventing downward settlement.

Lesson to be learned:

To avoid steps caused by differential settlement, concrete slab on grade must be either cast on well-compacted fill or designed to bear on adjacent walls.

8.8 CONSTRUCTION ERRORS

There are two common types of construction errors:

1. Temporary protection measures during the construction phase
2. Foundation work itself

Most foundation failures involve the first type of error, relating to temporary shoring and bracings and temporary cofferdams for lateral protection and pumping operations. Because of the temporary nature of these structures, safety measures are often kept to a minimum for economy reasons.

Many construction errors on foundation work also are a result of the same deficiencies that exist in superstructure concrete work, such as improper concrete quality and improper rebar placement. Where deficient material is provided, in general there is absolutely no difference between superstructure and foundation work. The exceptions are foundation elements such as piles and caissons where "blind work" is involved. Cavities within cast-in-place piles are often found where quality control was lacking. The omission of verification methods early in the construction progress is partially responsible for the enormous losses caused by such errors.

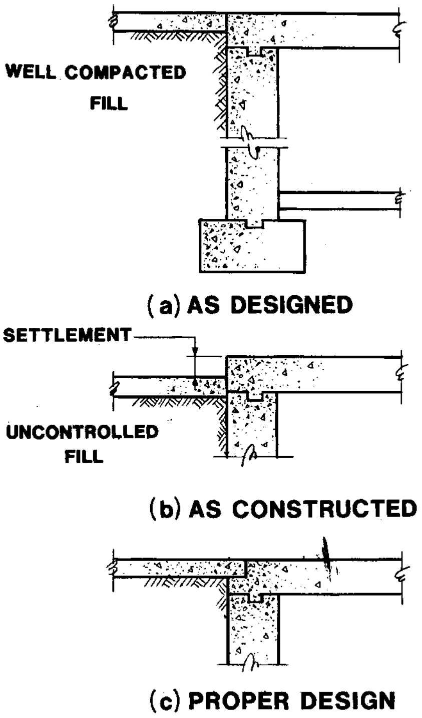


FIGURE 8.53 Slab design on fill.

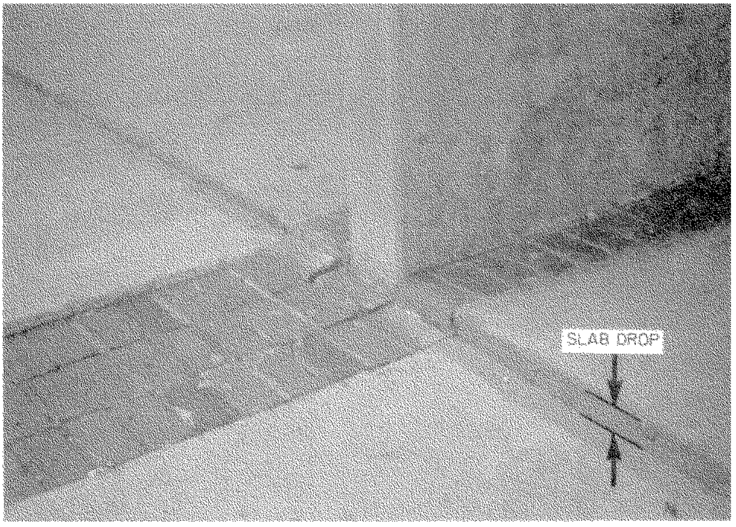


FIGURE 8.54 Slab settlement.

The unique cases involve the placement of foundation elements such as footings, piers, or piles on inadequate bearing soil strata—footings bearing on soft soils, or piles driven "short" so as to hang high in improper and insufficient bearing soil.

In 1958, a newly constructed 11-story building in Rio de Janeiro toppled and overturned. It was reported that the driven piles were about 20 ft (6.1 m) shorter than piles used for adjacent structures. Initially the settlement of the building was considered to be "natural." After several days of attempts to save the leaning building, the building was evacuated. Hours after the evacuation, the structure fell flat in *just 20 seconds.*

8.8.1 Lower East Side Collapses

On April 4, 1984, two structures located at Delancey Street in lower Manhattan collapsed during construction (Figure 8.55) about 1 h after a 3 ¼ in (89 mm) concrete floor slab had been poured. Two workers were killed by the collapse, including the general superintendent for construction.

On arrival on the scene a day later, I found that the debris was still being removed from the site. There was severe damage to the two adjoining structures, both of which were in precarious condition. One structure was subsequently demolished, and the other was braced and shored. Our investigation determined several improper construction procedures including inadequate field supervision. We also identified a number of design deficiencies.

Upon exposure after the collapse, several of the footings were found to have been cast as less than half (46%) the size required by the design drawings. Out of three interior footings, two footings had been cast eccentrically to the center line location [one 7% in (191 mm) and the other by as much as 11½ in (292 mm)]. The eccentrically positioned footings were visibly rotated and pitched from one end to the other by as much as 5 in (127 mm). The general layout of the building may be seen in Figure 8.56.

The structural analysis indicated that the bearing capacity of these footings was considerably reduced. Using the actual loads computed to be present at the time of collapse, we estimated that even using the optimum possible bearing capacity of these footings, they would have been loaded at 25% over ultimate capacity and had to fail. See Figures 8.57 and 8.58 for footing diagrams and load table.

FIGURE 8.55 Collapse of structure.

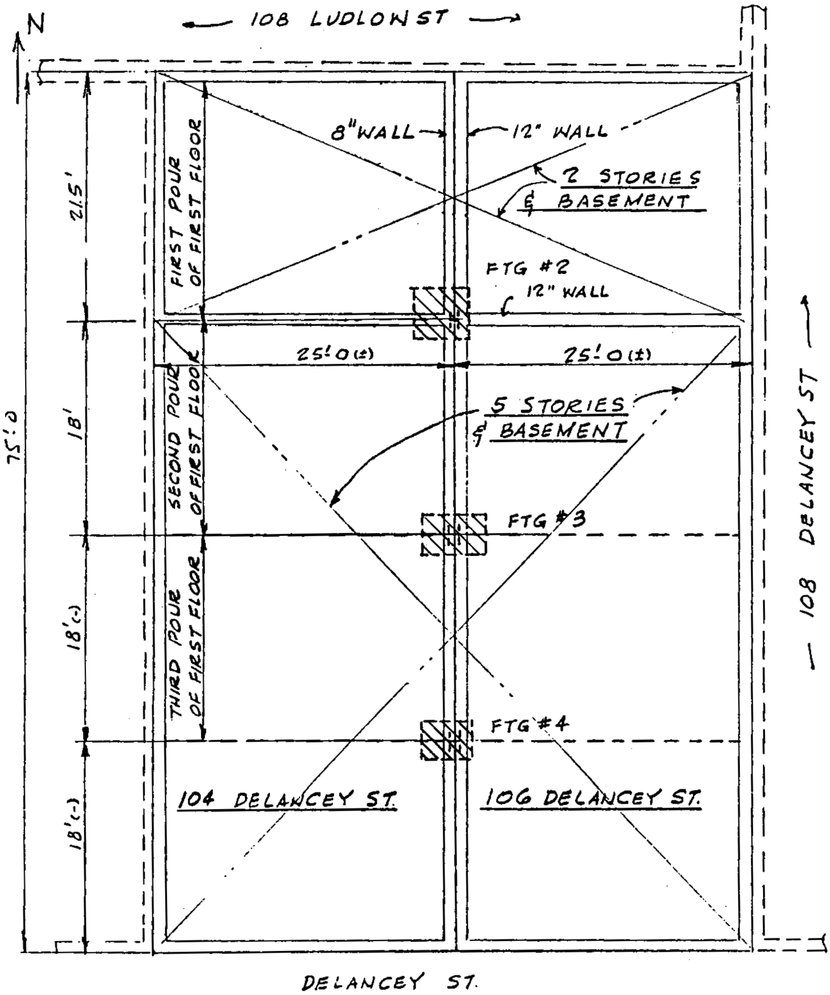


FIGURE 8.56 Floor plan of collapsed building (see Figure 8.55).

Another serious deficiency we uncovered related to the method of temporary support provided for the central bearing walls. These walls were subjected to a twisting rotational couple due to the weight of the two unequal bearing walls (Figure 8.59). The fact that the second floor was missing (it had previously been removed) compounded this effect through the lack of lateral support. The method of temporary support used by the contractor had a considerable inherent potential for instability. Thus, when the footings settled vertically, dislodging wedges and shims, the resultant movements initiated the total collapse.

One of the striking omissions in this failure was the lack of a construction procedure, which is essential to the successful execution of a difficult foundation reconstruction of an old structure.

The design deficiencies included main girders which were calculated to be stressed between 33 to 39 ksi (228 to 268 MPa) when subjected to full dead and live loads. Even the review of the design of the footings indicated overstresses in the range of 27 to 44%. Needless to say, the lack of shop drawings did not help matters.

FOOTING SIZE AND AREA			
FOOTING NO.	AS DESIGNED	AS CONSTRUCTED	REMARKS
FOOTING #2	5'-6"X 8'-6"=46.75sf	4'-8"X4'-7"= 21.4 sf	FOOTING CAST ECCENTRIC TO SUPPORTED COLUMN
FOOTING #3 (TOP AREA)	5'-6"X7'-0" = 38.5sf	5'-9"X3'-6" = 20.1 sf	FOOTING CONCENTRIC WITH SUPPORTED COLUMN BUT SIDE SLOPES, RESULTING IN AN ECCENTRIC BEARING SURFACE
FOOTING #3 (BOTTOM AREA)	5'-6"X7'-0"= 38.5sf	3'-9"X 3'-6" = 13.1sf	
FOOTING #4	5'-6"X7'-0" = 38.5sf	4'-2"X 3'-4"=13.9sf	FOOTING CAST ECCENTRIC TO SUPPORTED COLUMN

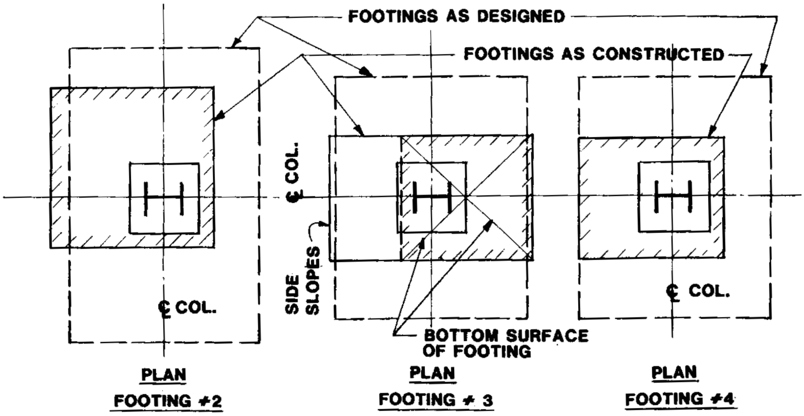


FIGURE 8.57 Substandard footings responsible for collapse (Figure 8.55).

Lessons to be learned:

1. Eccentrically loaded footings should be avoided at all costs, unless provided for in the design by oversized footings.
2. A construction procedure outlining step by step the method of temporary support, underpinning, shoring, and bracings is a must in construction of new foundations for old and deteriorated structures.
3. Lateral stability and eccentric effects on critical bearing walls must be considered and accounted for.
4. There is no substitute for good control of a project, including the preparation of shop drawings and thorough field inspection.

8.8.2 Apple Juice Factory—Drilled-in Piers

When I first saw this project, which had come to a halt, both owners and contractors were aware of the serious problem they were facing. The exposed sides of several concrete piers could be seen to contain concrete contaminated with large quantities of brown mud (Figure 8.60). Windsor probe tests had already been taken, and these confirmed the nonuniformity of the strength of the concrete. It was absolutely clear to everybody that expensive corrective work was required. The nature and extent of this work and the causes for the defects were sure to be in contention.

The foundation was being constructed to support an apple juice plant. The foundation system-

	1*	2*	3†	4†	5	6	7	8
	ALLOWABLE LOADS		DESIGN LOADS		DES.LD. ÷ ALLOW.LD.		DES.LD. ÷ ALLOW. LD.	
	N.Y.C. BLDG. CODE	ONE-THIRD OF ULTIMATE	WITH 1-12" & 1-8" WALLS	1-12" WALL ONLY	3 ÷ 1	3 ÷ 2	4 ÷ 1	4 ÷ 2
FOOTING #2	128^K	67^K	336^K	315^K	2.63 (163% OVER)	5.03 (403% OVER)	2.46 (146% OVER)	4.70 (370% OVER)
FOOTING #3 (FULL AREA)	121^K	216^K	358^K	313^K	2.95 (195% OVER)	1.66 (66% OVER)	2.59 (159% OVER)	1.45 (45% OVER)
FOOTING #3 (BOTTOM AREA)	79^K	272^K	358^K	313^K	4.53 (353% OVER)	1.32 (32% OVER)	3.96 (296% OVER)	1.15 (15% OVER)
FOOTING #4	83^K	114^K	358^K	313^K	4.30 (330% OVER)	3.14 (214% OVER)	3.77 (277% OVER)	2.74 (174% OVER)

	9	10	11**	12	13
	ESTIMATED LOAD AT TIME OF COLLAPSE		ULTIMATE LOAD	COLLAPSE LD. ÷ ULTIMATE LD.	
	WITH 1-12" & 1-8" WALLS	WITH 1-12" WALL ONLY	(PER WOODWARD-CLYDE)	9 ÷ 11	10 ÷ 11
FOOTING #2	251^K	217^K	200^K	1.26 (26% OVER)	1.09 (9.0% OVER)
FOOTING #3 (FULL AREA)	210^K	142^K	648^K	0.32 (32%)	0.22 (22%)
FOOTING #3 (BOTTOM AREA)	210^K	142^K	167^K	1.26 (26% OVER)	0.85 (85%)
FOOTING #4	210^K	142^K	342^K	0.61 (61%)	0.42 (42%)

ALLOWABLE BEARING DETERMINED BY WOODWARD-CLYDE CONSULTANTS INC.
*BASED ON ACTUAL FOOTING SIZES; N.Y.C. BLDG. CODE ALLOWABLE, 3 TONS PER SQ.FT.; EFFECT OF ECCENTRICITY NOT INCLUDED.
**EFFECT OF ECCENTRICITY CONSIDERED.
†DESIGN LOADS USED BY OUR FIRM.

	LIVE LOADS
1ST FLOOR:	100 PSF (RETAIL)
2ND FLOOR:	75 PSF (RETAIL)
3RD, 4TH, 5TH FLOORS:	100 PSF (LIGHT STORAGE)
ROOF:	30 PSF

FIGURE 8.58 Load table illustrates design flaws (see Figure 8.55).

consisted of concrete drilled-in piers (caissons), which were cast in place into big holes augured into the ground. These piers ranged in size from 30 to 42 in (762 to 1067 mm) diameter, with "bells" of 42 to 90 in (1067 to 2286 mm) diameter, and were 15 to 55 ft (5 to 17 m) long. The piers were designed to bear on top of bedrock with a bearing value of 12 tons per ft^2 (1149 kPa). Unfortunately, all the piers bad been completed at the time the defects were discovered.

The design of drilled piers with bells in clayey or silty sand is feasible provided the soil in which the bells are formed is cohesive. That condition may be realized where the bells are above the ground-water level. Where the shafts are flooded, as the case often was at this particular construction site, the success of belling is highly questionable. Especially doubtful is the possibility of installing the bells and maintaining them intact without the sides caving in and collapsing (Figure 8.61). The failed caissons confirmed this opinion. The presence of groundwater at several of the caissons should have been expected from the 23 borings taken at the site prior to construction.

What was more stunning was the interference of the design Engineer of Record with the contractor's methods and operations. When it became clear that it was impossible to pump the holes dry, the engineer directed the subcontractor to use larger pumps. [The ground-water infiltration was greater than the capacities of the pumps by more than 1000 gal/min (6308 L/s).] When the contractor did

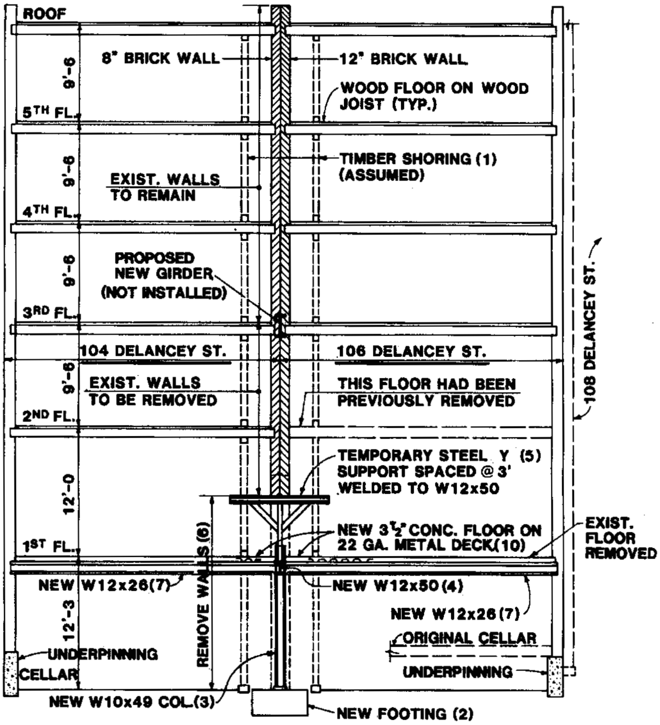


FIGURE 8.59 Construction alternatives contribute to collapse (see Figure 8.56).

not comply, the engineer bought large pumps using his own funds and delivered them to the contractor.

Thus the Engineer of Record committed three serious errors:

1. *A first major technical error.* It should be common knowledge that it is improper to cast concrete into holes while simultaneously pumping the holes. Such pumping only serves to disturb the wet concrete and mix it with soil, resulting in contaminated and defective concrete as per Figure 8.61 (properly filled caisson is shown in Figure 8.62).
2. *A second technical error.* There was no way for the engineer's inexperienced inspector to verify in the flooded shafts the conditions of the bearing rock and whether the rock was level or sloped (Figure 8.63).
3. *A policy error.* An engineer should never interfere with the contractor's methods and means, and definitely should not supply the contractor with the tools to perform the engineer's instructions, which also happened to be completely wrong. Needless to say, the subcontractor for the drilled piers should not have knowingly followed the erroneous instructions or should have known as a supposed expert in his field that they were flawed.

An experienced caisson contractor would also have known to continuously keep the bottom of the tremie (underwater concrete) pipe inserted into the fresh concrete (Figure 8.64). Whereas the selected foundation system for this structure was totally inappropriate (a system of H piles or even a continuous concrete mat should have been used), the construction methods used failed to produce a re-

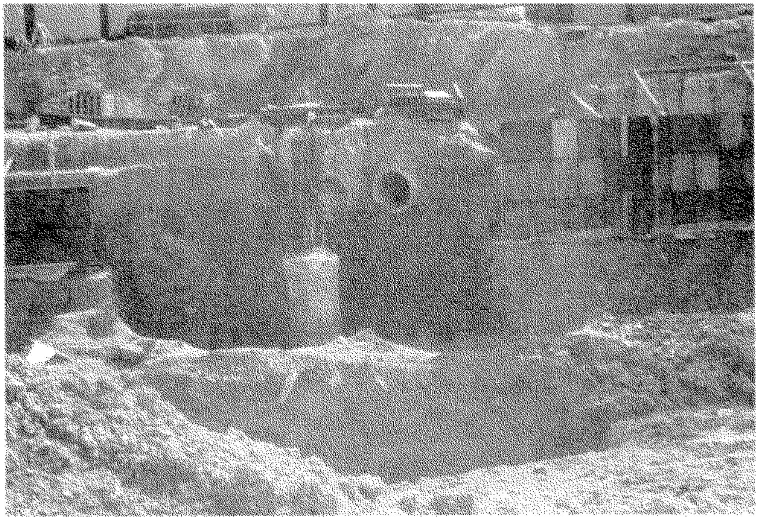


FIGURE 8.60 Defective concrete piers.

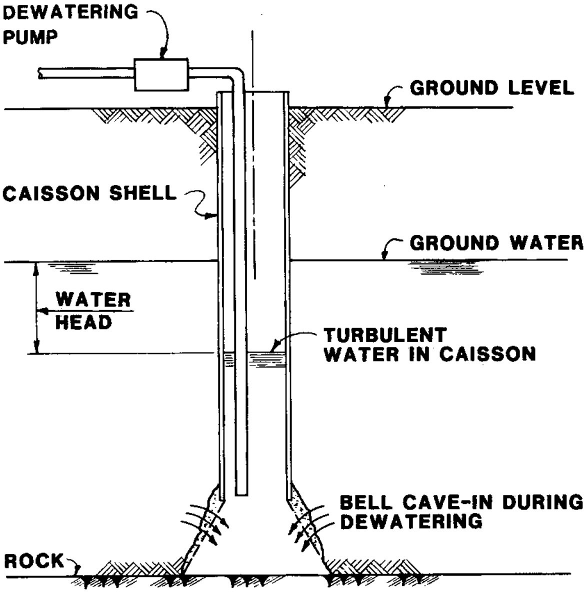
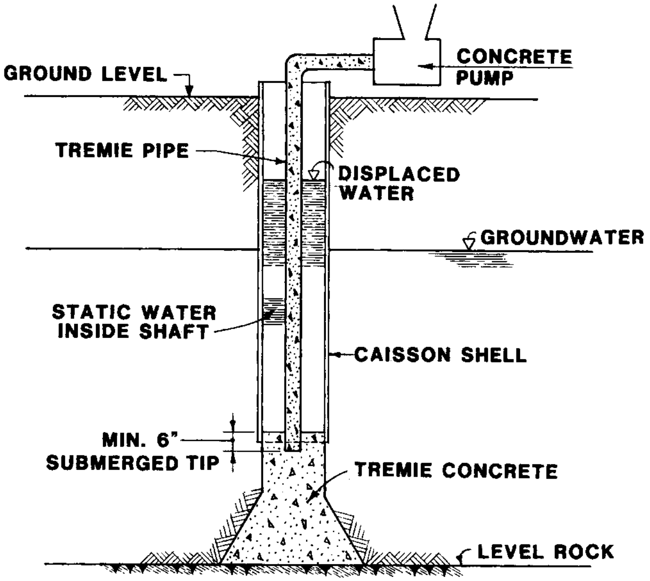
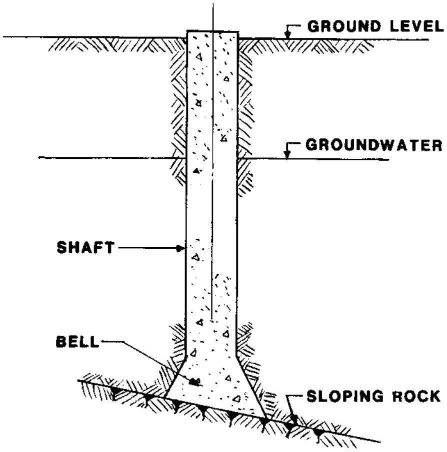


FIGURE 8.61 Caisson bell collapse.



CAISSON PROPERLY FILLED

FIGURE 8.62 Proper concrete placement in caisson.



IMPROPERLY SEATED CAISSON

FIGURE 8.63 Pier construction on sloping rock.

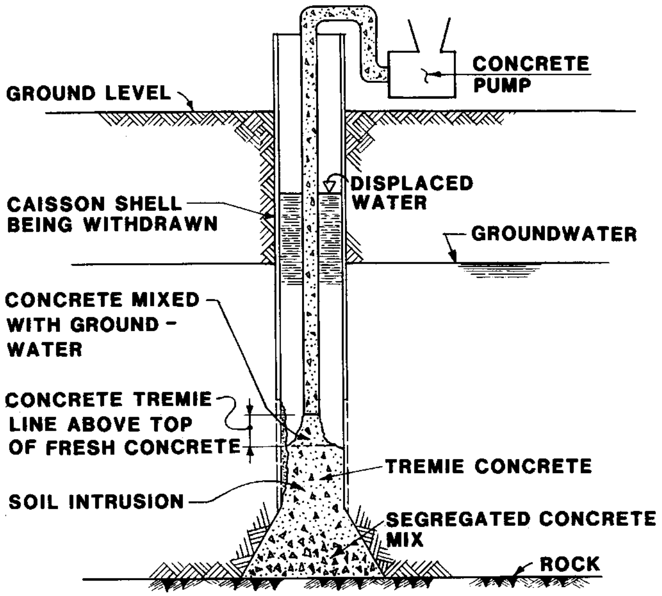


FIGURE 8.64 Improper use of tremie.

liable foundation. Not only was the concrete defective in many instances, but even the quality and levelness of the rock bearing strata could not be confirmed.

The sad ending to this story was that the foundation system as constructed had to be completely abandoned, and lengthy litigation ensued.

Lessons to be learned:

1. Designers should carefully check the feasibility of bell excavations in soil prior to concrete casting and especially in water.
2. Tremie placing of concrete should always be in still (static) water. Pumping during concrete placement operations should not be attempted.
3. The installation of drilled concrete piers should be performed only by experienced contractors.
4. Full-depth concrete test cores should be extracted from the first five piers, to verify the quality and strength of the concrete. The early detection of deficiencies is essential.
5. Engineers should not direct contractors' field operations, should not supply them with tools to perform the work, and should not interfere with their methods and means unless safety is involved.

8.8.3 Marble Hill

When cracks in the brick wall of the Marble Hill one-story structure started to widen, it became apparent that a serious condition existed below the concrete grade beam supporting the wall. After the soil adjacent to the grade beam was removed by excavation, a long diagonal crack was uncovered in the beam. The crack had been previously crudely patched with mortar, which itself was badly cracked and served no purpose whatsoever. There was no attempt to find the culprit! The failure was a classical shear crack with a vertical shift on both sides of the crack. A close look at this brick wall told the story of a settlement that had taken place prior to its construction. There was an additional course of brick on the left side of the crack (Figure 8.65). The solution to this mystery was found when, after further excavation, it was discovered that a pile designed to support the grade beam was missing. It was clearly a construction error. The attempt to patch the crack obviously was not adequate to correct the serious omission.

Lesson to be learned:

When cracking occurs, determine the cause of the distress. Do not guess! Only when the true causes are found can an efficient, long-term correction be designed and executed.

8.9 FLOTATION AND WATER-LEVEL CHANGE

Except in well-consolidated granular soils, a change in water content will modify the dimensions and structure of the supporting soil, whether from flooding or from dewatering. Many cases have been recorded where pumping by occupants for cooling water or by water companies to increase the capacity of their water supply resulted in receding of ground levels and in turn caused settlements with severe damage.

Pumping from adjacent construction excavations has also affected the stability of existing spread footings and even caused drag-down on short piles. This fact is in part responsible for the insertion of the requirement of recharging of ground-water levels in some foundation contracts. The effectiveness of these recharging pools has been and continues to be a touchy subject among foundation designers and geotechnical engineers.

Construction of new dams has also been found to be responsible for lowering of river levels, thereby causing severe damage and cracking of adjacent structures.

Clay heaves from oversaturation must also be expected and, in such soils, the structures must either be designed to tolerate upward displacement or else the supporting soil must be protected against flooding. Many parts of the world have trouble from heaving bentonitic clays, for which a

FIGURE 8.65 Foundation wall failure during construction.

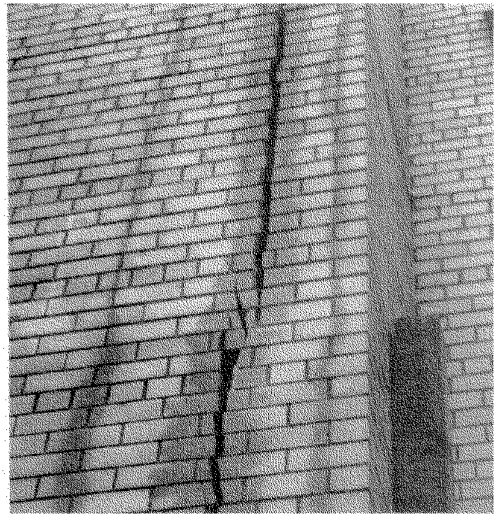


FIGURE 8.66 Vertical stress crack, hospital.

perfect solution is yet to be found. Where water infiltration into the soil is effectively prevented, and there is no change in water level, the soil volume should stay stable.

8.9.1 Brooklyn Hospital—Settlement of Delivery Room

This three-story hospital had performed perfectly for many years after its construction. Suddenly, masonry walls started to crack, "for no apparent reason" (Figure 8.66). The most severely affected wing of the building contained both the newborn delivery room and the general surgery room, whose continuous operation could not be stopped. I was actually measuring cracks inside the delivery room (dressed in scrubs) while a newborn was delivered. It was then that I found it necessary to temporarily support this wing, accomplished by the use of diagonal steel bracing system (Figure 8.67).

Once the structure was secured, we started the investigation of the sudden cracking and distress. To our amazement we found that the water supply company operating in the area had recently abandoned a great number of existing wells.

As a result, the ground-water level had risen sharply in the entire area. The immediate result was water penetration through foundation walls of the hospital which had not been waterproofed to such a high level. To solve the new problem of water infiltration into basement areas, pumps had then been installed by the hospital management. However, an unforeseen side effect was that the continuous pumping caused the removal of most of the fines in the soil directly below the footings supporting the affected wing, thereby causing the cracks and settlements.

There was no easy solution short of underpinning the building. This was accomplished by the use of jack-piles, which were pushed down into the ground, with the weight of the building serving as counterweights (Figure 8.68). The steel pipes which had been pretested for the required load were then filled with concrete, and then the jacks were removed.

Lessons to be learned:

1. Lowering or raising of ground-water level affects the bearing capacity of soils.
2. Pumping for new excavations may cause settlements of buildings. Therefore, water-level readings should be monitored, and protective measures taken.

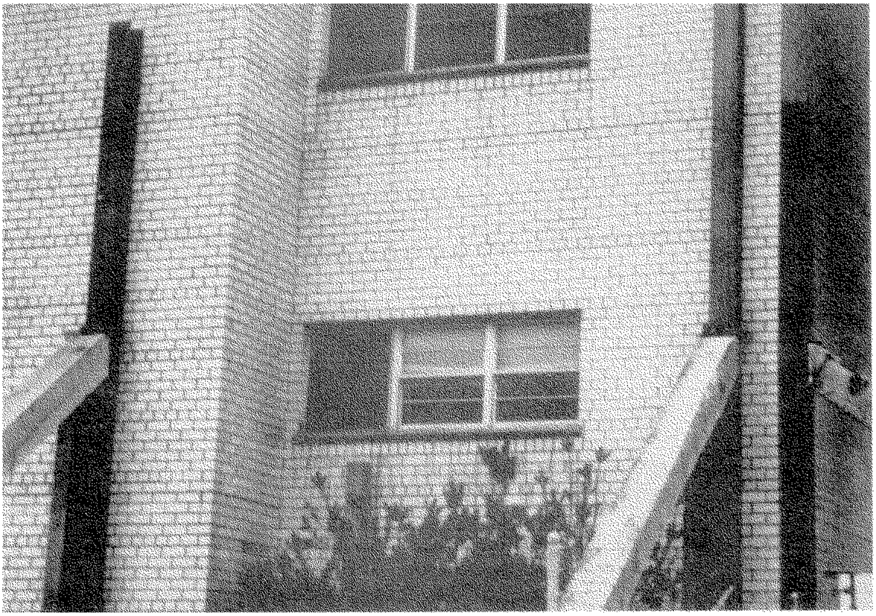


FIGURE 8.67 Shoring to abate settlement (see Figure 8.66).

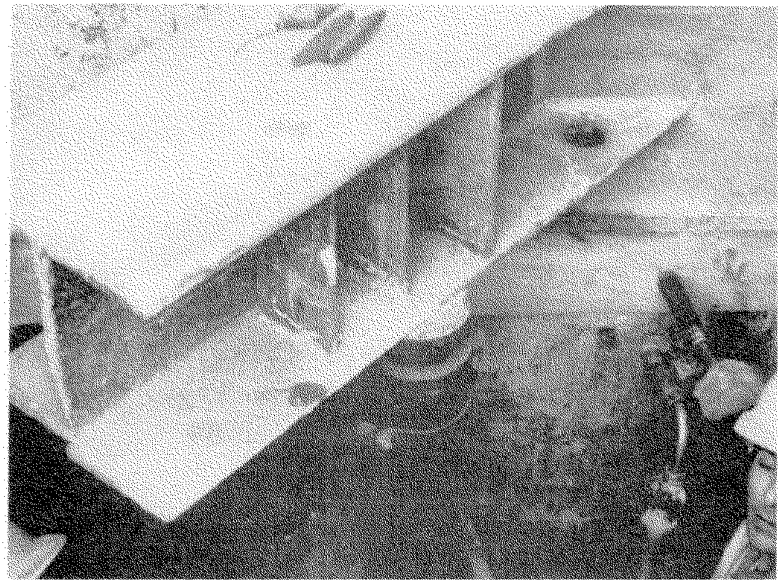


FIGURE 8.68 Underpinning and slabilizing building corner (see Figure 8.66).

8.9.2 Industrial Building, Hackensack, New Jersey

This one-story factory and office building was located in the bottom of a geological lake, atop deep glacial deposits of silts and clays in successive layers, forming what is known technically as varved silt deposit. In this very loose material, areas of soft shore land become marsh land. Considerable areas of such marsh land have been filled in to form usable development property. The existence of the soft underlying material should signal potential difficulties and must be taken into account in all construction operations. This type of soil is very susceptible to drainage and is associated with a considerable volume change.

The local sewer authority had issued a contract for the construction of a new sewer. A deep trench was excavated using steel "soldier beams" 10 ft (3.0 m) on center and timber sheeting. The excavation was braced at two levels. As the excavation approached the factory, various signs of damage started to appear in the form of cave-ins and vertical and diagonal cracking of walls and partitions (Figures 8.69 and 8.70).

The damage was reviewed by our firm, and it was evident that the undermining was caused by the drainage operations in the sewer trench causing a shrinkage of the subsoil under the building's foundation. This was followed by the lateral movement of soil from under the building toward the deep hole at the sewer trench. The nearest edge of the sewer excavation was approximately 220 ft (67 m) from the building. Yet the effect of the drainage of the sewer excavation extended to great distances on each side of the sewer, and soil settlement was even visible in the parking area in addition to along the walls of the building itself.

Fortunately, the main structure of the building was supported on piles which had been carried

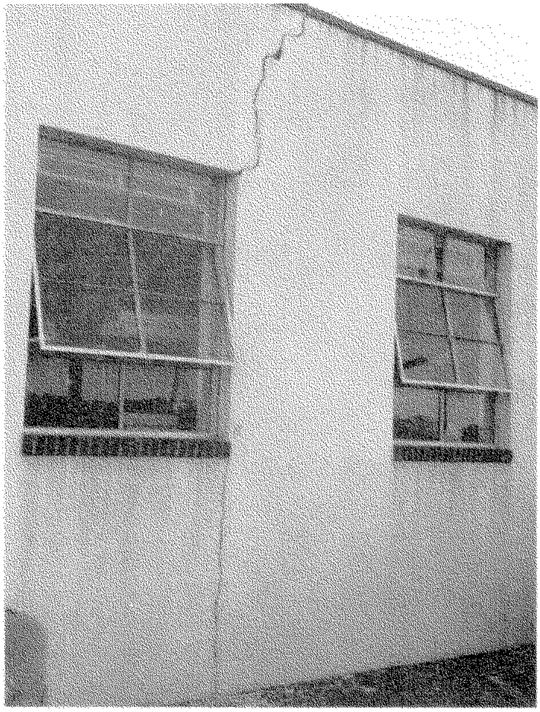


FIGURE 8.69 Masonry distress caused by soil shrinkage (loss of bearing support).

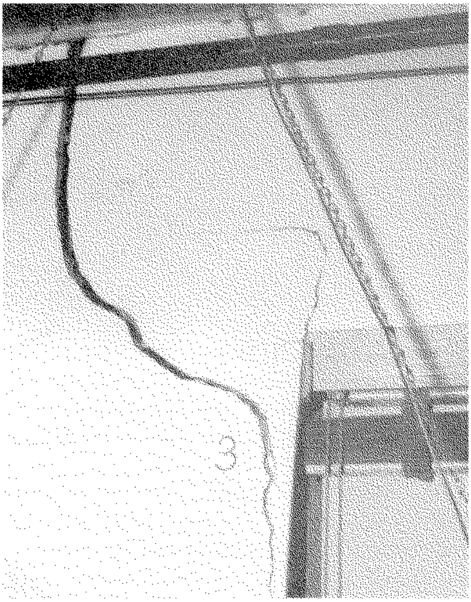


FIGURE 8.70 Interior distress (see Figure 8.69).

into materials deep enough and dense enough not to be affected by this drainage. There was, however, still the effect of "negative friction" on the piles, with the additional soil weight adding to the loads which were already present.

Once the pumping operations ceased, no additional damage was recorded. Slabs which had settled by as much as 3 to 4 in (76 to 101 mm) had to be removed and rebuilt. Slabs which settled to a lesser degree, leaving a hollow space between the underside of the slab and the ground, were restored by pressure grouting. Wall cracks were repaired using ordinary cement mortar and refinished.

Lessons to be learned.

1. The ground-water level outside a deep excavation should be monitored using piezometer pipes installed at the adjacent structures which may be affected. Criteria for the maximum permissible water-level drop should be set.
2. In the event of a drop exceeding the criteria and/or the initiation of damage, the pumping methods and procedures should be reviewed to minimize the damage.
3. The services of competent geotechnical and structural engineers should be solicited.

8.9.3 West 41st Street Structure—Wood Piles

Damage to the exterior brick masonry of this building was caused by settlement of timber piles resulting from lowering of ground-water level at an adjacent railroad construction. Probes below the building showed that the timber piles were rotted and could not accommodate the additional loads generated by the drop of the water level. The damage to exterior and interior masonry is shown in Figures 8.71 and 8.72.

8.10 VIBRATION EFFECTS

Earth masses which are not fully consolidated will change volume when exposed to vibration impulses. The vibration source can be construction equipment, especially pile drivers, mechanical

FIGURE 8.71 Interior distress (Figure 8.71).

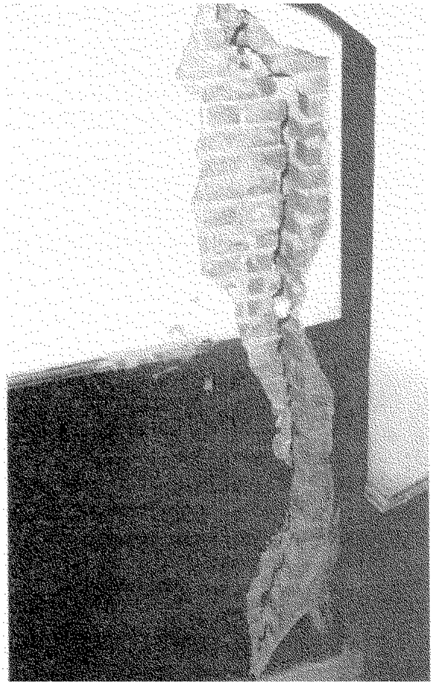


FIGURE 8.72 Interior distress (see Figure 8.71).

equipment in a completed building, or even traffic on rough or potholed pavement, as well as blasting shock.

We have investigated many cases of damage as a result of pile driving. Heavy damage can be caused when both impact hammers and vibratory hammers are used. In several cases, entire rows of buildings have had to be condemned as unsafe and were subsequently demolished.

Blasting operations must be carefully programmed, to avoid serious damage to adjacent structures. A criterion for maximum particle velocity, which is a measure of the intensity of vibration, has to be established, and constant readings by the use of seismographs have to be taken. Damage from blasting is often so potentially severe that the size of explosive charges have to be limited to keep the vibrations to tolerable levels. Factors such as the condition, rigidity, material brittleness, and age of nearby structures must be considered. Where adjacent structures are founded on rock, the seam structure of the rock itself also must be taken into account. A complete study of vibration transmission and correlation among intensity, wave length, and possible damage to various types of structures is given in a Liberty Mutual Insurance Company report, "Ground Vibration Due to Blasting," by F. J. Crandell.[4]

8.10.1 Lower Manhattan Federal Office Building

After heavy impact hammers were used to install the piles for a prestigious building, the lower Manhattan Federal Office Building, the adjacent structures were seriously shaken. Street pavements settled (Figure 8.73) as much as 16 in (406 mm) after only one-fourth of the piles had been driven. The installation of steel sheetpiling was then attempted in an effort to protect the streets against further

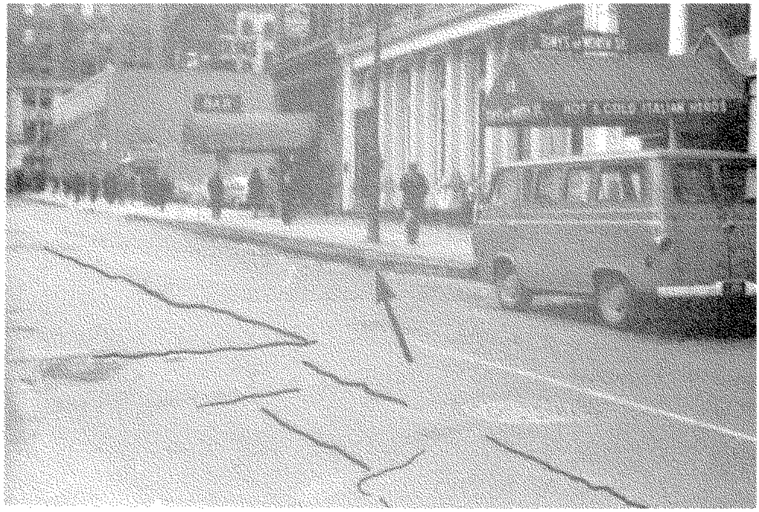


FIGURE 8.73 Pavement settlement due to pile-driving vibratins.

subsidence. This attempt was successful against further settlement and damage to sewer and water lines, but even after the contractor shifted to vibratory hammers, the damage continued. High-rise buildings as tall as 19 stories were also damaged. The damage was so heavy that eventually all buildings within a radius of 400 ft (122 m) were condemned as unsafe and had to be demolished. In a similar construction some 1000 ft (305 m) away, when first signs of trouble became evident, the adjacent buildings were underpinned on piles down to the tip level of the new building piles. From here on, the consolidation of sand layers as a result of pile driving had no ill effect, as these sand layers were no longer supporting building loads.

Lessons to be learned:

1. The use of vibratory pile-driving equipment must be avoided where possible. Such equipment should be used only after careful evaluation of potential damage.
2. Underpinning of adjacent structures must be seriously considered for support to minimize damage.
3. Where possible the use of augured piles should be considered.

8.11 COFFERDAMS—THE 14TH STREET COFFERDAM

In order to construct a deep pumping station picking up flow from a 9 ft (2.74 m) sewer on the east side of New York City, a temporary steel cofferdam was constructed. The size of the pit was 100 ft × 250 ft × 37 ft (30.48 in × 76.20 in × 11.28 m). Adjacent was a high-rise public housing building on short piles so that the excavation was 30 ft (9.14 m) below the pile tips.

The site was filled-in land overlying some organic silt and with sand 80 ft (24.38 m) down, showing full hydrostatic pressure. It was not possible to drive interlocking sheetpiling through the sand to impervious till or rock, about 95 ft (28.96 m), since major removal of the rough fill might affect the stability of the apartment building. The solution was to core through the fills and place 30 in (762 mm) pipe sleeves and drive soldier beams on 5 ft (1.52 m) spacing, first welding sheetpile interlocks to two edges of the wide flange (see Figure 8.74). The soldiers were driven to bedrock.

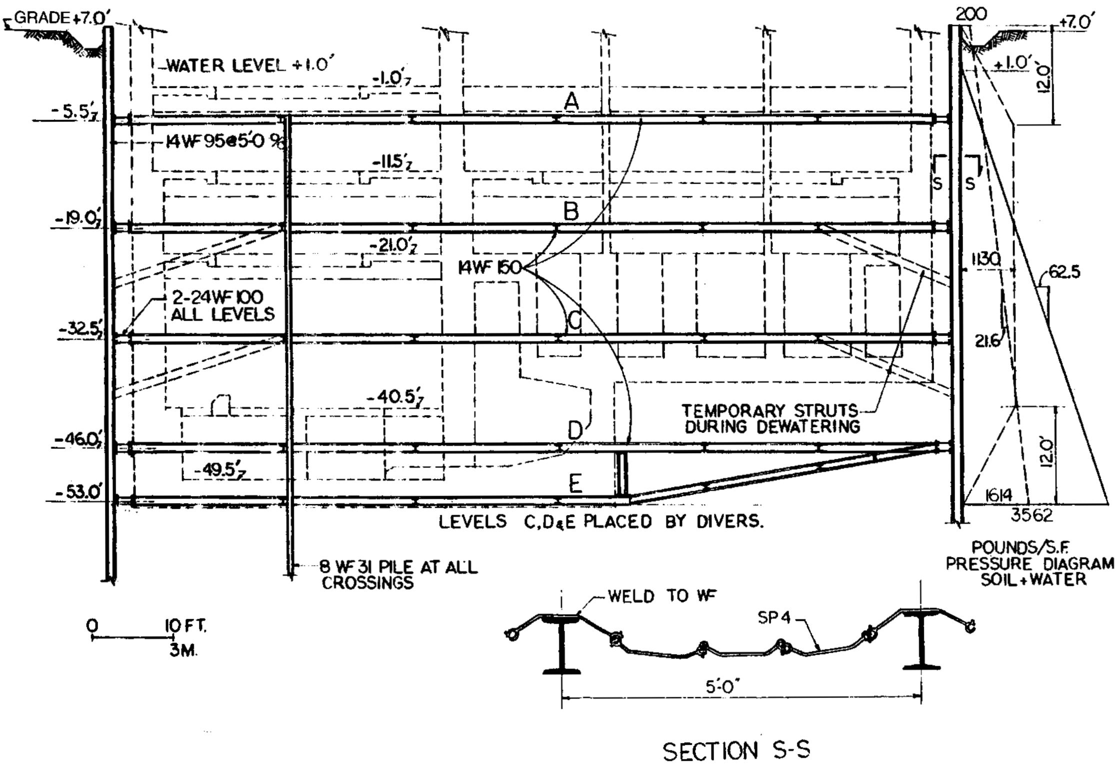


FIGURE 8.74 Soldier beams used to construct cofferdam.

The space between the soldiers was then filled in with three units of flat steel sheeting, draped into a catenary arc and designed to act as such. Sheeting was driven into the till layer. The horizontal cross-bracings were all installed by jacking design loads into all members in each direction. Bracing levels were installed as the cofferdam was excavated and pumped out. Water was controlled by two deep wells carried into the sand. Some of the sheeting engaged an old buried timber crib, and some of the interlocks were broken so that water cutoff was not accomplished.

Although the basic design of the interlocking sheet piles was proper, neither the 5 ft (1.52 m) spacing nor the verticality of the piles could be accurately controlled. Thus, the cofferdam walls failed at certain locations as the sheet-pile fingers tore, leaving the wall with open gaps for inflow of soil.

After several blows through the bent sheeting stopped progress, a slurry trench cutoff to rock was installed along the distorted and damaged section, which fortunately was located furthest from the apartment building. The open area near the apartment building was recharged with pumped water to maintain the original water table level, and no damage resulted either to the building or to the trees.

Lesson to be learned:

Driving of sheetpiling should be closely controlled so that extremely hard driving with potential damage to the sheets can be avoided.

8.12 CAISSONS AND PILES

8.12.1 The North River Project—Caisson Deficiencies

Manhattan's enormous North River water pollution project was constructed on a 30-acre (121,400 m^2) concrete platform supported on 2500 drilled caissons (Figure 8.75). The caissons extend to the Hudson River bottom and are socketed into the rock underlying the lower sand layers. They were from 80 to 250 ft (24 to 76 m) long and were constructed of steel pipe shells in diameters varying from 36 to 42 in (914 to 1067 mm).

For economy reasons, the caissons were designed to carry very heavy loads of up to a maximum of 5000 tons (45 MN) each. During the construction of the platform, two major problems arose:

1. Defective concrete was found to be cast in the caisson shells.
2. Caissons shifted laterally at the top and bent. These problems came to light after a substantial number of the caissons had been completed.

A full-scale study was launched, and it was determined that the problem of the defective concrete was caused by improper tremie concrete placement. It became clear early during the installation of the caissons that about one-quarter of the caissons had low-strength concrete and discontinuities of different varieties: uncemented aggregates, unhydrated cement, and sand layers intermixed with the concrete.

After driving the steel shells into the river, the shells were seated into the rock. Sockets were then drilled in the rock, and the shells were emptied of river mud and sand. The contractor made several attempts at casting the caissons in the dry cement after sealing the bottom and pumping the water out of the shell. When these attempts failed, a tremie method (casting concrete in water) was used. This method, which unfortunately did not utilize pressure pumping of concrete, also failed, a fact that was brought to light only later. This was so because the accidental lateral shift of the caissons

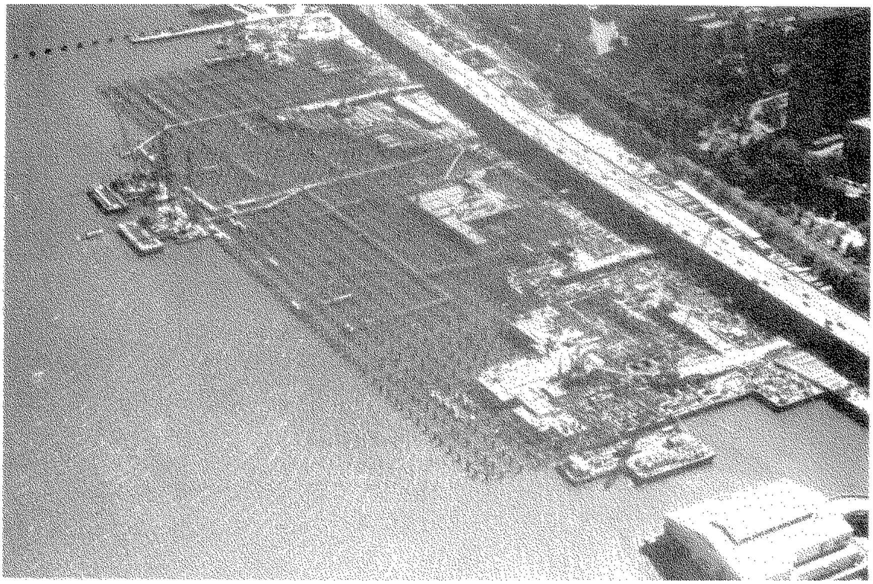


FIGURE 8.75 2500 caissons installed in Hudson River.

caused sufficient concern, necessitating full-scale in-place load tests. The first of the load tests (described later) resulted in failure of the tested caisson, which sank vertically, a failure which confirmed the concrete defects within the caissons.

The concrete subsequently was repaired by new high-strength rebar [75 ksi (517 MPa)] bundles installed in vertical holes drilled in the caissons. These holes were then pressure-grouted with high-strength grout.

The *shifting of the tops* was determined to be the result of one-sided soil dredging of a deep trench. The creation of the trench caused unbalanced lateral soil pressure on the caissons, resulting in horizontal movement and excessive bowing. This horizontal movement of the tops exceeded 2 ft (610 mm) in many cases. The caissons were later forced back into position by a “pulling program” involving the application of high lateral loads coupled with jetting of the caissons. It was only after the lateral shift of several hundred caissons occurred that the concrete problem was discovered, as it was decided to core-drill several of the caissons. The coring revealed many deficiencies within the caissons, where some “concrete” near the bottom of the caissons was nothing but sand (Figure 8.76). Before any remedial measures were considered and proposed, it was decided to load-test one of the affected caissons. To determine the load capacity of the shifted and bent caissons, a full-scale load test was ordered. The tested caisson was 36 in (914mm) in diameter, 98 ft (30 m) long, socketed 5 ft (1.5 m) into rock, with a design capacity of 680 tons (6.1 MN). The top of this caisson had been displaced laterally as much as 2.35 ft (0.71 m) after it was installed.

The vertical load was applied to the test caisson by means of four hydraulic jacks; the jacks reacted against a loading frame held in place by four vertical cable anchorages grouted 45 ft (13.7 m) into sound rock (Figure 8.77).

Four 0.001 in (0.025 mm) dial extensometer gages were mounted on an independent common frame to measure vertical displacements at the top of the caisson. The top of the test caisson was braced at the top to adjacent caissons to prevent lateral translation during the test. The braces were pin-connected to eliminate undesired transferral of the vertical load through the bracing system. The load was applied in two phases.

During the first phase, a load of 850 tons (8.5 MN) was applied. The load was cycled in small increments. All loads were maintained until the vertical displacements were stabilized. Only the 765- and 850-ton (6.8- and 7.5-MN) loads were maintained for 15 and 18 h, respectively. Later the load was *cycled five times.* The vertical displacement under a load of 850 tons (7.5 MN) (125% of the design load) after 17 h was 2.6 in (66 mm) with a permanent set of 2.2 in (56 mm).

During the second phase, the load was increased and cycled further in increments to a maximum of 1360 tons (12 MN).

This maximum load was held for 60 hours and then recycled again ten times. To everybody’s shock, the caisson failed to support the load and sank. An additional deflection of 3.6 in (92 mm) was measured for a total vertical displacement of 5.8 in (147 mm). The total permanent set was 4.5 in (114 mm) (Figure 8.78).

The excessive vertical displacement and permanent set were caused by either one or a combination of two factors:

1. The existence of uncemented zones within the lower 40 ft (12.2 m) of the caisson resulted in the transfer of the entire caisson load to the steel shell which was subjected to a stress of 50 ksi (345 MPa).
2. Lacking sound concrete in the socket itself (there was actually uncemented sand in the socket), the cutting edge of the shell was punched further into the socket.

   It was not possible to determine whether the lateral shift of the caisson was, in fact, a significant factor causing the vertical displacement of the caisson and its eventual failure. For this reason an additional caisson was tested 2 months later. This later test proved that the lateral displacement of 2.5 ft (0.76 m) at the top of the caisson had no discernible effect on the elastic behavior and load-carrying capacity of the structurally sound portion of the caisson. Therefore, all caissons with displacements of less than 2.5 ft (0.76 m) were incorporated into the structure. The unbalanced lateral loads resulting from the lateral shift were subsequently balanced by batter caissons.

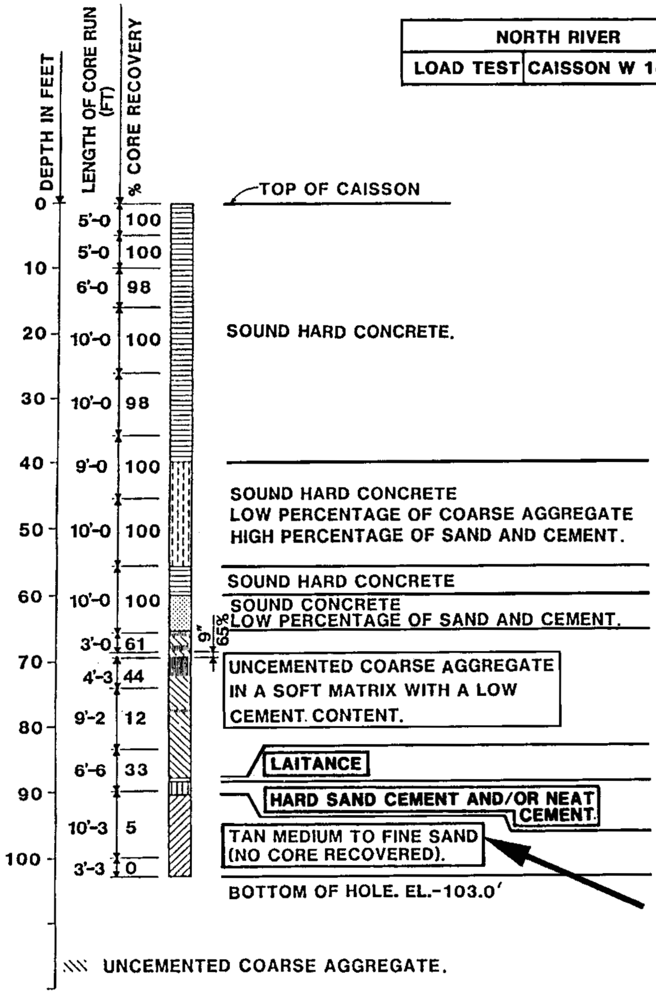


FIGURE 8.76 Coring log, load test caisson (see Figure 8.75).

A second defective caisson was then tested similarly. This caisson was also cored with an NX-size double tube core barrel prior to load testing. What we found was that the upper 80 ft (25 m) contained generally sound concrete, contrary to the first tested caisson. To measure the vertical displacement at the bottom of the caisson, measuring devices were inserted into 1 in (25 mm) diameter pipes installed in holes drilled in the caisson. The devices were ¼ in (13 mm) diameter smooth steel rod tell-tales lowered into the pipes. This caisson held the load for the required 60 hours, with a total displacement of only 0.35 in (9 mm). This led us to conclude that the caissons were repairable and could be incorporated into the structure without any reduction of their load capacity. We also determined that the presence of the powdery cement layer was apparently the result of the concreting procedures employed, whereby up to three bags of cement were deposited to absorb the water which was not removed by pumping out of the shell.

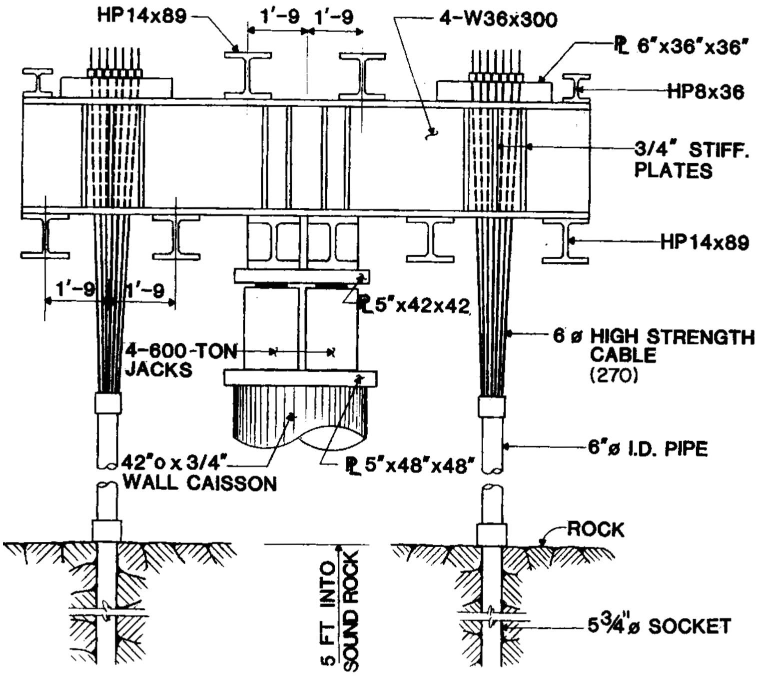


FIGURE 8.77 Test-loading questionable caissons (see Figure 8.76).

Remedial Work

The caissons were repaired by bridging the discontinuities with grouted rebars. Two 10 in (254 mm) holes were drilled within the concrete for the full depth of the caissons, including the socket, into rock. After each hole was cleaned of loose concrete, rebar bundles consisting of twelve no. 11 high-strength [75 ksi yield (517 MPa)] bars tied together around pipe spacers were lowered into the holes and grouted (Figures 8.79 and 8.80).

Lateral Shifting of the Top of the Caissons

As soon as it was discovered that a large group of caissons had shifted laterally after installation, an immediate investigation was launched. Survey teams using laser-based instruments took readings of 180 individual caissons to establish their accurate locations. The top of the caissons were surveyed for a year. (See typical movement chart in Figure 8.81). Lateral displacements of as much as 2.5 ft (0.76 m) were measured. Temperature variations were found (by analytical computation) to account for only a small portion of the movement. Examination of the horizontal steel 8HP36 staylath members (Figure 8.82) and their connections showed them to be in good condition and without any signs of distress. Analytical computations established that very high loads were, indeed, required to be generated to cause such a shift.

It was also determined that a one-sided soil pressure is capable of generating such high loads. The actual behavior of the group of caissons which were tied together by means of steel bracings followed the pattern of lateral shift resulting from an unbalanced lateral load. It was soon discovered that such unbalance did, in fact, occur as a result of a sizable dredging operation performed to remove existing boulders which were obstructing caisson-driving operations. It was established that, although most caissons moved west (toward the dredged trench, which was excavated parallel to the

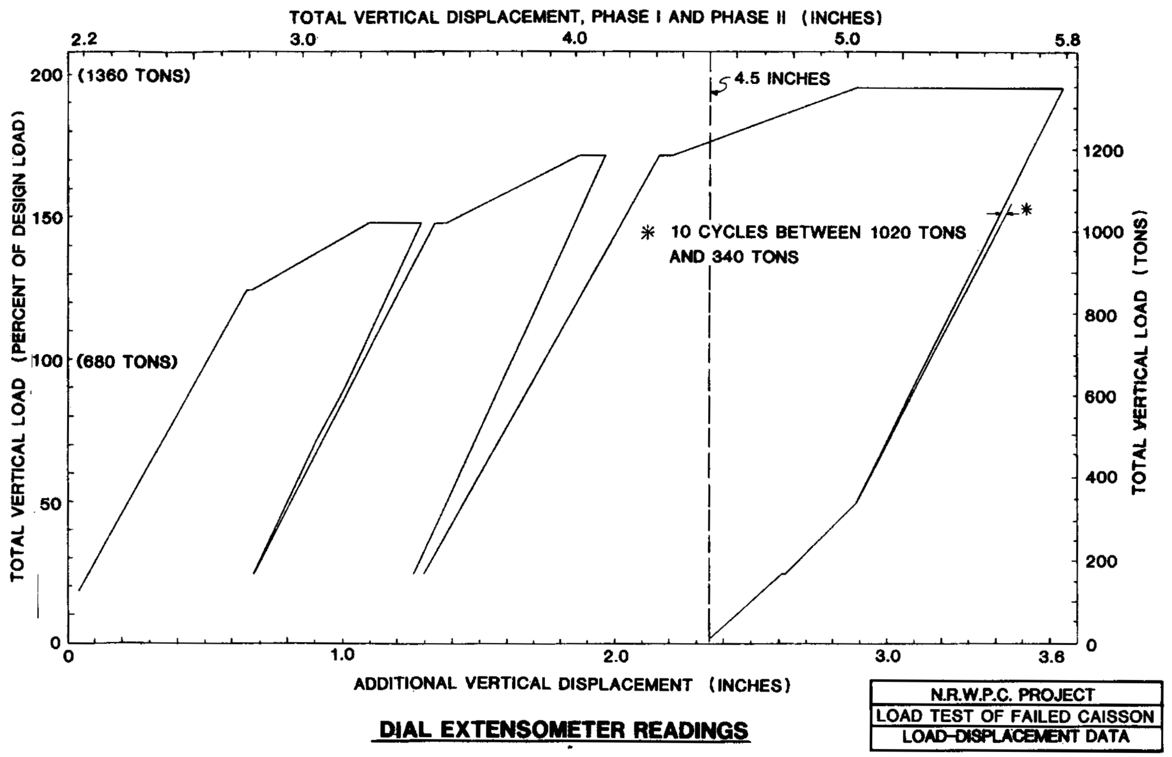


FIGURE 8.78 Vertical load versus vertical displacement for caissons.

shoreline), several caissons located west of the trench did, indeed, move toward the trench in the easterly direction.

At this time we had to determine whether the movements in the southeast section of the foundation systems were continuing and/or could be expected to continue after the completion of the concrete deck supported by the caissons. We also needed to assess the effect of such possible movements on the superstructure.

To answer these questions, we released a number of caissons from their staylath bracings and installed inclinometers in drilled holes within the caissons. The inclinometers, which are devices that measure tilt, or the angular deviation from the vertical, were the slope indicator type.

The results of the surveys (Figure 8.83) indicated that, except for the initial adjustment of the casing after installation, no movement was occurring. We therefore concluded that the westward movements of the caissons and/or soil in the southeast area of the project had ceased.

The main concern at this juncture was to establish whether the structural integrity of the caissons was still preserved. The stress analysis that followed did confirm this fact, and we then looked for a method of moving the displaced caissons back to their original vertical positions.

"Pulling" procedures were developed, using "come-alongs" to move the caissons laterally. The loads (measured by dynamometers) were applied in small increments, so as not to damage the caissons. Jetting of the river bottom was utilized to assist in pulling the caissons to the desired locations. It should be pointed out that the enormous construction difficulties encountered in this project were effectively resolved due to the excellent cooperation and combined efforts of the owner and both the design and construction teams.

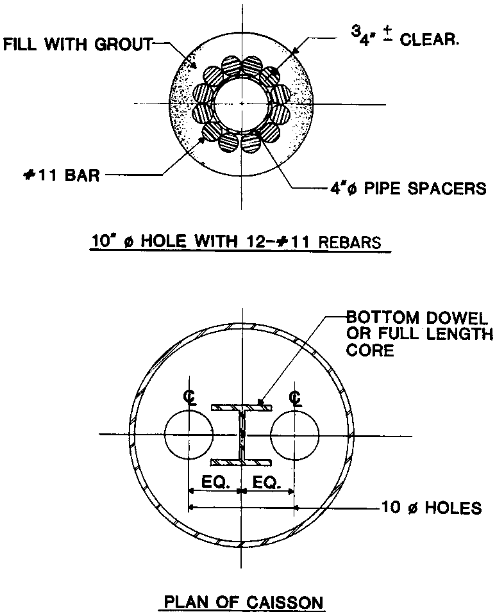


FIGURE 8.79 Repair of defective caissons.

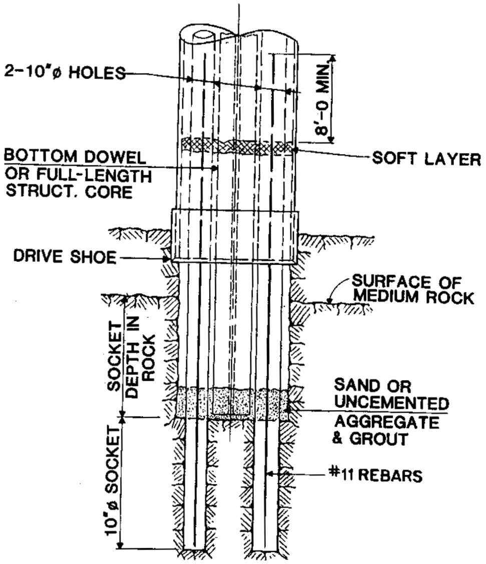


FIGURE 8.80 Schematic diagram for grouted rebar extended 12 ft (3.7 m) mb rock.

Lessons to be learned:

1. Tremie casting of concrete into long caissons should be performed under pressure by the use of pumps.
2. The bottom of the tremie pipe must at all times be embedded into the fresh concrete.
3. The quality and strength of concrete in caissons, and the reliability of the tremie concrete procedures, must be confirmed by core drilling at the beginning of the operation.
4. Cutting deep trenches adjacent to caissons or similar pile foundations will result in a lateral shift and should be avoided.
5. Good cooperation among the various parties involved in a construction failure is essential to a speedy, successful, and litigation-free resolution.

8.12.2 Philadelphia Federal Office Building—Caisson Settlement

In 1974, several piers of this 23-story steel frame high-rise building settled with one caisson settling approximately 2 in (50.8 mm). The base of the building was 274 ft × 240 ft (83.52 m × 73.15 m) with a narrow tower approximately 120 ft × 150 ft (36.58 m × 45.72 m) in size. Wind frames were constructed in two perpendicular directions by the use of welded and high-strength bolt connections.

After stress analysis was performed, it was decided to jack one of those columns that settled; however, no action was taken from 1974 to 1976 (2 years).

In 1977, four 400-ton (3559-kN) jacks were placed between the structural steel column and the caisson. The structure was to be raised 1 in (25 m) in two stages. The relative movement of the col-

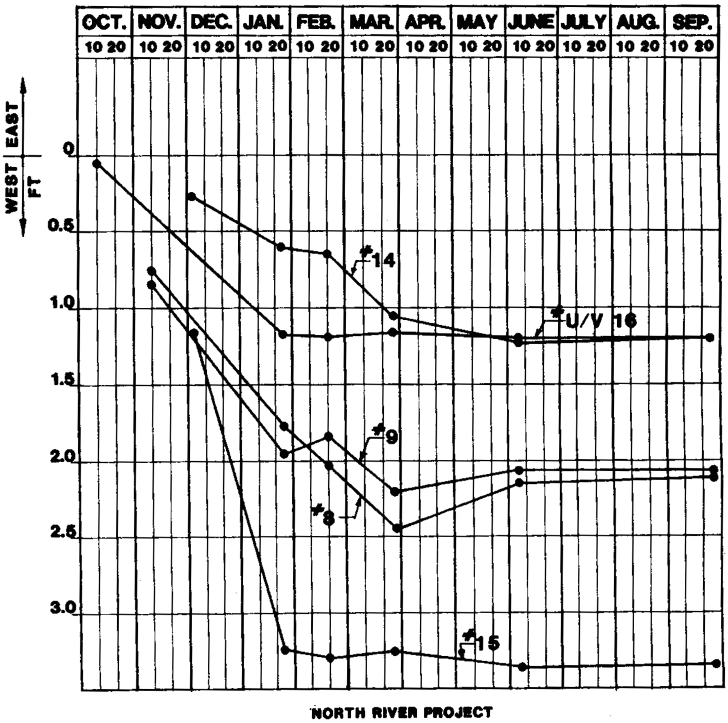


FIGURE 8.81 Test survey of caisson movement before and after stabilization.

umn in respect to the caisson was measured by dial gages. At this time accurate survey revealed that 44 caissons had settled.

The original plans called for the caissons to bear on rock at an approximate depth of 60 to 70 ft (18.29 in to 21.34 m), but later (1969) a change order was issued (with a $70,000 savings) which required the caisson to go down only 50 to 55 ft (15.24 in to 16.76 m).

During construction, there were reports of sudden drop of the top of the caisson and of water accumulation at the top of the caissons. There were two possible causes for the settlements of the caissons:

1. Inadequate bearing strata at the bottom of the caisson
2. Discontinuities and voids within the concrete caisson itself

The load test that was conducted was inconclusive, since the monitoring setup was not properly instrumented. Whereas only the top of the caisson was monitored, it was not possible to determine whether the bottom of the caisson dropped.

What was confirmed was that the jacking of the caisson actually resulted in a drop of the caisson instead of the lifting of the column above.

The final repair utilized grout which was injected into, around, and below the settled caissons.

Lessons to be learned:

1. Caisson construction must include coring of typical caissons to confirm the quality and continuity of the concrete. Coring program must be performed at the start of construction.

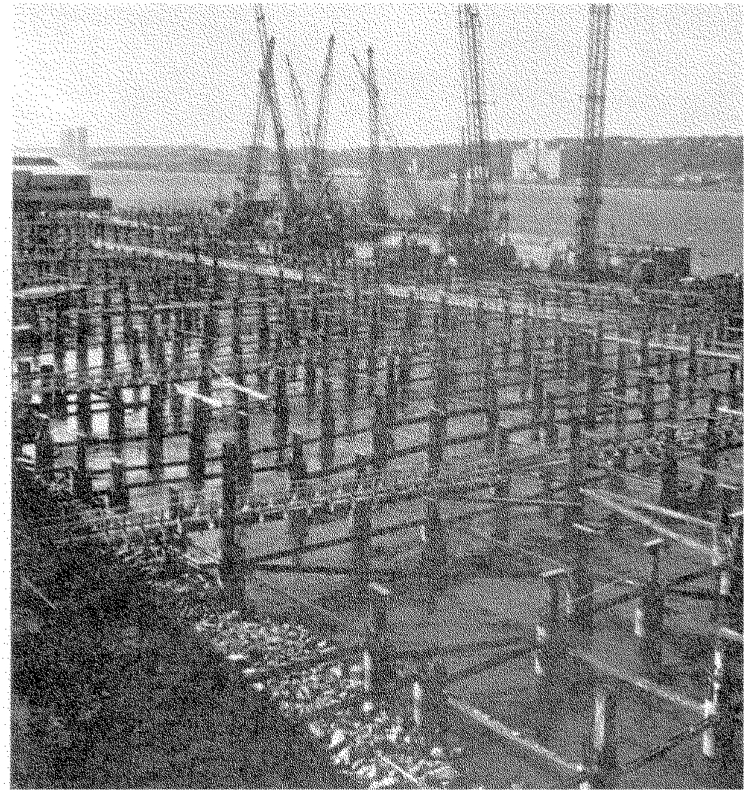


FIGURE 7.82 Bracing caissons (see Figure 8.75)

2. Bearing surface must be confirmed by 5 ft long (1.52 m) rock cores extracted prior to casting of the concrete.

Recommendations for design and construction improvement include avoiding common pitfalls and review of checklists.

8.13 COMMON PITFALLS

1. One structure supported on two different foundation systems, i.e., piles and spread footings
2. Inadequate drainage for retaining and basement walls, leading to buildup of hydrostatic pressure
3. Inadequate lateral stability of structures with one-sided soil pressure
4. Corrosion of piles and sheet piles in the splash zone
5. Lack of waterstops in foundation walls and pressure slabs

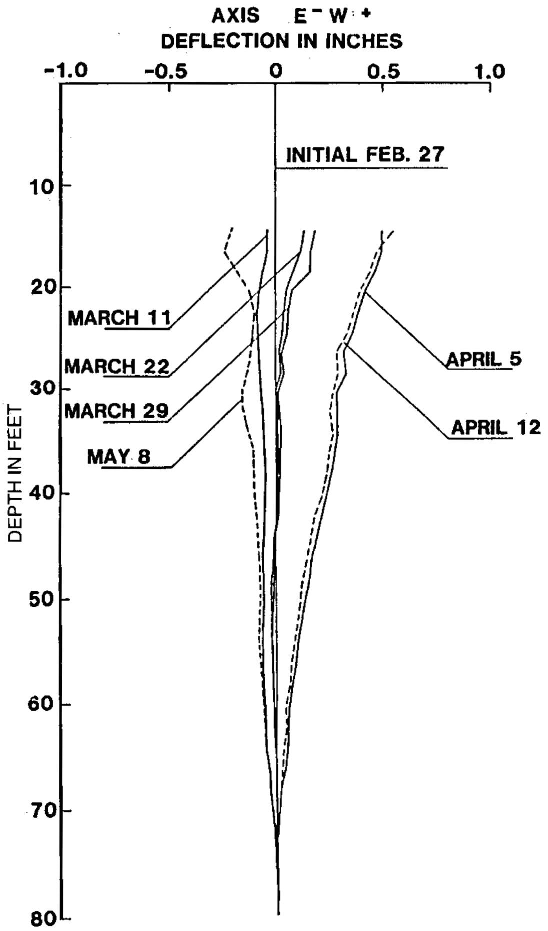


FIGURE 8.83 Inclinometer readings of caissons.

6. Ignoring negative friction effect on piles in unconsolidated soils
7. Lack of piles to support slabs on grade where soils are of poor bearing capacity
8. Insufficient or no borings
9. Borings not taken deep enough
10. No consideration of ground-water level
11. Failure to design for uniform settlement
12. Backfilling before foundation walls are adequately braced by floor framing
13. Inadequate depth of foundations subjected to scouring
14. Compacting upper 10 in (25.4 mm) layer of loose, unconsolidated soil and fills

8.14 CHECKLIST

1. Visit site to observe existing structures which may be affected by new construction. Find out type of foundation used for nearby existing buildings.
2. Take proper borings of sufficient number and adequate depth. Verify ground-water table.
3. Choose appropriate type of foundation. Consult geotechnical engineer.
4. Design for uniform bearing and settlement or provide for any differential settlements which may occur between parts of the structure, by use of expansion joints.
5. Brace foundation walls before backfilling. Do not load unbraced foundations with material or equipment surcharge.
6. Provide drainage at retaining or basement walls.
8. Verify that actual soil is as assumed in design, both for vertical support and for lateral load against walls below grade.
8. Check for effect that pumping and/or vibratory equipment may have on existing structures.
9. Design foundation for all directional loads and within allowable stresses and settlement tolerances.
10. Support slabs on piles when slabs are bearing on compressible or otherwise inadequate soils.
11. Provide step-by-step construction procedure when temporary support, underpinning, shoring, or bracing is required during installation of new foundation.
12. Tremie placing of concrete should always be performed in still water. Do not pump water out of the shell during placement operations.
13. For drilled pier and caisson installations, extract full-depth concrete test cores from the first five to verify integrity of the concrete and adequacy of bottom bearing.
14. Monitor ground-water level during construction, especially if pumping is required, since it may affect existing buildings and the bearing capacity for new structures.
15. Establish vibration criteria for effect on existing structures during pile-driving operations. Special care is required for old and historical landmark buildings.
16. Provide batter piles to resist lateral loads.
17. Avoid bending moments and eccentricities on footings.
18. Check pile caps for eccentricities of as-driven pile locations.

8.15 REFERENCES

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2. Building Code of the City of New York, Sec. 26-1903.1(b), Department of General Services, New York, May 1990.
3. A. Koral, “When the Earth Opens and Walls Move,” *The New York Times,* July 3, 1977.
4. F. J. Crandell, “Ground Vibration Due to Blasting,” Liberty Mutual Insurance Company.

P • A • R • T • 9

MISCELLANEOUS CONCERNS

SECTION 9A
FORENSIC ENGINEERING

ROBERT WADE BROWN
TOM WITHERSPOON

9A.1 INTRODUCTION

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

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

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

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

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

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

*American Society of Forensic Engineers.

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

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

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

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

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

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

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

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

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

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

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

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

TABLE 9A.1 Engineers' Pier Specifications

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

*Too few reports included to be statistically significant.

9A.2 ENGINEERS' SPECIFICATIONS

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

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

TABLE 9A.2 Soil Bearing Capacity Versus Underpinning Design

Assumptions:

Structural load on perimeter beam

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

Foundation construction

Beam: 24″ deep (20″ net) × 10″ wide

2 #4's top and bottom (3″ cover).

Slab: 4″ thick

#3's 18″ OCBW

Soil Characteristics

γ = 120#/ft3

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

PI > 15

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

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

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

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

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

(continued)

TABLE 9A.2 *(continued)*

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

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

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

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

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

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

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

9A.3 SUMMARY OF TABLE 9A.1

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

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

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

9A.4 BEARING CAPACITY OF VARIOUS UNDERPINS

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

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

9A.5 CONCLUSIONS

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

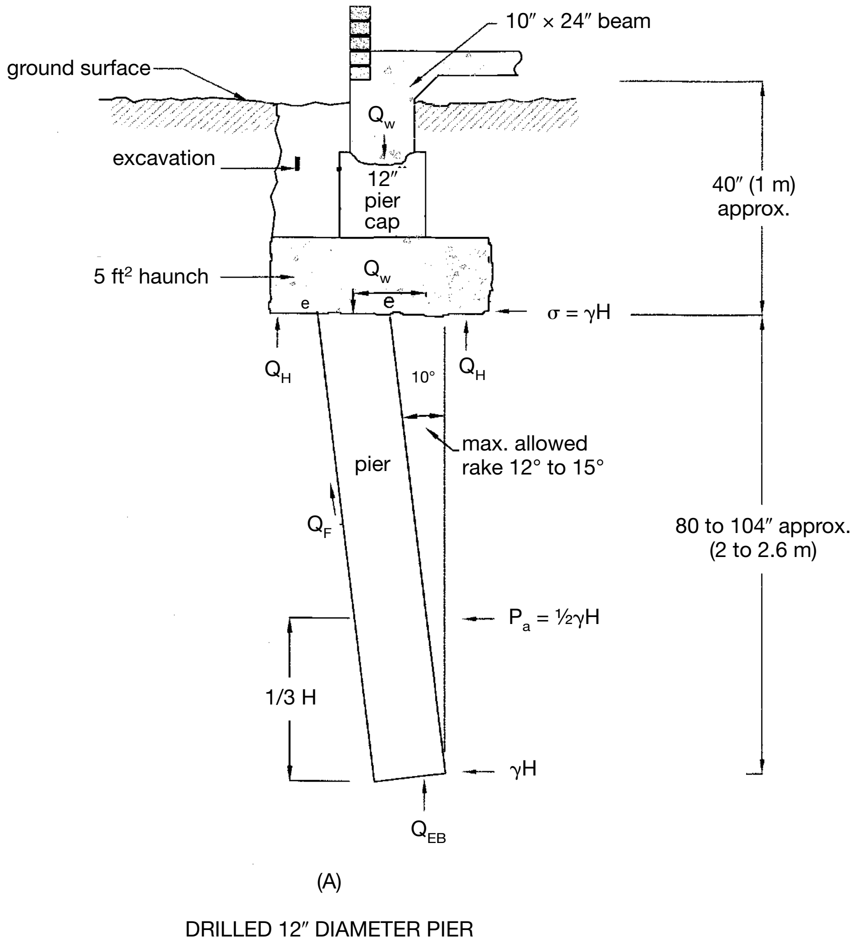
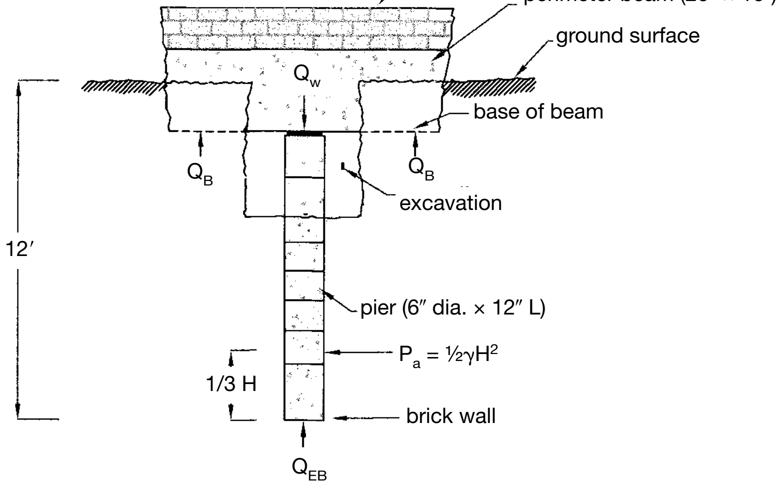


FIGURE 9A.1 Load capacity of selected underpins.

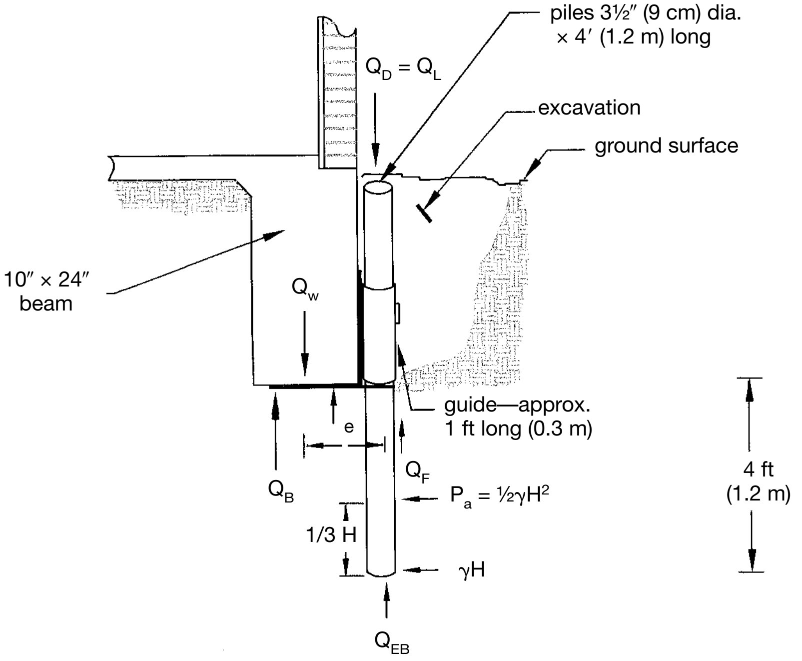
Aside form the aforementioned dilemma in dealing with repair methods, these engineers find, almost without exception, that sewer leaks never causes foundation problems. (Heave is a casualty loss generally covered by the home owners insurance.) Rather, they tend to blame tree roots or "natural doming."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

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



(B)

PRESSED PILE



(C)

ECCENTRICALLY DRIVEN PILES

FIGURE 9A.1 *(continued)*

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

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

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

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

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

Table 9A.3 Easy References for Controversial Topics

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

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

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

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

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

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

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

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

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

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

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

(continued)

Table 9A.3 *(continued)*

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

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

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

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

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

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

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

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

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

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

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

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

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

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

(continued)

Table 9A.3 *(continued)*

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

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

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

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

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

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

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

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

No doubt—maintaining soil moisture will abate foundation movement.

Tree Roots:

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

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

Natural Doming:

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

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

(continued)

Table 9A.3 *(continued)*

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

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

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

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

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

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

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

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

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

Tree Roots:

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

Upheaval:

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

(continued)

Table 9A.3 *(continued)*

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

Moisture Barrier:

7) "A horizontal barrier does little to prevent or lessen the moisture build-up beneath a foundation over a time span of several years." (p. 262)

8) "Vertical barriers tend to reduce first year or so; however, after 4 to 5 years the results become nearly the same with or without the barrier." (p. 262)

Lime Slurry Pressure Injection (LSPI)

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

Settlement:

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

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

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

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

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

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

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

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

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

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

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

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

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

(continued)

Table 9A.3 *(continued)*

Trees:

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

Intrusive Damage to Slabs:

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

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

Sewer Leaks versus Heave:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

(continued)

Table 9A.3 *(continued)*

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

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

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

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

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

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

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

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

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

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

G. Settlement (foundation):
- I. Chen

(continued)

Table 9A.4 *(continued)*

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

9A.6 APPENDIX

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

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

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

(continued)

TABLE 9A.A *(continued)*

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

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

SECTION 9B
HUD

RICHARD SAZINSKI

9B.1 INTRODUCTION

The U.S. Department of Housing and Urban Development (HUD) covers the United States with numerous field offices (Figure 9.B.1). One or more of these offices may be located in each state. The various HUD field offices apply, monitor, or oversee the varied HUD programs, pursuant to Congress's allocation of funds, through appropriate HUD Handbooks and procedures.

The United States Congress has mandated that HUD create conditions for every family to have decent and affordable housing. Numerous HUD programs[1] and activities may, in part, involve new or rehabilitation design and construction of several types of foundation systems for varied types of structures and projects. The current and future long-term stability of any new or rehabilitated foundation system, in any HUD program area, is critical to the economic interest of the American taxpayer and necessary to assure occupant safety.

Several HUD programs discussed in this section that are involved with foundation design, construction, repair, or rehabilitation include single-family (SF) homes, multifamily (MF) projects (new or rehabilitated), existing SF properties/MF projects, policy development and research, HUD's Technical Suitability of Products Program, affordable housing (via lower-cost foundations), and manufactured housing (mobile homes).

The views presented in the HUD part of this handbook are those of the author as an individual and do not necessarily represent the views of HUD.

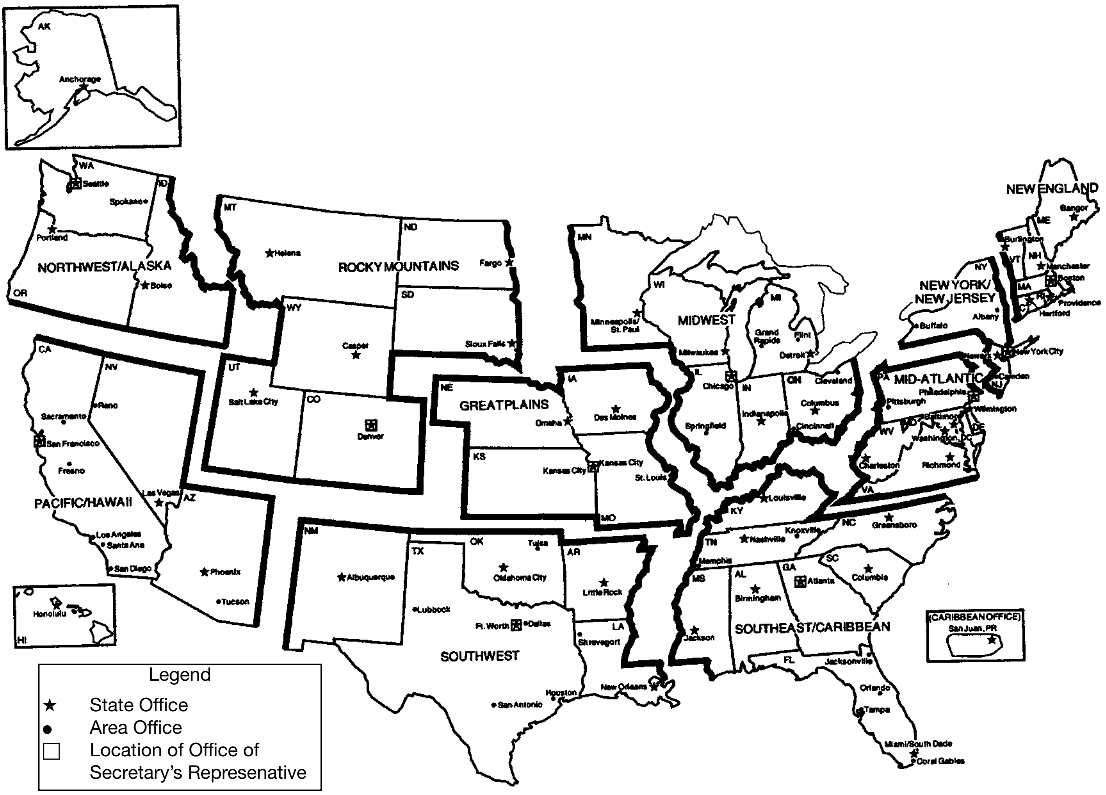


FIGURE 9B.1 HUD field offices.

9B.2 SCOPE

Numerous HUD programs assist borrowers in acquiring SF homes or are used for developing MF projects[1]. Such homes and projects are planned, located, and constructed to serve the present and predictable needs of tenants and to encourage owners in the maintenance and preservation of their investment. HUD may provide mortgage insurance for a loan obtained from a financial institution to purchase an SF property or finance an MF project. Acceptable risk to HUD's insurance fund (mortgage insurance) can be defined where a mortgage is secured by a property or project that meets the requirements of HUD's Minimum Property Standards (MPS)[2]. The strength of an owner's desire to retain ownership will depend largely upon continuing satisfaction with a property or project. Dissatisfaction arising from any cause, including foundation-related problems, lessens motivating interest in ownership and increases mortgage risk.

HUD relies on local codes or national model codes, in addition to standards listed in HUD's MPS for Housing,[2] as the basis for defining acceptable foundation standards and foundation design and construction practices. National model codes include the Council of American Building Officials (CABO) One and Two Family Dwelling Code, the Uniform Building Code (UBC), the Building Officials and Code Administrators (BOCA) Basic/National Building Code, and the Standard Building Code (SBC). HUD typically requires that foundation design and construction, as well as reports or repair methods related to foundations, be designed and completed by qualified professionals, that is, licensed architects, engineers, or contractors. Certifications from qualified professionals may also be required by HUD as a condition of mortgage reinsurance. Usually, completed foundation construction carries a one-year workmanship and materials warranty. For any foundation-related repairs, corrections should be permanent in nature and commensurate with the value and type of property or project.

HUD SF properties or MF projects are involved in nearly the full range of foundation types, design, construction, inspections, and rehabilitation that have been covered in this handbook. HUD SF and MF areas involving foundations are further discussed in the sections that follow and cover new (proposed) construction, rehabilitation construction, or existing construction.

9B.2.1 Single-Family Units

SF units can be detached, semidetached, duplex, or row houses, either site-built or fully or partially factory manufactured. For example purposes, this section deals with typical site-built, detached SF homes covered by FHA mortgage insurance.

9B.2.1.1 New (Proposed) SF Construction

New (proposed) construction of SF units must comply with HUD's MPS for One and Two-Family Dwellings (Appendix K of Reference 2). HUD currently utilizes, wherever possible, acceptable state or local codes as a portion of its MPS for any particular property.

Should no acceptable state or local code be available in an area, HUD will utilize the CABO One and Two Family Dwelling Code. HUD field offices maintain lists of jurisdictions and their appropriate codes, which HUD utilizes as part of its MPS. HUD enforces and interprets codes for its own purposes and not for any local entity.

The CABO code, or any other acceptable state or local code, will address foundation systems. Any foundation system section of a code used by HUD as part of its MPS must address foundation depths, footings, and foundation materials criteria. Such codes also typically reference acceptable engineering practices and standards relative to foundations constructed of varying types of materials and methods, such as brick, masonry, treated wood, and concrete. HUD also defines acceptable material standards and engineering practices related to foundations (Appendices C, E, and F of Reference 2).

HUD SF processing procedures for new proposed construction are defined in HUD Handbook 4145.1 REV-2.[3] These procedures define exhibits that must be submitted for each unit. These exhibits include a separate foundation plan and technical reports and other exhibits when the mortgage risk could be affected by unstable soil or other differential ground movement. Such reports or exhibits might include, but are not limited to, an engineer's report on soil exploration and testing along with special foundation designs for conditions found.

At times, HUD may receive complaints from homeowners relative to foundation problems. If the property is covered by one of the typical 10-year structural warranty policies offered by various entities, HUD will rely on said warranty company as the first avenue of resolution (Appendix 10 of Reference 3). If no assistance is available from one of these warranty companies, HUD does have a process available to assist homeowners in correcting structural defects.[4] Under Section 518(a) of the National Housing Act, HUD can assist in the repair of structural defects reported within the first four years of the issuance of mortgage insurance. Problems with foundations can possibly qualify for assistance if deemed to be the builder's responsibility and the builder refuses to repair or rectify the situation. HUD may contract for repairs, reimburse the homeowner for completed repairs, or buy back the property from the homeowner.

9B.2.1.2 Rehabilitation of SF Construction

HUD promotes and facilitates the restoration and preservation of the nation's existing SF housing stock. One avenue provided by HUD in which a property can be acquired and rehabilitated is via

Section 203(k) of the National Housing Act. There is a minimum $5000 requirement for the cost estimate of eligible rehabilitation or improvement items. As in other SF mortgage insurance programs, a Section 203(k) mortgage is funded by a HUD approved lender, and the mortgage is insured by HUD.

The procedures of the 203(k) program are detailed in HUD Handbook 4240.4 REV-2.[5] Under this program, one of the architectural exhibits required by HUD to be submitted is an inspection report from a qualified architectural, engineering or home inspection service defining the adequacy or required upgrading of existing systems, such as heating, plumbing, and foundation. Homes that have been demolished, or will be razed as part of the rehabilitation work, are eligible, provided the existing foundation system is not affected and will still be used. The complete foundation system must remain in place. A report from a licensed structural engineer is required stating that the existing foundation is structurally sound and capable of supporting the proposed dwelling construction.

Other foundation structural alterations and reconstruction, if necessary, could also be covered under a 203(k) mortgage in the form of foundation rehabilitation. A guidebook produced by HUD[6] addresses SF foundation rehabilitation areas that could qualify as eligible rehabilitation items under a Section 203(k) mortgage. The guidebook covers topics from the design and engineering of rehabilitated foundation systems to shoring and repair, waterproofing, crack repair, drainage, and insulation. New foundation elements for new additions could also qualify as eligible items under a Section 203(k) mortgage. Any new foundation construction must conform with local or state codes and HUD's MPS (Appendix K of Reference 2). HUD Handbook 4905.1 REV-1,[7] which addresses acceptable conditions of existing units, applies in part to this program when defective conditions, including foundation problems, must be rectified (see Section 9B.2.1.3 for further discussion).

9B.2.1.3 Existing SF Construction

HUD requirements for the quality of existing SF properties, including technical acceptability, are defined in HUD Handbook 4905.1 REV-1.[7] Existing SF units with defective conditions are unacceptable for involvement in normal HUD program areas. Such conditions include defective construction, evidence of continuing settlement, or other conditions impairing the structural soundness of the dwelling. Such defective conditions shall render the property unacceptable until the defects or conditions have been remedied and the probability of further damage eliminated. Also, properties shall be free of hazards that may adversely affect the structural soundness of the improvements.

HUD's treatment of existing properties is also outlined in HUD Handbook 4150.1 REV-1.[8] HUD technical personnel, either HUD staff or fee staff (architects, engineers, construction analysts, and so on), are usually not involved in an inspection of an existing SF property for determining acceptability, unless specifically requested. Usually, a HUD approved fee appraiser will determine the value of a property for FHA mortgage insurance purposes. In addition to providing an estimate of value, the appraiser inspects the property for any visible deficiencies that may affect the health and safety of the occupants or the continued marketability of the property. HUD makes no warranties as to the value or condition of the house. Therefore, the borrower must determine that the price of the property is reasonable and that its condition is acceptable. The borrower is also urged to hire a home inspection service to check out a property for adequacy.

The condition of existing building improvements is examined at the time of appraisal to determine whether repairs, alterations, or additions are necessary. When examination of existing construction reveals noncompliance with the general acceptability criteria defined in HUD Handbook 4905.1 REV-1,[7] the appraiser must define an appropriate specific correction of the deficiency, if correction is feasible. A typical condition requiring repair, which would be noted by the appraiser, might involve visible foundation damage or damage related to foundation problems. Usually, the appraiser may either define the required repairs, or, more commonly, the appraiser will request an inspection by a licensed or registered structural engineer defining the cause and cure of any foundation-related problem. Any cures defined by the structural engineer may also be required to be inspected and cleared by the same structural engineer as to adequacy of completion.

9B.2.2 Multifamily Projects

MF projects can involve buildings designed and used for normal MF occupancy, including both unsubsidized and subsidized insured housing. Buildings with five or more living units are typically involved.

9B.2.2.1 New (Proposed) MF Construction

MF projects involved in HUD programs[1] cover a wide range of foundations and structures (single story, stick-built to multistory, high-rise concrete). Such structures typically must comply with HUD's MPS for Housing,[2] along with a partially or fully acceptable state or local building code. If no partially or fully acceptable state or local code is in force or acceptable, a recognized model code must be utilized (such as UBC, BOCA, or SBC). HUD field offices maintain lists of jurisdictions with typically utilized or accepted codes for MF HUD project development. HUD enforces and interprets codes for its own purposes and not for any local entity.

Any acceptable state or local code, as well as any recognized model code, covers foundation systems. Foundation system code sections must cover soil tests, foundation depths, footings, foundation materials criteria, piles (materials, allowable stresses, design) and excavations. Codes also will reference acceptable engineering practices and standards relative to foundations which are constructed of varying types of materials and methods (brick, masonry, treated wood, concrete, and so on). HUD also provides similar or additional listings of foundation-related acceptable material standards and engineering practices (Appendices C, E, and F of Reference 2).

HUD requires that reliable subsurface exploration information be available (soils reports, test boring logs, test pit data, soil bearing values, geotechnical study, and so on).[9] Usually soils reports will recommend foundation types along with design parameters for HUD MF projects. Also, HUD requires MF project structural foundation plans to be stamped by a registered architect or professional engineer.[9]

9B.2.2.2 Rehabilitation of MF Construction

MF-type dwelling structures have always been, and probably will continue to be for the foreseeable future, a major source of housing for lower and moderate-income families. Over the years, because of economic conditions and higher operating costs, many of these dwelling units have been neglected, and some are seriously deteriorated. Typical HUD procedures for MF rehabilitation are defined in HUD Handbook 4460.1 REV-1 (refer to Chapter 4 of Reference 9).

A building should undergo a preliminary examination in order to clarify whether or not it is generally suitable for rehabilitation. After said preliminary examination, HUD may require an engineering report to examine major building components, such as the foundation system, to define any upgrading or replacement to assure structural soundness.

The services of a state licensed or registered engineer will be required by HUD when the design and construction of an MF rehabilitation project necessitates plans and specifications to properly define the scope and concept of the rehabilitation if structural changes are necessary or an addition is proposed to an existing building. Rehabilitation must comply with applicable local codes and ordinances. All new construction or additions that enlarge existing buildings must meet applicable codes and HUD's MPS for new construction. (Refer to Section 9B.2.2.1 and Reference 2). Typical types of foundation repairs or new foundation elements involved have been covered in this handbook.

Examples of HUD MF projects, involving foundation rehabilitation are briefly summarized as follows[10]:

1. Minerva Place Apartments (St. Louis, MO/FHA #085-25-339) Constructed in 1942 as a two-story YMCA with a swimming pool, it was converted to a 56-unit apartment complex with a new third floor added. A subsurface soil investigation, which determined the soil bearing capacity, required installation of new larger footings.

2. Douglas Manor Apartments (Webster Groves,MO/FHA #085-35-350) Constructed in 1945 as a two-story public elementary school with classrooms and gymnasium, it was converted to a 41-unit apartment building. The critical element in the structure was found to be the allowable soil bearing under existing column spread footings. Soil analysis indicated that the shear strengths of the soil were low and erratic, and the allowable bearing capacity would be less than the load applied to the soil due to the installation of additional floors. Therefore the 12 columns supporting the gymnasium were shored so that the spreadfootings could be underpinned and enlarged for an allowable soil pressure of 2700 lb/ft^2 (13,200 kg/m^2).
3. Patterson Place (Bismarck, ND/Proj. #094-35040-LD-SR-L8) Constructed in 1910 as a six-story hotel with approximately 100 rooms, over the years (around the 1930s) four more floors were added to extend the structure to a 10-story height. The proposed rehabilitation consisted of converting the hotel into 117 apartments for the elderly. Because of the addition of four more floors, the foundation and columns became underdesigned. To reduce the loads on column footing pads, the rehabilitation required the removal of 2 to 4 in (0.8 to 1.6 cm) thick concrete floor toppings, nonbearing concrete partitions, and heavy plastered ceilings. Steel stud drywall partition walls were installed, and overloaded columns from the basement to the fifth floor were upgraded.

9B.2.2.3 Existing MF Construction

HUD offices nationwide are involved with a large inventory of varying types of existing MF structures (wood frame, reinforced concrete, steel, and so on) originally constructed under various HUD programs [such as 221(d)(4) insured apartments, 202 elderly, or public housing].[1] At any time, an MF structure might develop a foundation problem necessitating action. Such foundation problems may surface during an inspection by HUD staff, or, they may be reported by project personnel or by tenants themselves. A report from a registered or licensed structural engineer, defining causes and cures, may be requested. Any rehabilitation (cure) or retrofit of foundation elements could be accomplished under various HUD programs, depending on the type of MF project.

As an example, a project known as Jackson Heights Elderly Highrise (Proj. #SD-045-001W), constructed in 1978 in Rapid City, SD, started to exhibit stress conditions on several interior wall surfaces in 1984/85 (cracked drywall, door frame racking, and so on). A structural engineer and a soils consultant were employed to define causes and cures. The structure was six stories in height, consisted of 106 units, and was constructed of standard concrete columns bearing on isolated concrete footing pads. A drastic change in the moisture content of a subsurface bearing strata layer caused an approximate 2 in (5 cm) differential settlement between adjacent reinforced concrete columns and pads spaced from 17 ft to 20 ft (5.2 to 6.1 m) apart. 12 isolated concrete pad footings of one wing were stabilized via compaction grouting. An experienced contractor completed the necessary repairs. For a discussion of compaction grouting, see Section 6B.

9B.3 POLICY DEVELOPMENT AND RESEARCH

To carry out presidential and congressional mandates in the areas of housing, community development, and fair housing efficiently and effectively, HUD is structured so that research, economic and policy analyses, and program evaluations are the responsibility of the Assistant Secretary for Policy Development and Research (PD&R) in Washington, DC. All research activities are centralized in PD&R. The research data HUD uses in policy development are made available to interested parties such as state and local governments, financial institutions, builders, developers, universities, and colleges.

The research program of PD&R has focused, in part, on granting research contracts to various groups to investigate and report on hazards to housing (expansive soils, lead-based paint, radon gas, and so on). Relative to foundation systems, work carried out under research contracts is usually based on developments in applied structural and geotechnical theory. Cooperative efforts of research institutions (colleges, universities, and so on), contractors, practicing engineers, and others involved in such research contracts have produced a better understanding of the factors that contribute to foundation performance. Through HUD's PD&R efforts, a guidebook was produced as a rehabilitation guide for SF foundations.[6] (Refer to Section 9B.2.1.2). Examples of HUD funded research involving foundation systems includes a study of remedial measures for houses damaged by expansive clay,[11] as well as a study of experimental residential foundation designs for use on expansive clay soils.[12]

9B.3.1 Remedial Measures for Houses Damaged by Expansive Clay

One such research contract investigated certain remedial techniques that could be used to restore house foundations damaged by expansive clay soils.[11] All techniques were directed towards stabilizing a mass of soil beneath house floor slabs to a depth of 5 ft (1.5 m) or more. The 10 houses included in the investigation were HUD-owned properties identified by the HUD Dallas Office, and all were severely distressed by vertical movement and volume change of the soil. Data acquisition continued over approximately 2 years, or four seasonal climatic cycles.

The remedial techniques included lime slurry pressure injection, a subsurface irrigation system, and three types of moisture migration barriers (granular, recycled rubber, and lean concrete). The lime slurry injection process, utilized around the foundation perimeters to stabilize the soil mass and inhibit moisture migration, was very costly and not very effective. The subsurface irrigation system was not completely effective, due to additional climatic effects and difficulty in keeping the mechanical components of the system active. All three of the moisture migration barriers (granular, recycled rubber, and lean concrete) appeared to be viable techniques when accompanied by increasing the foundation soil moisture 2 to 3% above the plastic limit. These barriers, approximately 5 ft (1.5 m) deep and the width of trenching equipment, isolated a mass of soil beneath the floor slab and caused the slab and soil to interact as a layered system. Assuming that a stable equilibrium condition is realized after water infusion, cosmetic repairs could be effected both inside and outside the damaged structure and the foundation restored to a usable condition.

9B.3.2 Experimental Residential Foundation Designs on Expansive Clay Soils

Preventive measures, applied at the time of construction, were also studied under a research contract.[12] These have also been discussed in Section 7C. The study involved 11 experimental homes, each with a different type of foundation system, constructed over expansive clay soils in Grand Prairie, TX. The performance of the homes was monitored over a 3 year period, or six seasonal cycles. Foundation systems utilized various reinforced concrete, posttensioning, moisture barrier stabilization techniques, and subsurface irrigation systems.

The typical reinforced concrete systems (BRAB waffle slab design) performed adequately, yet exhibited a prohibitively high cost. Posttensioned systems exhibited reasonably effective performance (See Section 9B.5.4 and References 13–15 for further discussion). Active subsurface irrigation systems proved not to be justified because of indeterminate effects of interruptions of water service, tampering with controls, and concerns for the longevity and durability of any buried pipe. Perimeter passive vertical moisture barrier stabilization techniques were deemed viable, provided the soils encompassed by any barrier were preswelled and moisture was contained. A futher discussion of the prolonged behavior of all 11 expirmental homes was published by Robert Wade Brown in *Design and Repair of Residential Foundations,* McGraw-Hill, New York, 1990.

9B.4 TECHNICAL SUITABILITY OF PRODUCTS PROGRAM

Section 521 of the National Housing Act of 1965 directs HUD to adopt a uniform procedure for the acceptance of materials and products to be used under HUD housing programs. To comply with this mandate, HUD developed the Technical Suitability of Products Program for review and acceptance of building systems, components, products, and materials. The objectives of this program are the acceptance of new and innovative building materials and systems, and to encourage improvements in, and development of, technological advances in home building.

The program and processing procedures are outlined in HUD Handbook 4950.1 REV-3.[16] Under this program, one such type of acceptance document that can be issued by HUD is a structural engineering bulletin (SEB). An SEB can be issued to accept complex components that have structural features not addressed by, or not in compliance with, HUD requirements for new construction. A floor, wall, or roof system can be covered by an SEB issued by HUD headquarters.

Wall systems can, in part, cover foundation systems. An example is SEB #1117, issued to Superior Walls of America, Ltd., for their precast concrete stud wall panels utilized in SF home basement construction. The wall panels, in themselves, are designed to act as a reinforced concrete monolithic concrete panel with a footing (Figure 9B.2). Over 10,000 Superior Walls foundations have been installed throughout the northeastern United States. The precast reinforced concrete insulated foundation basement wall panels can be used for dwellings up to 2½ stories (plus basement). Wall panels can be precast in 4 ft (1.2 m), 8 ft (2.4 m), and 10 ft (3 m) heights and up to 20 ft (6 m) wide. The panels consist of reinforced concrete exterior skins, interior studs, and interior insulation. Minimum concrete strength is 5000 psi (34.5 MPa).

The concrete exterior skins are 1¾ in (4.5 cm) thick, monolithic with 10½ in (26.7 cm) top ledger and bottom footing reinforced with 2 #3 horizontal bars. The reinforced concrete interior studs are 2¼ in (5.7 cm) by 6¾ in (17.1 cm) with a #4 vertical bar. Studs are spaced 24 in (60 cm)

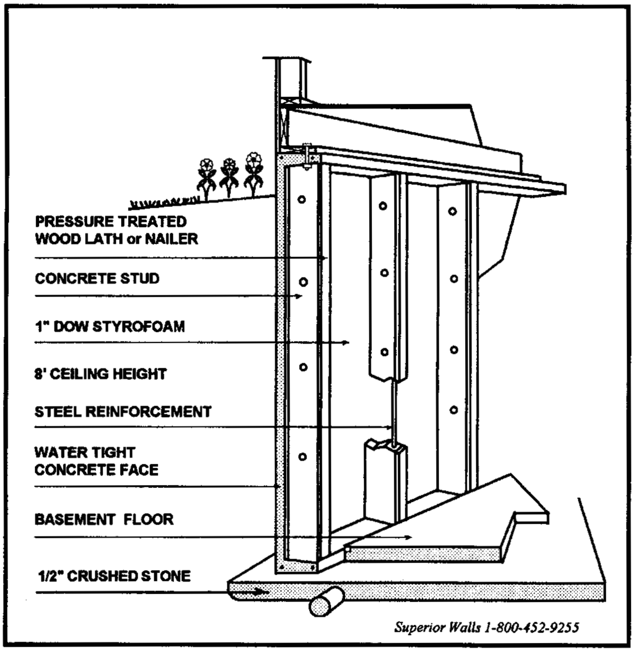


FIGURE 9B.2 Superior Wall and Foundation System.

center to center and are mechanically anchored to the concrete skin, top ledger, and bottom footing. Treated wood nailers are attached to the interior face of the studs. Basement foundation wall panels have a maximum allowable load (earth pressure) of 60 lb/ft^3 (960 kg/m^3) equivalent fluid pressure and a maximum allowable vertical load (building structure plus live loads) of 4200 lb/lin ft (6250 kg/m).

9B.5 LOWER-COST FOUNDATIONS

Homes that are safe, durable, marketable, and affordable must be provided to meet national housing goals. In January, 1982, HUD announced the formation of the Joint Venture for Affordable Housing (JVAH) as a public–private partnership to reduce housing costs. Over the years, several HUD JVAH demonstration projects have utilized lower-cost structural foundation systems to assist in reducing housing costs.[17,18] Lower-cost foundation systems used in the JVAH effort, as well as other cost-saving foundation systems, are discussed in this section.

9B.5.1 Concrete Footings, Foundation Walls, and Slabs

Footing widths could be reduced if accurate soils data were utilized rather than providing local practice minimum widths. Footing reinforcing can be deleted for footings placed on undisturbed stable soil. Reinforcement in foundation walls is usually not required in nonexpansive soil and in areas of low seismic risk. Under stable base conditions, concrete slab floors do not require typical welded-wire fabric (WWF), concrete slab thickness can be reduced to 2½″ (6 cm), and concrete slab strength can also be reduced to 2000 psi (13.8 MPa). Highly expansive soils may necessitate specialized construction.

9B.5.2 Frost-Protected Shallow Foundations (FPSF)

Perimeter insulation is added along with a shallow foundation system in lieu of a full-frost-depth foundation (Figure 9B.3).[19] Insulation dimensioning is according to design guides of the Norwegian Building Research Institute along with some extrapolation and adjustment for U.S. energy code minima. Corner insulation is typically extrawide due to three-dimensional heat loss. Cost savings have been projected from $1239 to $2875 for insulated shallow foundation systems versus full-frost-depth foundations used in several areas of the country [1612 ft^2 (150 m^2) home, block and concrete foundations].[19]

More recent demonstrations of FPSF technology were completed in 1994 by the National Association of Home Builders (NAHB)/National Research Center (NRC) for HUD.[20] Five FPSF demonstration homes were constructed, monitored, and evaluated for foundation performance and construction costs. One home each was constructed in Vermont, Iowa, and Alaska, with two built in North Dakota; they were built according to European FPSF design guidelines and a proposed simplified design method for adoption by the major model building codes.[21] This demonstration concluded that the homes performed well and that houses and other structures may be built on shallow, slab-on-grade foundations in cold climates when properly insulated to protect against frost heave. Cost savings to a home buyer could range from approximately 1 to 4% of the cost of a conventional slab-on-grade home and even greater when compared to basement construction.

9B.5.3 Permanent Wood Foundation (PWF)

The PWF system (Figure 9B.4),[22] previously identified as the AWWF (All-Weather Wood Foundation), is basically a below-grade stud wall built of pressure-preservative-treated lumber and Ameri-

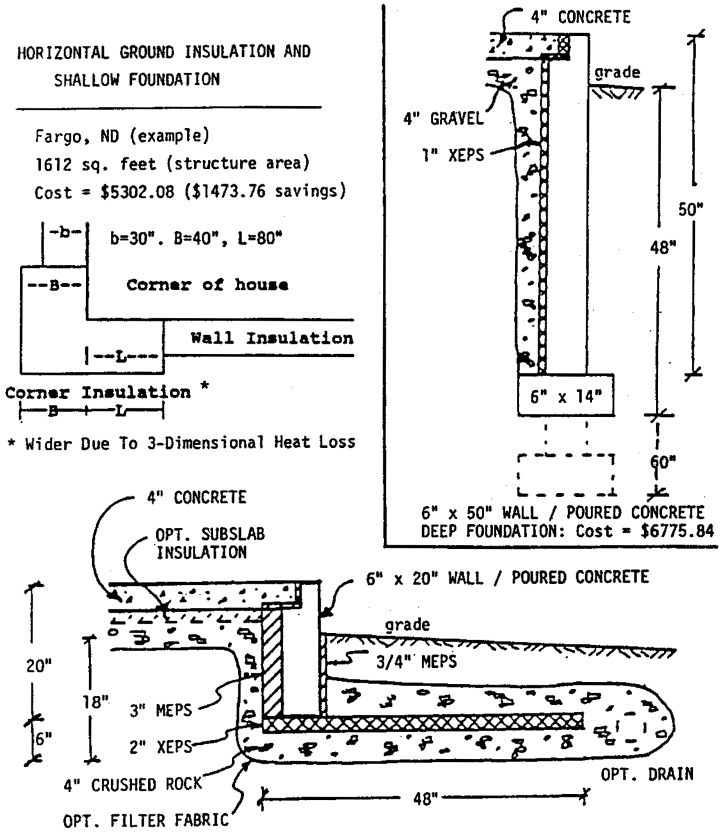


FIGURE 9B.3 Frost-protected shallow foundation and full-frost-depth foundation.

can Plywood Association (APA) trademarked plywood. Basic advantages of the PWF are installation in nearly any weather condition, speed of construction, ease of interior finishing, and versatility. HUD JVAH demonstration projects in Tulsa, OK, and Fairbanks, AK, utilized the PWF system, and cost savings amounted to $1470 and $1035 per unit, respectively.[18] The PWF system is also identified in HUD's rehab guide for foundations[6] as a possible replacement option for damaged sections of existing SF foundation walls.

9B.5.4 Posttensioned Slab-on-Grade

Certain foundation configurations are designed to be relatively insensitive to the fluctuations in soil moisture and the consequent variations in supporting soil volume. Typical foundations depend on stable supporting soil. The use of posttensioned slabs-on-ground is most often specified in locations with highly expansive soils or in other situations where the foundation-supporting material provides uncertain or variable supporting qualities.[13,14]

The system can be utilized for sites with highly expansive or compressible soils where the use of conventional foundations might require extensive site soil modifications (soil removal and replace-

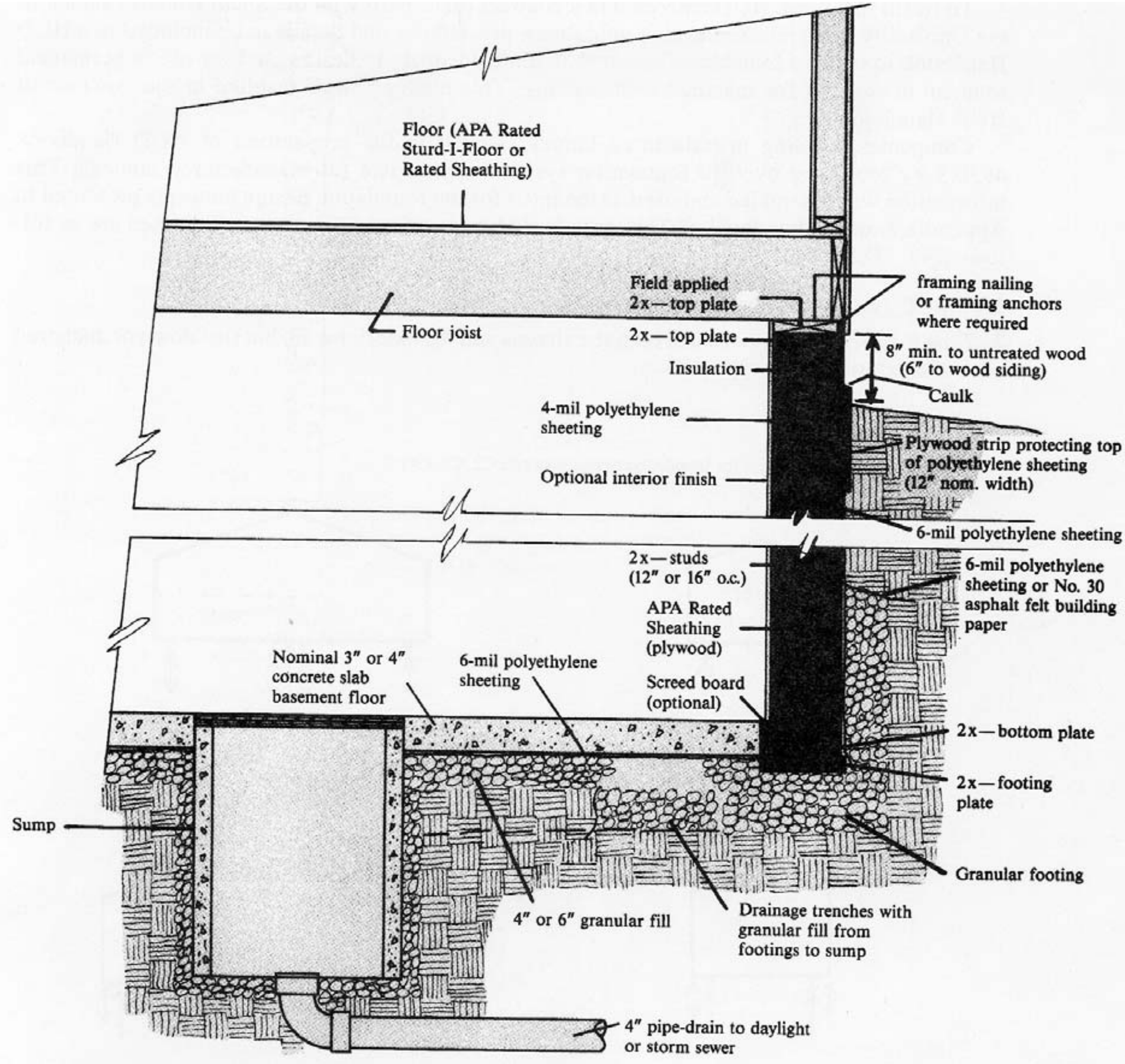


FIGURE 9B.4 Permanent Wood Foundation (PWF) system.

ment). More extensive deep foundation systems otherwise might be required. Costs vary with site soil conditions and building configurations. Part 4 further discusses this type of foundation system.

In 1990, some HUD concerns were expressed relative to these types of systems and their utilization in Texas for units involving FHA mortgage insurance.[15] However, HUD concerns could be acceptably mitigated with assured certifications as to defining proper soil design parameters, following current design procedures, and providing for adequate supervision over construction.[15]

9B.6 MANUFACTURED HOUSING (MOBILE HOME) FOUNDATIONS

In 1983, HUD extended eligibility for typical FHA SF mortgage insurance to manufactured homes constructed in conformance with the Manufactured Home Construction and Safety Standards (MHCSS), when permanently attached to a site-built foundation. Neither HUD's MPS nor the Manual of Acceptable Practices (MAP) adequately covered special requirements for permanent foundations for manufactured housing. Due to this inadequacy, HUD headquarters realized an urgent need to furnish HUD field offices with technical guidance concerning acceptance of permanent site-built foundations for manufactured housing if they are to be covered by typical 30-year FHA SF mortgage insurance.

To fulfill this need, HUD entered into a contract with the Small Homes Council of the University of Illinois to develop guidelines, procedures, and details to be included in a HUD Handbook to contain foundation concepts that are adequate in design, and for use in permanent foundation systems for manufactured housing. This contract work resulted in the 1989 issuance of HUD Handbook 4930.3, Permanent Foundations Guide for Manufactured Housing.

In 1996, the Building Research Council of the School of Architecture at the University of Illinois at Urbana/Champaign, under contract with HUD, updated and revised the 1989 version of HUD Handbook 4930.3.[23] Whereas wind alone governed the information on overturning and sliding in the 1989 handbook, stringent seismic criteria made it necessary to review both wind and seismic forces to determine which should control the foundation design. To account for this significant issue, the tables in the handbook were modified to include seismic data and to highlight those values controlled by seismic considerations.[23]

The foundation design concepts, presented in Appendix A of the Handbook,[23] were condensed from over 40 systems submitted by the manufactured housing industry. The three basic types of foundation systems, for single-section and multisection units, are defined as follows (Figures 9B.5 and 9B.6):

1. Type C: Support and vertical anchorage occurs at equally spaced joints along the chassis beam lines only.
2. Type E: Support occurs at the exterior longitudinal foundation walls as well as at equally spaced points along the chassis beam lines. Vertical anchorage occurs continuously along the exterior longitudinal foundation walls for single-section or multisection units (two ties), or a combination of vertical anchorage can occur continuously along the exterior longitudinal foundation walls and along the equally spaced pier locations along interior chassis beams (four ties).
3. Type I: Support occurs at the exterior longitudinal foundation walls as well as at equally spaced piers along the chassis beam lines, just as for type E, for single-section or multisection units. Vertical anchorage occurs at the equally spaced piers along the chassis beam lines only for single-section or multisection units (two ties or four ties).

Type C1 and E1 systems are depicted in Figures 9B.7 and 9B.8. Once the system is defined, Appendix B of the Handbook[23] presents foundation design load tables for use in determining foundation anchorage and footing sizes for all foundation types. Seismic, wind, and snow loads were computed based on ASCE 7-93, Minimum Design Loads for Buildings and Other Structures. Minimum wind and minimum roof live loads were based on Appendix K of HUD's MPS.[2] Normal ranges of allowable footing bearing pressures are also defined in the Handbook.[23] Foundation capacity tables are presented in Appendix C of the Handbook[23] to determine the sizes and spacing of anchors, required size and depth of footings, and necessary reinforcement. Horizontal and vertical anchorage and withdrawal resistance capacities are defined for foundation systems of reinforced concrete, grouted masonry, isolated piers, and all-weather wood systems (on concrete or gravel) (Figures 9B.9 through 9B.13).

The Handbook is a logically organized easy-to-use reference for the permanent foundation

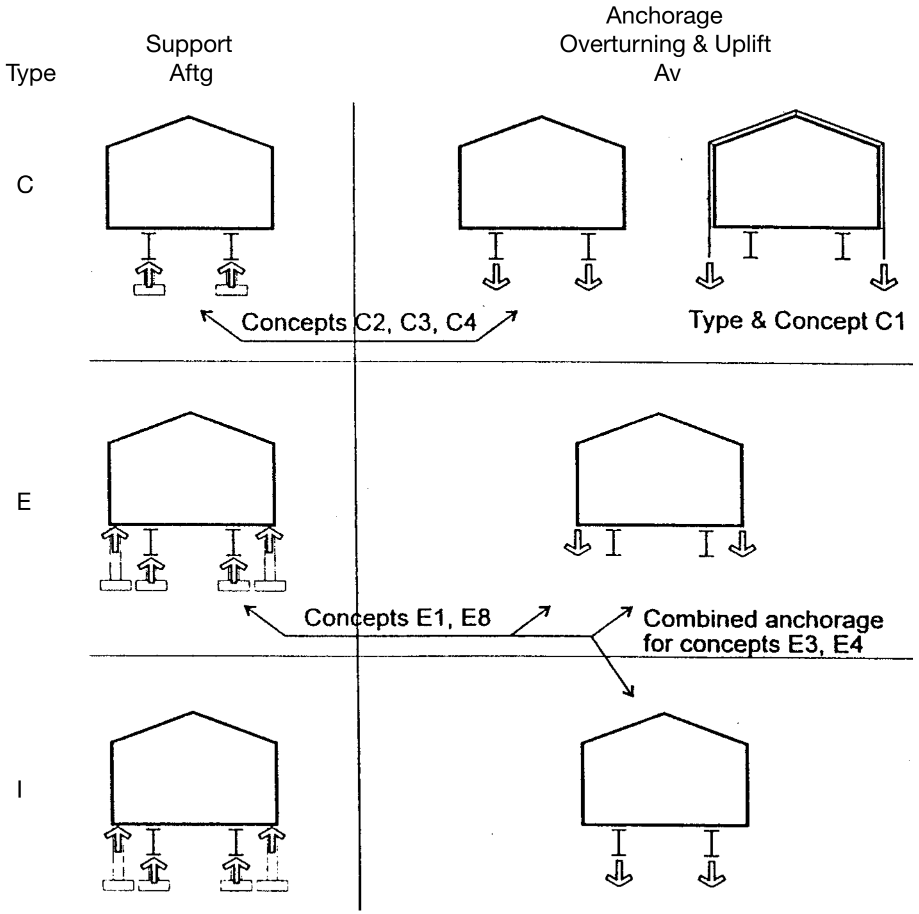


FIGURE 9B.5 Foundation design concepts: single-section units.

process and for the design of anchorages that will assure adequate structural performance for manufactured homes. Companion computer software, and a guide for same,[24] is also available.

9B.7 CONCLUSION

The previous sections have described several HUD program areas involved with foundation design, construction, repair, and rehabilitation. Numerous types of foundation systems and repair techniques are and have been utilized in various HUD program areas. HUD typically relies on foundation design and construction procedures defined in national model codes, acceptable state and local codes, or defined in its MPS for Housing. Policy development and research activities have been sponsored by HUD to study remedial measures for damaged foundations as well as acceptable foundation systems for use over expansive clay soils. HUD's Technical Suitability of Products Program is an avenue that has been used by sponsors to obtain a review and acceptance of foundation-related

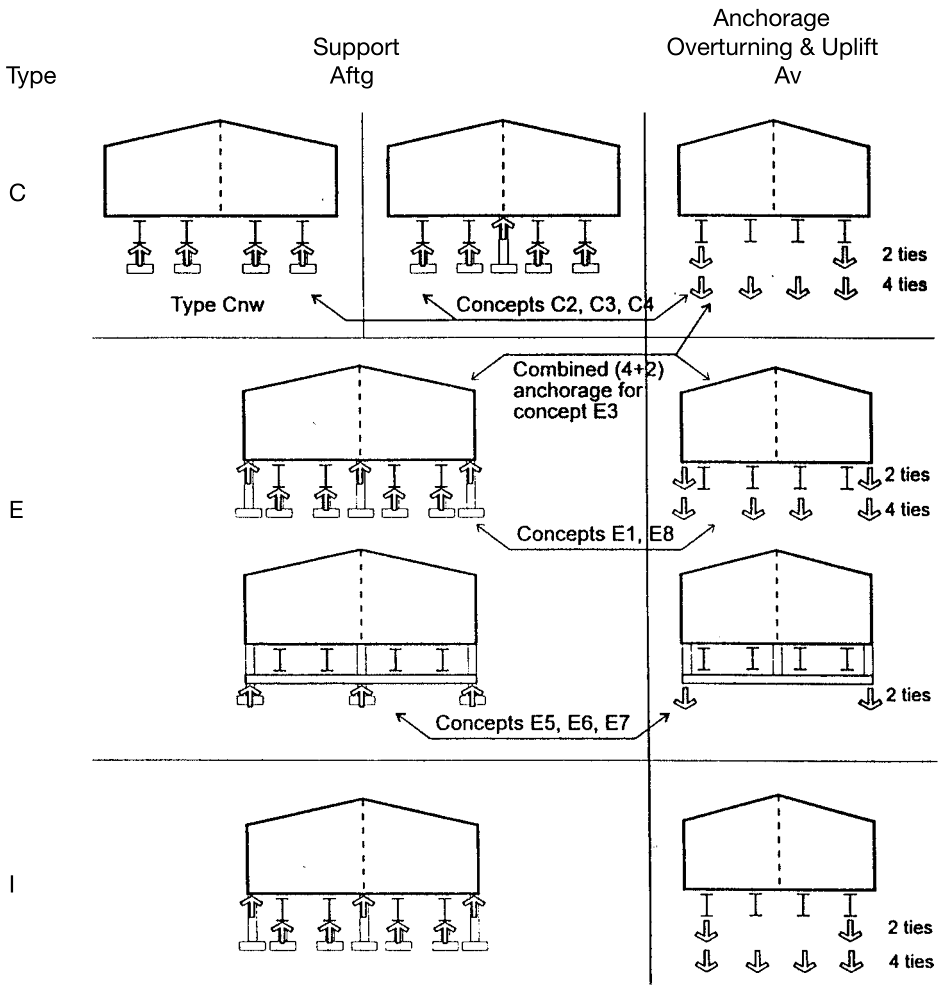


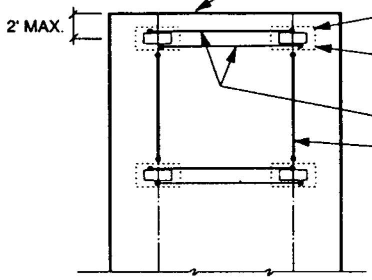
FIGURE 9B.6 Foundation design concepts: multisection units.

housing components or systems. Various types of economically built foundation systems have been shown to contribute to affordable housing in demonstration projects developed either under HUD's Joint Venture for Affordable Housing (JVAH) or under other HUD sponsored efforts. Manufactured housing (mobile home) foundations, considered permanent in nature, have been defined by HUD through cooperative efforts of the industry and the University of Illinois.

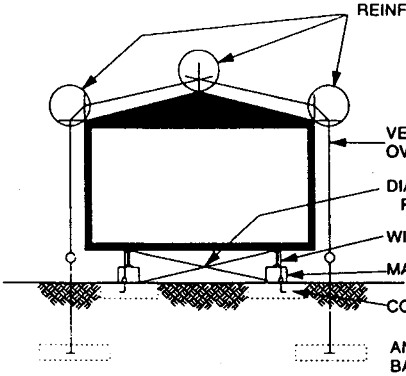
It has not been possible in the course of these brief sections to cover all the types of HUD program areas that may involve foundation design and construction. Omitted, for example, are discussions of public and Indian housing projects, community planning and development grants, and HUD-held properties or projects. However, in such omitted program areas, similar procedures are utilized and are based on the use of acceptable foundation standards, complying with building codes, involving qualified design professionals, and utilizing experienced contractors. Key goals are to assure structural integrity and longevity, occupant safety, durability, protection of HUD's mortgage insurance fund, and the efficient and economical use of Congressionally allocated funds.

FOUNDATION TYPE Reinforced masonry piers w/ wire tie downs and diagonal tie	SYSTEM NUMBER
SUPERSTRUCTURE TYPE Chassis supported single-wide	C1

SINGLE-WIDE



FOUNDATION PLAN

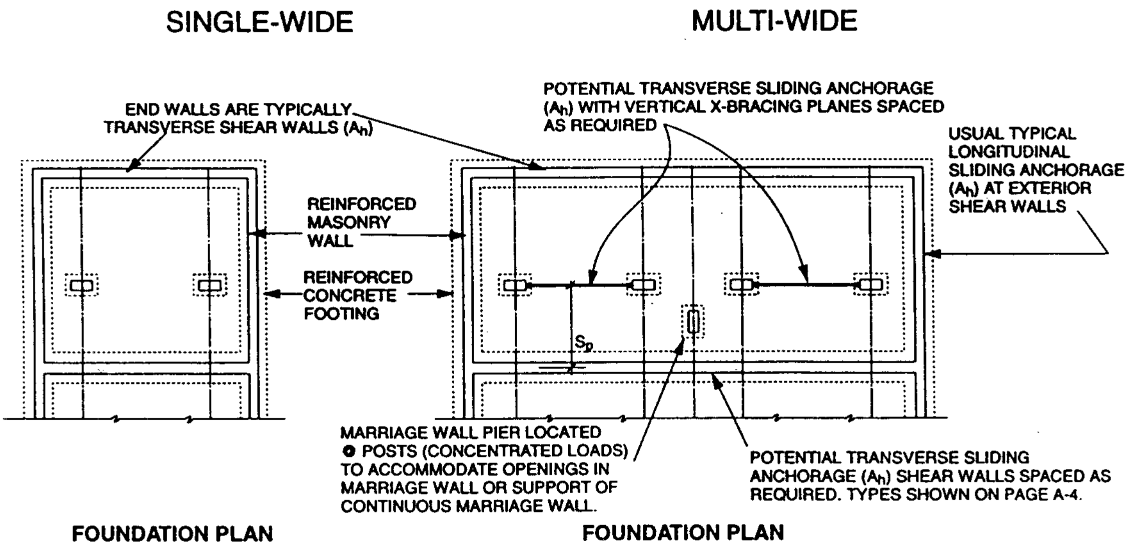


BUILDING SECTION

NOTE: TYPICAL STEEL TIE-DOWN STRAP: 1/32" × 1-1/4"
MINIMUM BREAKING TENSION STRENGTH = 4750 LB (ULTIMATE LOAD)
(ASTM D3953-83) OR
FEDERAL QQ-S-781G

FIGURE 9B.7 Type C1 foundation type.

FOUNDATION TYPE Reinforced perimeter wall, unreinforced piers at chassis	SYSTEM NUMBER
SUPERSTRUCTURE TYPE Exterior anchored, chassis supported single- and multi-wide	E1



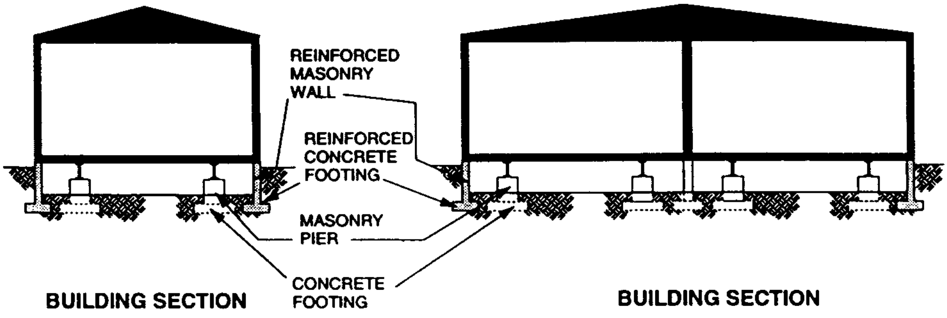
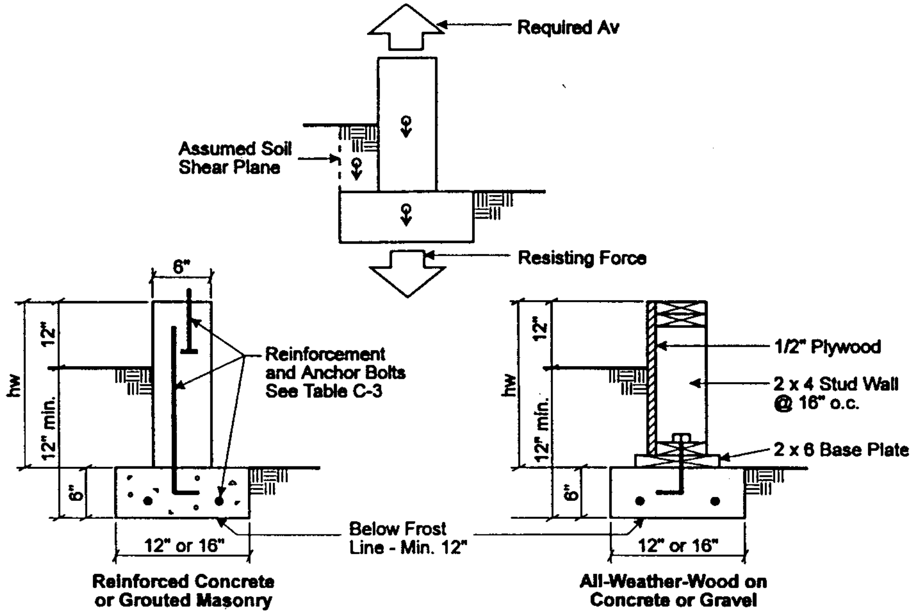


FIGURE 9B.8 Type E1 foundation type.

Table C-1
Withdrawal Resistance[1]
Longitudinal Continuous Foundations[2, 3]
(In pounds per linear foot of wall)



hw	Reinforced Concrete		Masonry-Fully Grouted 6" CMU		Masonry-Grouted @ 48" o.c.		All-Weather Wood w/ Conc. Footing	
	Footing Width		Footing Width		Footing Width		Footing Width	
	12"	16"	12"	16"	12"	16"	12"	16"
2'-0"	255	300	231	276	195	240	126	171
2'-8"	325	383	293	351	245	303	154	212
3'-4"	395	466	355	426	295	366	182	254
4'-0"	465	550	417	502	345	430	211	296
4'-8"	535	633	479	577	395	493	240	337

[1] Potential resistance to withdrawal is the maximum uplift resistance which can be provided by the foundations shown. It is computed by adding the weights of building materials and soil over the top of the footing, plus the footing weight. To fully develop this potential, adequate connections to the footing and superstructure must be provided. Material weights used: concrete (nlwt) = 150 psf; 6"solid grouted CMU = 63 psf; 6" CMU grouted @ 48"o.c. = 45psf; grout wt assumed = 140 pcf; CMU units nlwt; wood = 35 pcf; soil = 120 pcf.

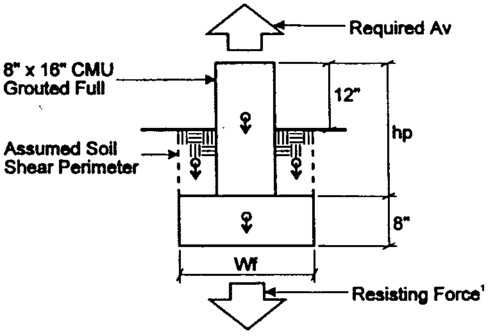
[2] Foundations must be designed for bearing pressure, gravity loads, and uplift loads in addition to meeting the anchorage requirements tabulated in the Foundation Design Tables.

[3] Values shown in this table could be increased by widening the footing, provided the system is designed for the increased load, or by a more detailed analysis of the shearing strength of the soil overburden.

FIGURE 9B.9 Withdrawal resistance.[23]

Table C-2
Withdrawal[1] Resistance For Piers[2, 3]
(In pounds per pier)

Hp Depth	Width of Square Footing: Wf			
	1'-0"[4]	2'-0"	3'-0"	4'-0"[4]
2'-0"	279	997	2097	3755
2'-8"	361	1322	2824	5049
3'-4"	442	1643	3541	6325
4'-0"	525	1967	4267	7617
4'-8"	607	2292	4994	8911



[1] Potential resistance to withdrawal is the maximum uplift resistance which can be provided by the foundations shown. It is computed by adding the weights of building materials and soil over the top of the footing, plus the footing weight. To fully develop this potential, adequate connections to the footing and superstructure must be provided. Material weights used: concrete (nlwt) = 150 psf; nlwt 8"CMU = 84 psf grouted solid; grout (nlwt) = 140 pcf; soil = 120 pcf.

[2] Foundations must be designed for lateral soil pressure, bearing pressure, gravity loads, and uplift loads, in addition to meeting the anchorage requirements tabulated in the Foundation Design Tables. The bottom of the footing must also be below the maximum depth of frost penetration.

[3] Values shown in this table could be increased by widening the footing, providing the wall system is designed for the increased load, or by a more detailed analysis of the shear strength of the soil overburden.

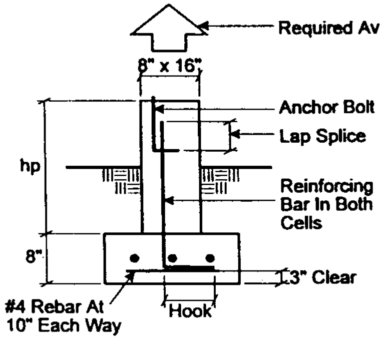
[4] Assumes 8" x 8" pier for the 1'-0" square footing, and 16" x 16" pier for the 4'-0" square footing.

Table C-3
Vertical Anchor Capacity For Piers[1, 2]
(In pounds)

Anchor Bolt Dia.	Capacity Per Number Of Bolts	
	1	2
1/2"	4240	8480
5/8"	6620	13240

Table C-3A

Anchor Bolt Dia.	Vertical Rebar	Minimum Lap Splice	Rebar Hook
1/2"	# 4	16"	6"
5/8"	# 5	20"	7"

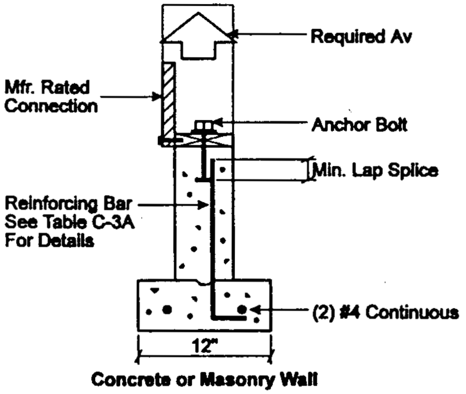


[1] The vertical anchor capacity is based upon the working capacity of ASTM A-36 rod stock anchor bolts in 2500 psi concrete or grout. To fully develop this capacity, anchor bolts must be properly lapped with the pier's vertical reinforcement.

[2] The capacity is based on f_c = 2500 psi; F_y = 36,000 psi.

FIGURE 9B.10 Withdrawal resistance and anchor capacity.[23]

Table C-4A
Vertical Anchor Capacity For Longitudinal Foundation Wall[1]
(In pounds per linear foot of wall)



Vertical Capacity[5] lbs./ft.		Required Anchorage[2, 3]		
Standard Washer	Over-Sized Washer	Anchor Bolt	Rebar[4]	Spacing[5]
146	239	1/2"	# 4	6'-0"max.
164	270	↓	↓	5'-4"
187	307	↓	↓	4'-8"
218	359	↓	↓	4'-0"
262	431	↓	↓	3'-4"
327	538	↓	↓	2'-8"
437	718	↓	↓	2'-0"

[1] Compare with required Av for Type E units.
[2] Values are based on vertical capacity per foot of wall.
[3] Assuming 1 1/2" thick sill plate, 3/4" edge distance for wood or composite nailer plates or 20 diameter end distance for plywood sheathing; APA rated, properly seasoned wood; Group III woods, not permanently loaded, and a 25% length of bearing factor increase.
[4] It is assumed that a reinforcing bar of the same diameter and spacing as the anchor is adequately embedded in the footing and lapped with the anchor.
[5] Spacing and capacity is based on allowable compression of wood perpendicular to grain for F_c = 565 psi and washer as define below:

Standard washer: 1 3/8" O.D. and 0.5625" I.D. washer (for 1/2" ϕ bolt)
Over-sized washer: 1 3/4" O.D. and 0.6875" I.D. washer (for 5/8" ϕ bolt) placed under the standard washer.

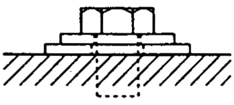
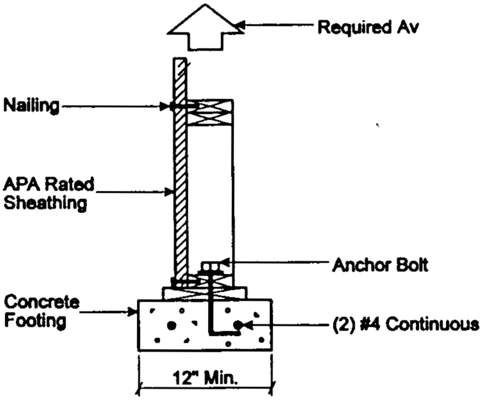


FIGURE 9B.11 Vertical anchor capacity for concrete or masonry wall.[23]

Table C-4B
Vertical Anchor Capacity For Longitudinal Foundation Wall[1, 2]
(In pounds per linear foot of wall)



Treated Wood Wall

Vertical Capacity lbs./ft.	Required Nailing[4, 5] (Edge Spacing, in.)	Min. Plywood Thickness	Required Anchorage[2, 3]	
Standard Washer			Anchor Bolt Diameter	Bolt Spacing[6]
146	6d @ 6" o.c.	3/8"	1/2"	6'-0"max.
164				5'-4"
187				4'-8"
218	8d @ 6" o.c.			4'-0"
262	8d @ 4" o.c.			3'-4"
327	8d @ 4" o.c.	15/32"		2'-8"
437	10d @ 2 1/2" o.c.			2'-0"
***				

*** For required Av greater than 437 lbs./ft., consider using a different foundation material or utilize an engineered design with a higher capacity.

1 Compare with required Av for Type E units.
2 In the case of a treated wood foundation wall, the wood wall and its connections must be designed to transfer the anchor load to a concrete footing. This table does not apply to treated wood foundation walls on gravel bases.
3 Values are based on vertical capacity per foot of wall.
4 Assuming 1 1/2" thick sill plate, 3/4" edge distance for wood or composite nailer plates or 20 diameter end distance for plywood sheathing; APA rated, properly seasoned wood; Group III woods, not permanently loaded, and a 25% length of bearing factor increase.
5 Nailing schedule in this table is intended to secure the superstructure to the foundation only, and not to provide required edge fastening for plywood siding or sheathing.
6 Spacing and capacity is based on allowable compression of wood perpendicular to grain for F_c = 565 psi and standard washer = 1 3/8" O.D. and 9/16" I.D. washer (for 1/2" ϕ bolt).

FIGURE 9B.12 Vertical anchor capacity for treated wood wall.[23]

Table C-5A
Horizontal Anchor Capacity For Transverse or Longitudinal Shear Walls[1]
(In pounds per foot of wall)

Concrete or Masonry

Horizontal Capacity[2] lbs./ft.	Required Anchorage[5]		
	Anchor Bolt[4]	Rebar	Spacing[6]
300	1/2"	#4	72" o.c. max.
600			36" o.c.
675			32" o.c.
900			24" o.c.
1350			16" o.c.
1800			12" o.c.
***			
See Table C-3A For Rebar Details			

*** For required Ah greater than 1800 lbs./ft., consider using an engineered design with a higher capacity.

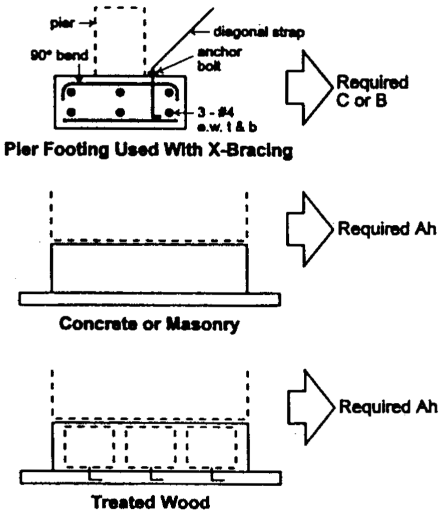


Table C-5B

Treated Wood

Horizontal Capacity[2] lbs./ft.	Required Nailing[3, 4] (Edge Spacing, in.)	Min. Plywood[4] Nailer Thickness	Required Anchorage	
			Anchor Bolt Diameter	Bolt Spacing[7]
300	8d @ 4" o.c.	7/16"	1/2"	4'-0" max.
360	8d @ 4" o.c.	15/32"		3'-4"
449	10d @ 4" o.c.	15/32"		2'-8"
600	10d @ 3" o.c.	19/32"		2'-0"

1 Compare capacity with required Ah in transverse or longitudinal direction.
2 Values are based on horizontal load per foot of wall. Select Ah for pier spacing of 4 feet for use with this table.
3 Assuming 1 1/2" thick sill plate, 3/4" edge distance for wood or composite nailer plates or 20 diameter end distance for plywood sheathing; APA rated, properly seasoned wood; Group III woods, not permanently loaded.
4 Nailing schedule in this table is intended to secure the superstructure to the foundation only, and not to provide required edge fastening for plywood siding or sheathing.
5 It is assumed that a reinforcing bar of the same diameter as the anchor is adequately embedded in the footing and lapped with the anchor. In the case of a treated wood foundation wall, the wood wall and its connections must be designed to transfer the anchor load to a concrete footing. This table does not apply to treated wood foundation walls on gravel bases.
6 Spacing based on bearing capacity of bolt against concrete/grout.
7 Spacing based on capacity of anchor bolt in bearing against the wood plate. (see also #5.)

FIGURE 9B.13 Horizontal anchor capacity.[23]

Some important programs, projects, or properties may regrettably have gone unmentioned because they have not come to the attention of the author. Nevertheless, it is hoped that the preceding review, which has attempted to emphasize several HUD programs, projects, and property types, will provide an informative guide as to how HUD handles the critical housing subject of foundations. For more detailed information on HUD programs, including available publications or handbooks, contact the HUD field office in your area.

9B.8 ACKNOWLEDGMENTS

Grateful acknowledgment should be expressed to many colleagues, especially to the limited nationwide staff of HUD technical professionals (architects, engineers, and construction analysts) who strive to assure that properties and projects involved in HUD program areas have adequately designed and constructed foundations. A special acknowledgment is also expressed to G. Robert Fuller (Retired, HUD Headquarters), Kenneth L. Crandall (Retired, HUD headquarters), and Robert Wade Brown, whose helpful suggestions and cooperation have contributed significantly to the preparation of this section.

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