# Hydrocompression and Hydroswell—New Terms in the Geotechnical Dictionary

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## ABSTRACT

Consolidation of the skeletal fabric of soil mixtures upon their initial absorption of free water is not a new concept. This concept was initially introduced in soil mechanics literature in 1932, but only since 1989 has the term "hydrocompression" appeared in the engineering literature; however, in that short time this process has come to be recognized as a unique mechanism of permanent ground settlement worthy to be reckoned with in engineered fill embankments. The absorption of free moisture can also cause volumetric expansion of a soil mixture depending on clay mineralogy, initial density, and confinement. The interaction of these opposing mechanisms complicates the assessment of future settlement. Since the 1930s, traditional engineering assessments of embankment settlement have centered on one-dimensional consolidation theory based upon the measured expulsion of pore water from saturated soil mixtures. This article briefly examines the various mechanisms of ground settlement recognized in the engineering literature, realizing that laboratory research has tended to concentrate on either naturally occurring soils or mechanically compacted embankments. Because workers with different scientific backgrounds have studied similar phenomena, there is little standardization of nomenclature. To acquaint the reader with the various theories, mechanisms, and methods of evaluation, mention is made of similar evaluations and discoveries emanating from the geotechnical engineering and engineering geology communities dating back 70 years. A comprehensive list of references is included.

## INTRODUCTION

In the late 1960s heavy rains in coastal California caused considerable differential settlement in large embankments graded during the previous decade for urban development. Settlement was most damaging adjacent to cut-fill transition areas and deeply incised ravines filled with compacted fill. Until that time, engineering assessment of potential settlement was limited to the theory of one-dimensional primary consolidation espoused by a German engineering professor named Karl Terzaghi in the late 1920s. By the early 1960s, Terzaghi and other geotechnical engineers began to recognize other mechanisms of settlement, including collapse upon wetting in semiarid areas and secondary consolidation of saturated marine clays. However, neither primary or secondary consolidation could explain the settlements observed in the late 1960s, and some engineers postulated that piping of fine grained soils had occurred in buried subdrains beneath the fills. Subsequent surveys revealed that the great majority of the observed settlement was not reoccurring, despite even heavier doses of moisture brought about by heavy rainfall between 1980–1983, which had seen the greatest volume of precipitation in a 3-year period ever recorded (since 1849).

The concept of absorbed water causing volumetric adjustments of the soil skeleton, or fabric, was introduced to the engineering community by Professor Arthur Cassagrande at Harvard University in 1932. Farmers in California's Fresno County began to notice water-related subsidence as early as the turn of the Century (Bull, 1964), and farmers in Kern County began to notice collapsing soil phenomena in the 1930s (Roberts, 1965). Oil and gas companies reported similar phenomena in western Fresno County (Sneddon, 1951). Buildings in Taft, California, reported noticeable settlement in the 1950s, as did hydraulic structures in areas underlain by wind-deposited silt in Nebraska (Lofgren, 1965) and flood outwash deposits in the semiarid portions of eastern Washington (Holtz and Hilf, 1961). By the mid-1960s many engineers began to recognize the potential for eolian deposited soils (loess and sand dunes) to collapse upon the addition of moisture (Roberts, 1965).

In 1954 the Inter-Agency Committee on Land Subsidence in the San Joaquin Valley was formed, with the late Joseph F. Poland of the U.S. Geological Survey as its chairman. The purpose of this committee was to plan and coordinate a program of cooperative research to provide useful information on the extent, magnitude, rate and causes of various types of land subsidence in the San Joaquin Valley (Inter-Agency committee, 1958). Nine agencies were involved in this cooperative effort and the details of their milestone findings are reported elsewhere in this volume. The results of field tests in the San Joaquin Valley by the Inter-Agency Committee coincided with smaller scale research then being carried out by one of the project participants, the U.S. Bureau of Reclamation, in Washington, Nebraska, and Nevada (Holtz and Hilf, 1961).

The recognition of the Inter-Agency studies of that era was slow in reaching practicing engineers because the results were not always published in mainstream engineering journals and the problem was thought to be unique to the western San Joaquin Valley. Geologists studying the water-induced settlement liked to term the phenomenon "compaction," whereas engineers always favored the term "consolidation" or "compression" to describe any manner of soil volume loss. In the early literature, ground subsidence caused by compaction due to wetting was reported as "hydroconsolidation" by Krynine and Judd (1957) and "hydrocompaction" by Prokopovich (1963) and Lofgren (1965). Researchers from the Inter-Agency Committee generally reported land subsidence associated with the addition of water at the ground surface as "near-surface subsidence," limited to the upper 150–200 ft (Bull, 1961, 1964). Dramatic land subsidence due to artesian head decline was a different mechanism of settlement within the same general area (west side of Kern County), but occurred at considerable depth due to compaction of unconsolidated upper Cenozoic sediments due to ground-water withdrawal. This mechanism is not of concern to geotechnical engineers evaluating settlement of man-constructed embankments, and is discussed elsewhere in this volume.

Concurrent with the gradual appreciation of water-induced subsidence was the more easily appreciated phenomenon of swelling clay, which received considerable attention from the engineering community worldwide, dating back to the early 1930s (Simpson, 1934). In the early 1950s geotechnical engineers began to grapple with the foundation damage that expansive soils caused in lightly loaded structures, such as houses (Dawson, 1953; Tschebotarioff, 1953). Because of the increasing damage caused by expansive clays, basic research was carried out that revealed that swelling was a function of initial moisture content and effective confining stress (Holtz and Gibbs, 1956; Ward, 1957; Zeitlen and Komornik, 1961). Practicing engineers, hungry for cost effective solutions, developed simple qualitative tests for expansive soil tendency, resulting in the expansion index test procedure introduced in the Chapter 70 Appendix of the Uniform Building Code in 1964. This phenomena has loosely been termed "swell" or "hydroswell."

As residential development spread across the southern California coastline, problems with differential settlement of foundations became increasingly common in areas underlain by old coastal dunes. By the mid-1960s geotechnical engineers in southern California began to recognize the potential for so-called collapsing soils in areas of eolian deposits (Roberts, 1965). In 1970, Dudley published an article that established the case for odometer testing of low-density soils, especially silts and wind-blown sands in the greater Los Angeles area, and this simple evaluation became commonplace.

Between 1980 and 1983 three extranormal rainfall years occurred in California, back-to-back. Embankments constructed of mechanically compacted fill experienced marked differential settlement, far beyond that expected or explained with conventional consolidation theory. Geotechnical engineering researchers, led by Professor Iraj Noorany at San Diego State University, began to evaluate the hydrocompression phenomenon in soils laboratories, mostly as the result of litigation involving differential settlement of deep urban fills (Brandon and others, 1990). Their research was coincident with that of other pioneering workers, such as Richard Fragaszy at Washington State University, who were making similar evaluations, but of predominantly granular soil mixtures. These two groups of hydrocompression researchers pursued different test methods, one invoking soaking before loading and the other only allowing soaking after loading. The debate as to which test approach best replicates actual field loading conditions continues, but the recognition of hydrocompression and hydroswell as important mechanisms requiring serious consideration has come to the fore of the geotechnical engineering profession, some 30 years after it swept the engineering geology profession.

The balance of this article seeks to introduce the established geotechnical engineering theories of soil density and moisture conditioning that drive design and construction of engineered fill embankments; the development of theorems to explain and predict future ground settlement; the various mechanisms that can give rise to settlement, including pseudo-settlement; and what changes we are likely to see in the standard-of-practice for designing and constructing large urban fill embankments over the coming decade. Particular emphasis is placed upon recognizing those contributions that have appeared in the past decade and profoundly influenced the engineering community, but may not have been reported in benchmark geotechnical engineering texts.

## MOISTURE-DENSITY RELATIONS IN COMPACTED SOILS

In engineering literature, the term "saturation" refers to the percent of the pore space filled with water. The moisture content is reported as the ratio of the weight of the pore water divided by the dry weight of the soil particles. In this way a marine clay might be reported as having 100 percent moisture content. Naturally occurring soils lying below the zone of evapotranspiration typically retain moisture. Sandy soils generally contain between 3 and 20 percent moisture, silts between 6 and 30 percent and clays between 15 and 50 percent moisture by weight. In

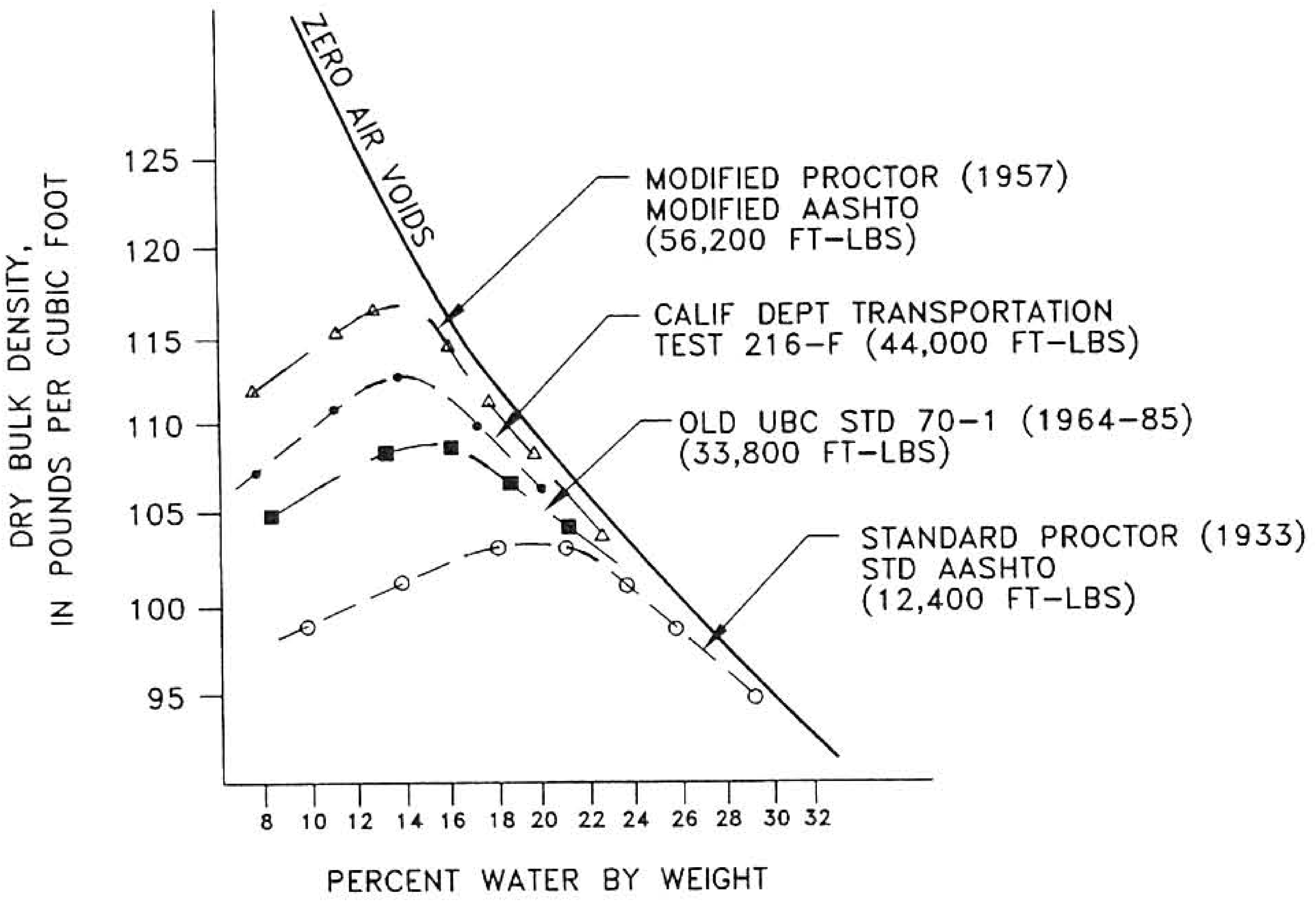



**Figure 1. Relations between moisture and density for Pittsburg silty clay. This plot, commonly referred to as a "compaction curve," allows evaluation of compacted soil density with varying water content to better determine how much water should be added to a given soil mixture to aid mechanical compaction. Results from the four most recognized test procedures are presented. Note how an increase in compactive effort lessens the percent water (by weight) needed to aid densification. The Modified Proctor test is the only procedure legally recognized by the Uniform Building Code since 1985.**

engineering, the bulk density (referred to as "density" or "dry density") of soils is reported as the dry weight of a given volume of soil, such as pounds per cubic foot (fig. 1).

The most common method of graphically depicting laboratory-derived soil moisture-bulk density relationships was developed by Proctor (1933a, b, c) and Zimmerman (1935) during the construction of the Bouquet Canyon Reservoir embankments in southern California. This methodology involves the input of compactive energy upon a known volume of soil in thin lifts, or layers, conditioned to various moisture contents. Known colloquially as a "compaction test" or "Proctor test," many variations of this basic method exist, involving a wide range in compactive energy from 12,400 ft-lb/ft$^3$ of soil ("Standard Proctor" test, ASTM D-698) to 56,200 ft-lb/ft$^3$ of soil (the "Modified Proctor" test, ASTM D-1557) (shown in fig. 1). A separate test of the soil particle relative density is necessary to determine a more precise relationship between soil bulk density and moisture. Generally, the test accuracy is no more than ±2.5 percent because of uncertainties in soil mineralogy and composition.

Proctor (1933a, b, c) and Zimmerman (1935) discovered that the addition of moisture to the soil helped lubricate particles so that the maximum bulk density was achieved at about 80–85 percent of saturation, referred to as the "optimum water content" (fig. 2). The Modified Proctor test was introduced in 1958 because construction equipment had become substantially larger and capable of delivering greater compactive effort than those machines available in 1932 when the methodology was conceived. Gradually, the Modified Proctor test has come to replace other tests employing less input energy as the standard-of-choice in the engineering industry (fig. 1). In 1985, the

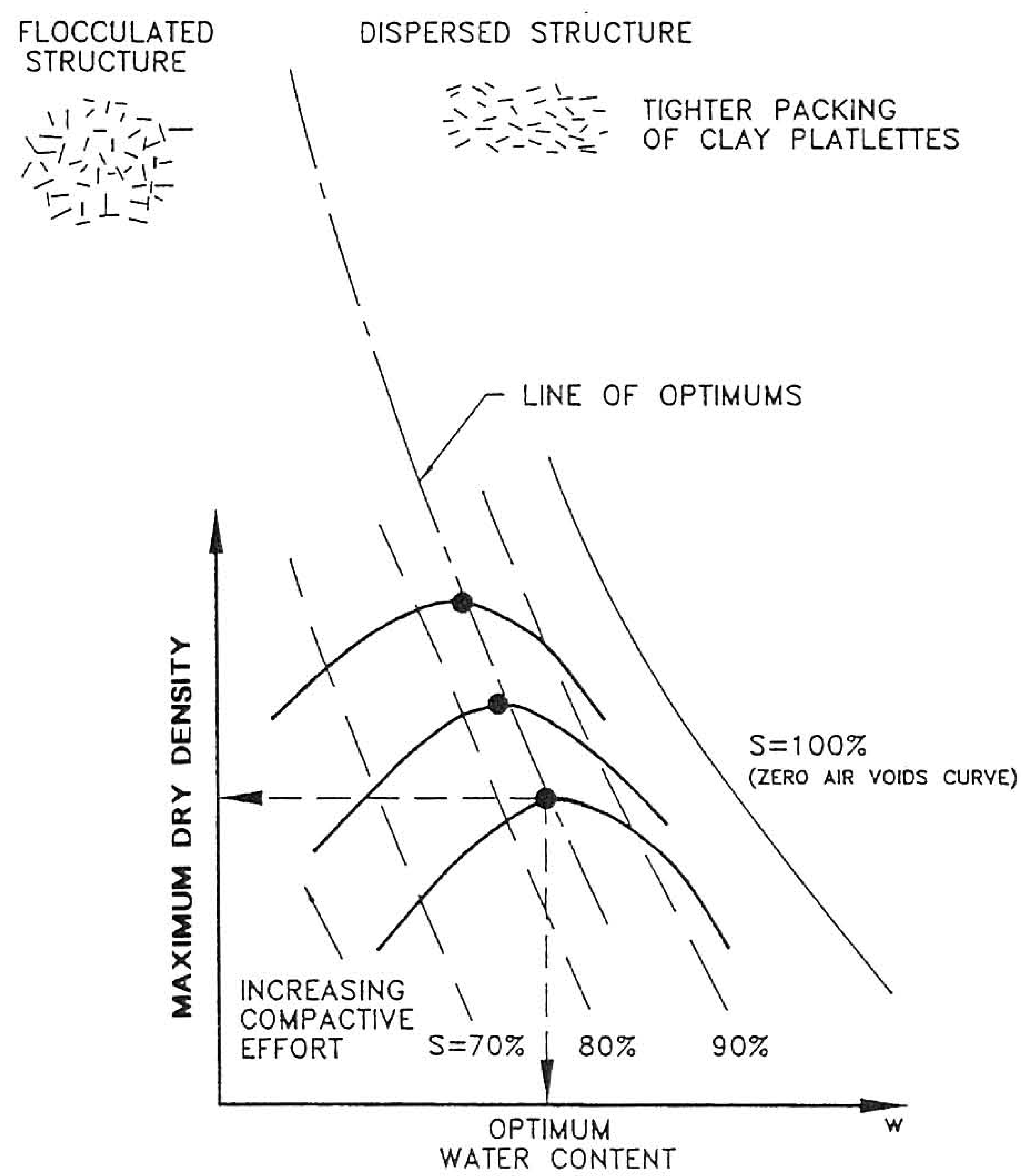



**Figure 2. Relations between soil water content, dry density, percent saturation, and compactive effort. Optimum moisture is defined as that water content (by weight) corresponding to the maximum achievable density by a defined quantity of compactive energy. The greater the input energy, the less water is necessary to achieve greater density. In general, the "line of optimum" moisture content approximates 85 percent saturation. Clayey materials tend to be dispersed wet of this line and flocculated dry of the optimum moisture content.**

Uniform Building Code appendices were amended to mandate all mechanical compaction of soils be greater than 90 percent of the maximum density achieved by the Modified Proctor test (ASTM D-1557). Similar adjustments have been proposed to the ASTM Committee D-19 on soil and rock in the late 1980s, but representatives of some midwestern states refuse to part with the Standard Proctor test, despite its being outdated. However, some latitude is afforded with the D-1557 standard, such as the case of diatomaceous soil mixtures (Khilnani and Capik, 1989). Many state transportation departments also employ less restrictive compaction standards, as shown in figure 1.

## IMMEDIATE SETTLEMENT AND INITIAL CONSOLIDATION

When a soil is loaded, either by fill surcharge (burial) or structures, some soil deformation will occur, particularly near free surfaces. This phenomenon has often been referred to as "bulging," "lateral bulging," or simply "horizontal movement" (Hardy and Ripley, 1961). This deformation was originally noted in detailed theodolite surveys of earthen dam embankments while they were under construction and during their initial filling. The vertical component of deformation is commonly referred to as "settlement." The movement of soil may be downward or upward, depending on expansivity of the soil and the initial moisture content.

In the old geotechnical engineering literature, "immediate settlement" referred to that portion of settlement that initially occurs in saturated soils upon loading (Terzaghi, 1943). Later, it was learned that the same phenomenon was even more noticeable in partially saturated soils due to closure of air voids, and was eventually termed "initial consolidation" (discussed below). Early workers ascribed immediate settlement to that deformation that will occur when a soil is initially loaded, or reloaded, depending on soil/rock elasticity. Although soil is not actually elastic, theorems of elasticity were originally used in estimating the immediate settlement, generally through plate load tests, the preferred method of foundation testing in the late 1930s. In fine-grained soils, this response is not always immediate, but may take time to occur, as the newly imposed stresses are transferred to the underlying soil mass.

Steinbrenner (1936) and Terzaghi (1943) presented theorems by which computation of settlement could be estimated due to loads on the surface of semielastic soil or rock foundations. This approach incorporated elastic theory, and required that the soil modulus of elasticity and Poisson's ratio are known. These parameters are seldom known in compacted fills, which are highly anisotropic, due to their layered structure. In addition to these physical properties, the contact stress distribution within the soil must be estimated and considered in the analysis. Although elastic theory is used in such computations, soils generally exhibit elasto-plastic behavior, meaning there is a recognizable component of permanent (plastic) deformation. In addition, we can expect that properties of soil deformability, like those of discontinuous rock, are both stress dependent and strain dependent, further complicating any analytical estimation. However, researchers in the field of pavement design have long recognized that soil deformability increases with increasing moisture content (Monismith, 1990).

Initial consolidation is a different phenomenon from immediate settlement, although both mechanisms may occur simultaneously. Initial consolidation is an older term (Terzaghi and Peck, 1948) that was used to describe settlement that occurred largely through the compression of voids and solution of air in the soil. The tendency to undergo initial consolidation is largely a function of percent saturation. Saturated soils will yield virtually no initial consolidation, whereas partially saturated soils will almost always exhibit some measurable degree of initial consolidation. Initial consolidation can be estimated in ordinary odometer tests through evaluation of detailed time-settlement data (change in void ratio with initial seating load).

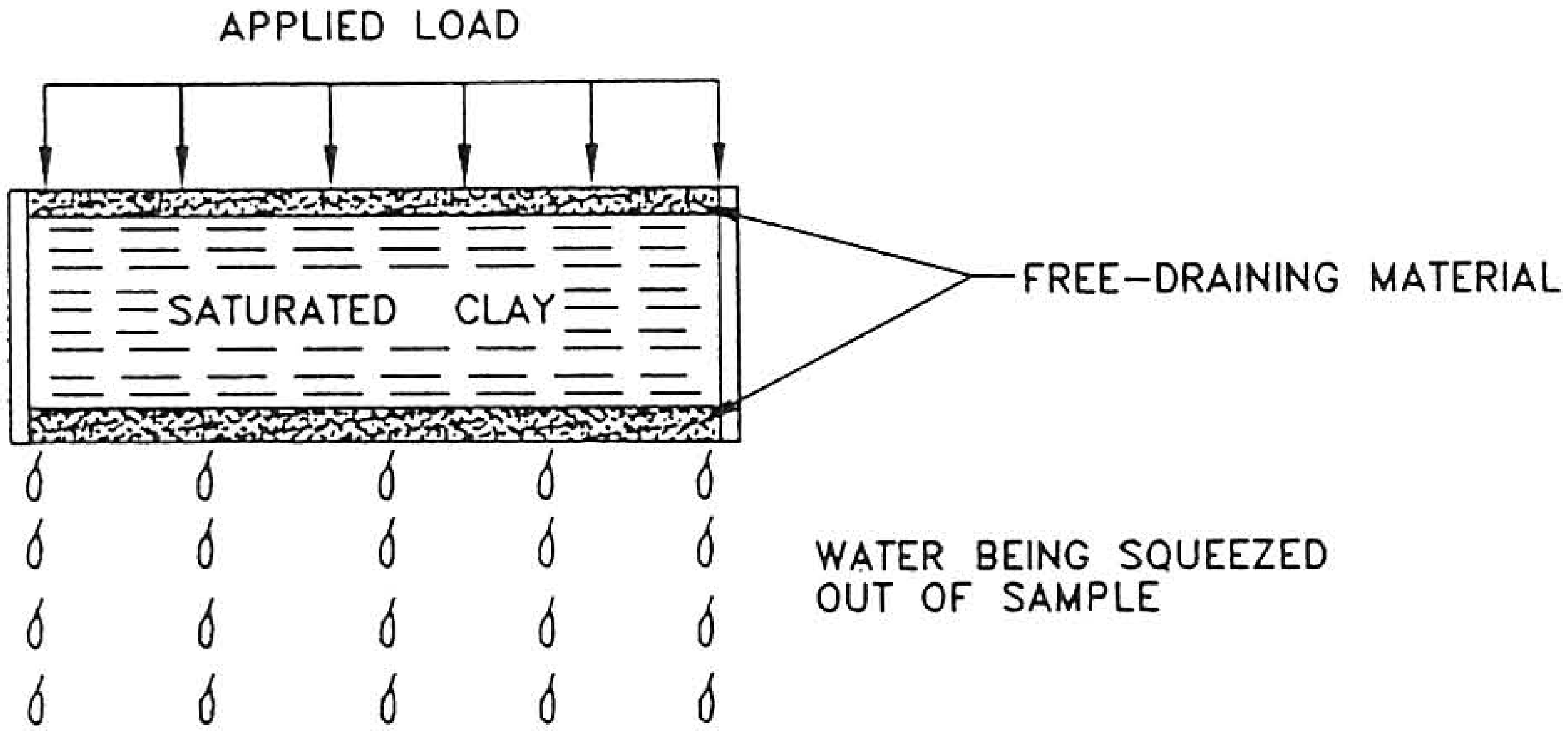



**Figure 3. Primary consolidation occurs when interstitial pore water is squeezed out of saturated clay.**

Immediate settlement and initial consolidation are terms that are used infrequently today because geotechnical engineers no longer depend on plate load tests for determining foundation bearing capacity, nor do they worry about immediate consolidation and settlement that, by definition, would be expected to occur in a compacted fills long before structures are built; however, the phenomena still occur when deep fills are loaded and air voids are slowly compressed due to advancement of load through the soil mass. There is some time rate of dependence involved in the closure of air voids in deep fill embankments, and this phenomena is therefore slightly different from primary consolidation.

## PRIMARY CONSOLIDATION

Primary consolidation of clayey soils occurs through the expulsion of interstitial pore water under newly imposed loads (fig. 3). In engineering terms, primary consolidation is the reduction of soil void ratio as a result of the time-dependent expulsion of soil pore water, shown diagrammatically in figure 4. The expulsion of pore water is a time-dependent process, most recognizable in saturated clay mixtures. The rate of expulsion is a factor of seepage path distance and effective hydraulic conductivity. The simple assessment of one-dimensional consolidation is the most commonly recognized and studied form of soil subsidence.

The consolidation of saturated soft marine and lacustrine clays in response to loading has long been recognized in low-lying coastal estuaries, such as the bays surrounding San Francisco, Boston, and Rotterdam (Lee, 1935; Terzaghi and Frohlich, 1936; Terzaghi, 1943). This methodology was expanded upon by subsequent workers, including Leonards and Girault (1961), Leonards (1973), Schmertmann (1983), and Nwabuokei and Lovell (1986). Lee and Praszker (1969) provided a detailed description of primary consolidation effects and analysis of the San Francisco Bay Mud, incorporating data accumulated over more than 40 years.

The basic analytical approach in evaluating primary consolidation was developed by Karl Terzaghi (1925), a professor in Vienna at the time, and test procedures in America were subsequently introduced by Arthur Cassagrande in 1932. The current format of the Cassagrande one-dimensional consolidation (odometer) test is designated as ASTM Test D-2435, adopted in 1965. In one-dimensional consolidation tests, soil specimens are laterally restrained in a brass ring while the sample is loaded axially in increments, as presented schematically in figure 4. Measurements are made of the change in specimen height with each load increment, giving a graphic depiction of the change in soil void ratio with increasing effective soil stress, as presented in figure 5. As water is squeezed out under newly imposed loads, the soil compresses most dramatically, along the steeply inclined path shown in figure 5, known as the "virgin compression curve."

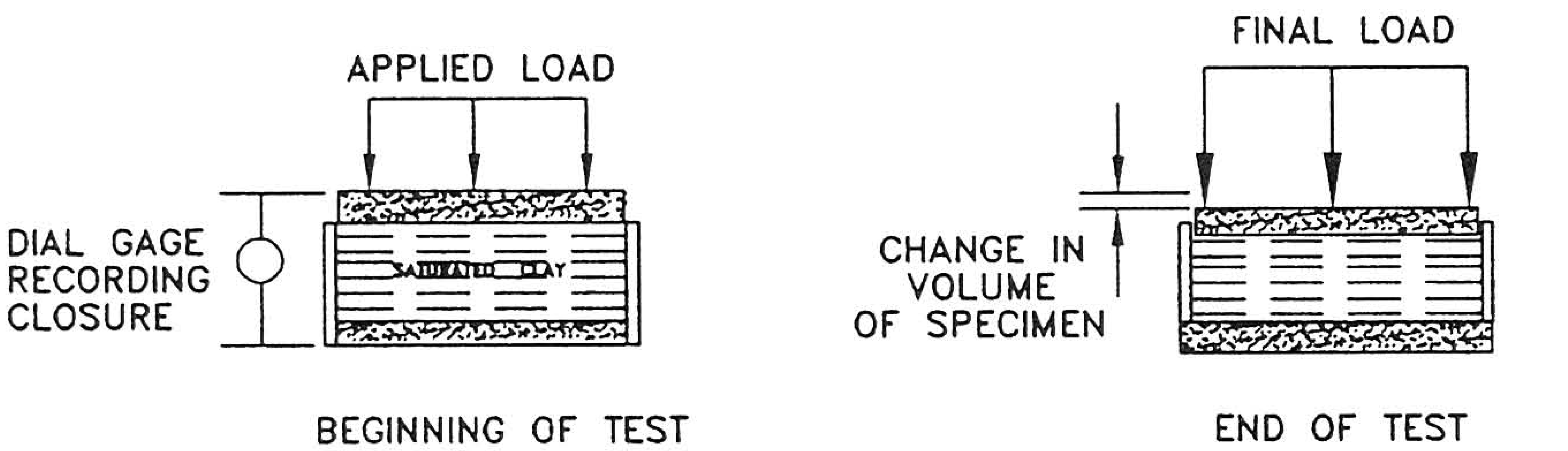



**Figure 4. One-dimensional consolidation test measures the change in soil volume.**

Duncan and Buchignani (1976) wrote a basic handbook for assessing primary consolidation that was revised in 1987. Representative constants and input values for consolidation assessments are contained in similar computational outlines, such as the Navy's Soil Mechanics Design Manual (NAVFAC DM-7.1), published by the federal government in 1968 and 1982. The other benchmark reference for this methodology is the U.S. Corps of Engineers' *Settlement Analysis*, Technical Engineering and Design Guide No. 9, published by the American Society of Civil Engineers (ASCE) in 1993. Both of these manuals address one-dimensional primary consolidation. Over the past decade several good overviews and technical discussions of primary consolidation have appeared in ASCE's *Journal of Geotechnical Engineering*, including Mesri and Rokhsar (1974), Mesri and Choi (1985a), Young and Townsend (1986), and Clarke and others (1993).

In the testing lab, most geotechnical engineers have two basic choices for assessing one-dimensional primary consolidation: the older ASTM D-2435-90, or "Cassagrande test procedure," and the newer strain-rate-controlled consolidation procedure outlined in ASTM D-4186-89, initially adopted in 1982. Young and Townsend (1986) provided an up-to-date compendium on all the various aspects of soil testing and analysis related to settlement. An abundance of representative case histories are contained in the proceedings for the International Settlement Conference held in 1994 in Houston (ASCE Geotechnical Special Publication 40).

Primary consolidation is not a dominant long-term mechanism in most urban embankments because fill soils are only partially saturated when they are placed (Hilf, 1956, 1984). In most cases, the lower portions of deep canyon fills will undergo primary consolidation as overburden pressures rise sufficiently to effect saturation, by reducing the volume of soil voids (sometimes referred to as "load densification").

Making simple assessments of primary consolidation characteristics would be obligatory when working on soft compressible (saturated) clays and silts intended for significant surcharging, either by embankments or structures. For embankments, the test regimen is generally performed when assessing fills over 25 ft deep, although some consultants choose to estimate fill settlements as being between 0.2 and 0.4 percent of total fill thickness, depending on soil plasticity, water content at placement and familiarity with past performance of embankments in the area. No specific requirement for quantifying primary consolidation is cited in the Uniform Building Code. One of the inherent problems in assessing long-term primary consolidation is unanticipated changes in soil moisture due to changing water levels not foreseen at the design stage.

## SECONDARY CONSOLIDATION

Secondary consolidation, sometimes also referred to as secondary compression (Terzaghe and Peck, 1967), is a continuation of the volume change that began with primary consolidation, only at a much slower rate. This mechanism applies only to saturated clay mixtures and peats. The phenomenon was recognized as early as the mid-1930s (Buisman, 1936), and was the subject of considerable discussion among geotechnical engineers in the 1950s (Terzaghi, 1953; Zeevert, 1957). Introductory discussions were included in

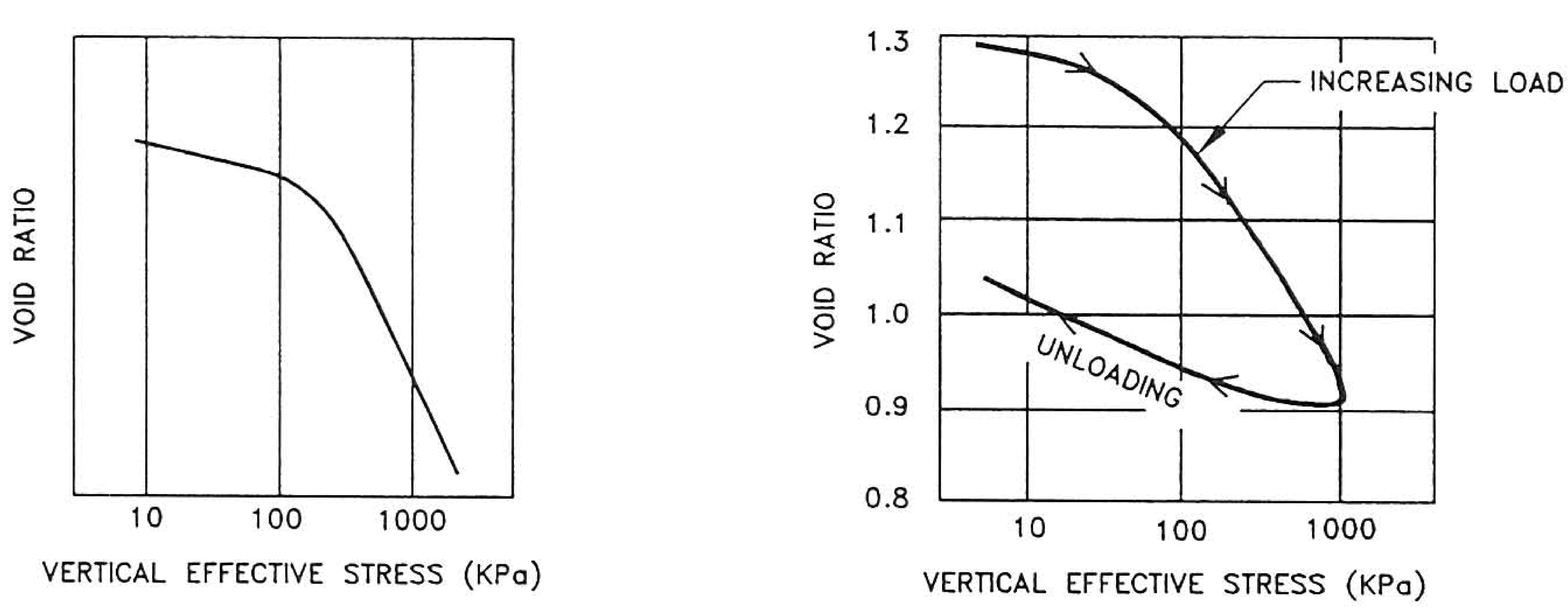



**Figure 5. Traditional representations of primary consolidation tests.**

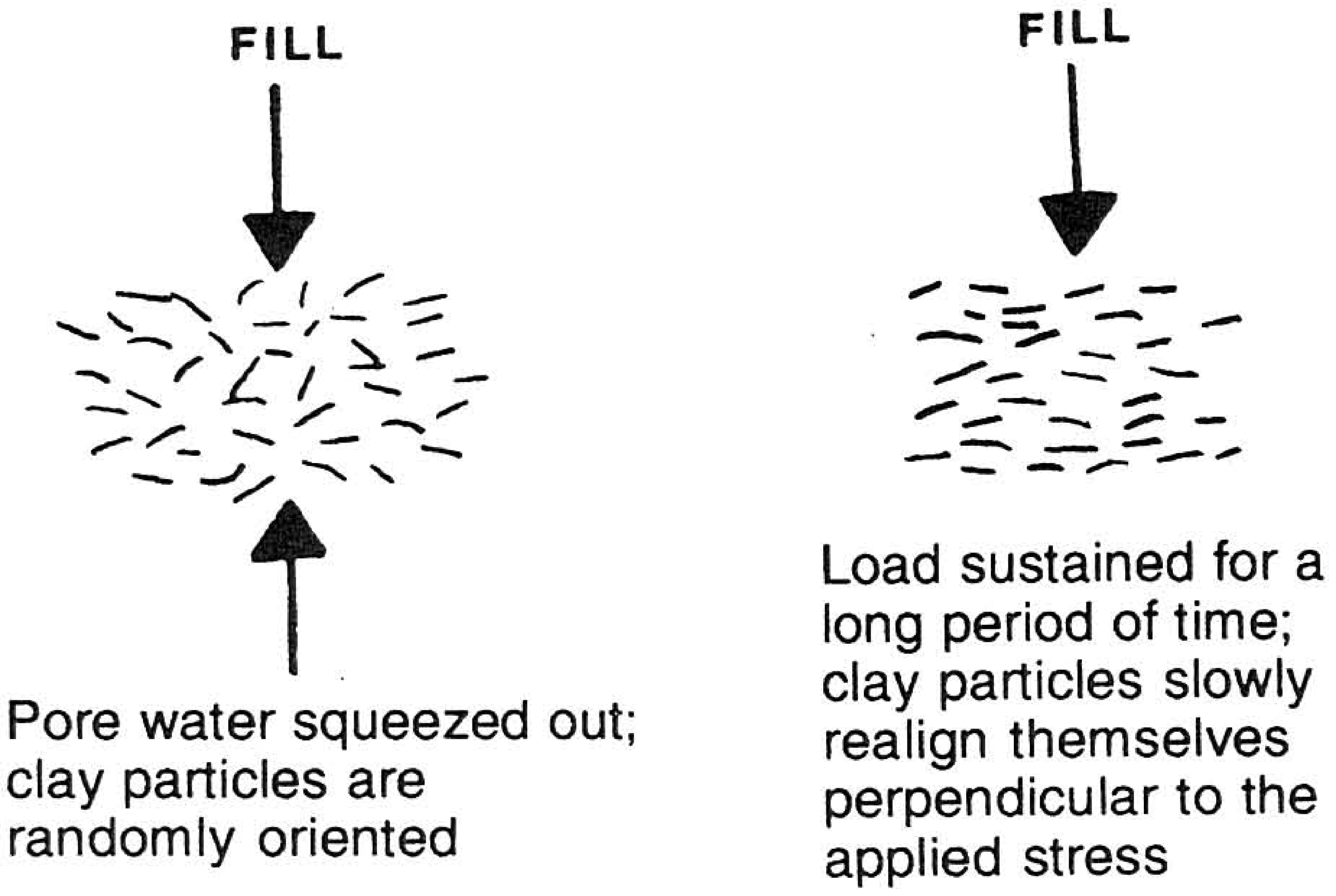



**Figure 6. Basic mineralogical phenomena responsible for secondary consolidation. Pore water expulsion made possible by long-term realignment of saturated clay platelets under sustained loads is believed to be the mechanism under which secondary consolidation (sometimes termed "secondary compression") occurs.**

most of the classic soils engineering texts of the 1960s (Leonards, 1962; Terzaghi and Peck, 1967). However, the watershed references on secondary consolidation did not appear until 1973, when Gholamreza Mesri (1973; also Mesri and Godlewski, 1977) and his associates at the University of Illinois began to publish landmark articles. A number of collaborative commentaries have since appeared, including those by Aboshi (1973), Raymond and Wahls (1976), Ladd and others (1977), and Mesri and Castro (1987). The currently accepted methodology for assessing secondary consolidation is contained in Mesri and Godlewski (1977, 1979), but an easy-to-read overview of secondary consolidation theory is also contained in the soil mechanics textbook by Holtz and Kovacs (1981).

Unlike primary consolidation, secondary consolidation occurs at a constant

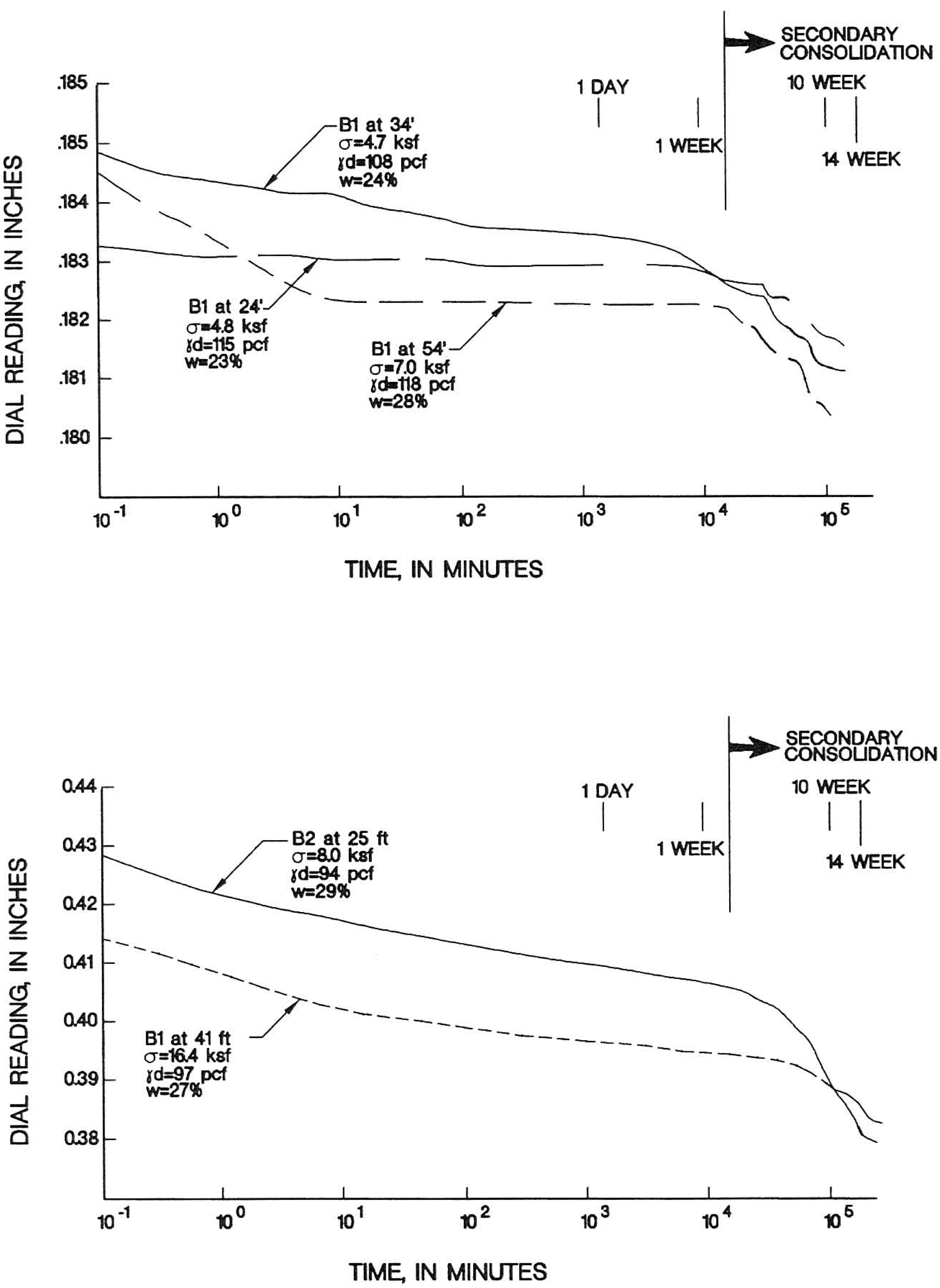



**Figure 7. Raw secondary compression test data, as measured in 14-week-long, one-dimensional consolidation tests. Secondary compression usually begins more than a week after starting the standard test sequence. Samples were taken from deep urban fill placed in 1963, from depths between 24 and 54 ft below land surface.**

effective stress, after excess pore pressures (due to surcharging) have dissipated. Secondary consolidation appears to be ascribable to moisture expulsion made possible by the fabric rearrangement of clay platelets due to breakage of cationic bonds between such platelets (fig. 6). Many workers (such as Terzaghi and Peck, 1967) had long observed near-constant rate, long-term settlements, long after primary consolidation had ceased. The phenomenon also had been recognized on the campus of the Massachusetts Institute of Technology (MIT) located in Cambridge, Massachusetts, where masonry buildings constructed in the late 19th century settled as underlying clayey deposits began to undergo secondary consolidation approximately 35 years after primary consolidation ceased. (Harl Aldrich and others, 1983, written communication).

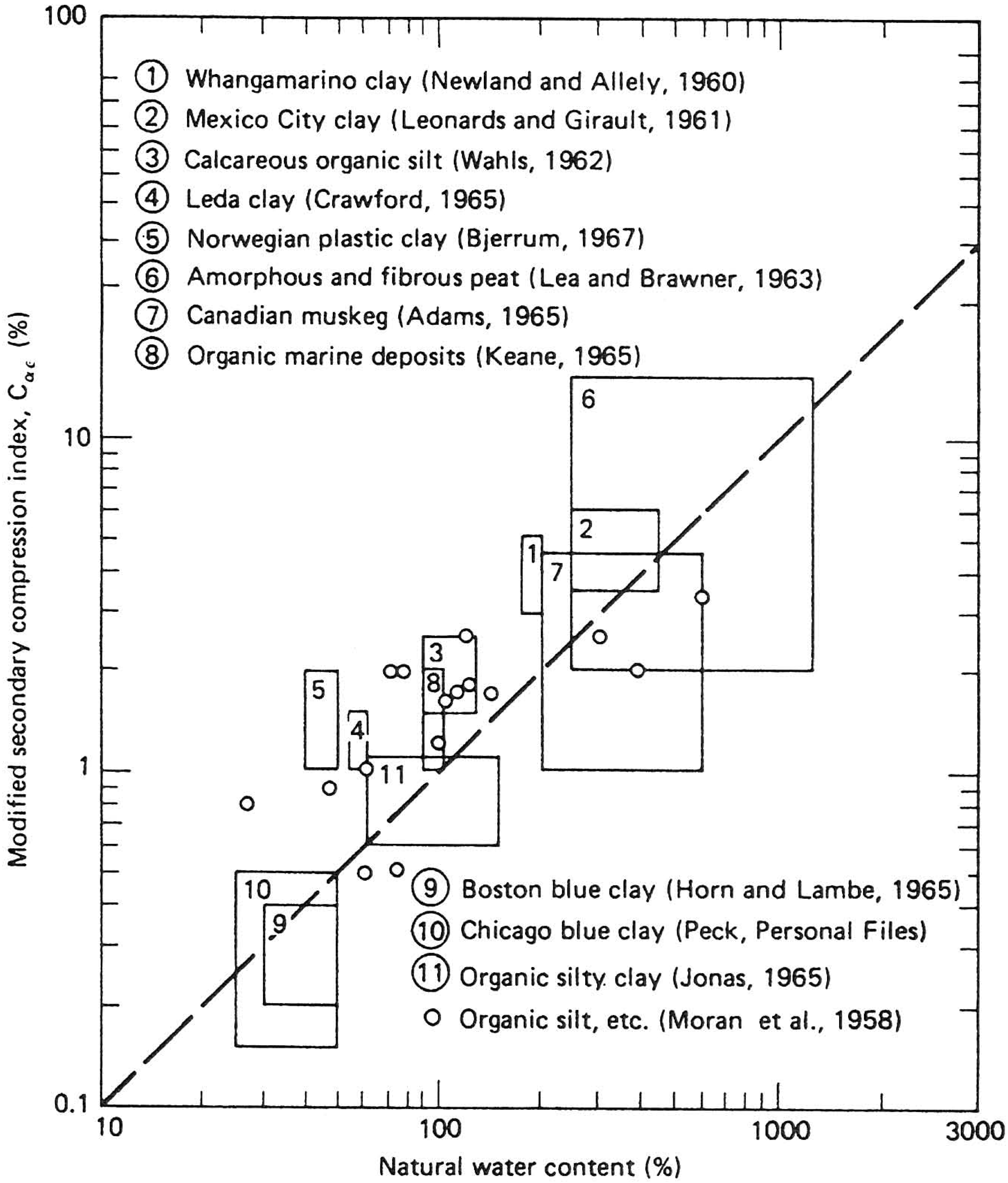



**Figure 8. Relation between natural water content and secondary compression index, as introduced by Mesri (1973 and references therein). Simplified forms of this chart have been reproduced in most soil mechanics texts, as well as in NAVFAC DM-7. 1-2 (1982). Note how the natural water content of the various saturated soils controls the compression index value. Samples of San Francisco Young Bay Mud normally have water contents between 30 and 50 percent moisture by weight (weight of water divided by weight of solids). Taken from Duncan and Buchignani (1976).**

Secondary consolidation also can be triggered by to increased overburden load, due to mass surcharge or lowering of ground-water levels in overconsolidated clays. Mesri and Godlewski (1977) presented a rational procedure by which secondary compression can be estimated, using longer term primary consolidation test data (such as that shown in fig. 7). One of the most problematic aspects of secondary consolidation is that it continues exponentially with time without definite termination (fig. 7). Mesri and Godlewski (1977, 1979) introduced the coefficient of secondary compression, $c_{\infty}$, that is calculated from long-term primary consolidation tests. A relationship between the coefficient of secondary compression (or consolidation) and natural water content (fig. 8) was originally presented by Mesri (1973) and has since been reproduced in a number of texts, including Duncan and Buchignani (1976), Holtz and Kovacs (1981) and NAVFAC DM-7.1 (1982). The test results shown in figure 8 are helpful in making preliminary assessments of expected secondary consolidation of estuarine muds in the United States.

Unlike primary consolidation, secondary consolidation does not appear to be affected by drainage-path length, but can be triggered by lowering of ground water, that causes the effective stress to increase (due to a loss of buoyancy) in overconsolidated clays. The rate of secondary consolidation appears to be controlled by initial soil moisture content and clay mineralogy (fig. 8).

By 1985, Mesri and Choi (1985b) had developed a methodology for preloading soft soil sites to effect predictable settlement. The following year Mesri (1986) began to address secondary compression concepts as applied to peats. In 1987 he published his paper, "Fourth Law of Soil Mechanics: a Law of Compressibility," in the proceedings for the International Symposia on Soft Soils (Mesri, 1987). A few months later Mesri and Castro (1987) presented a new method for predicting at-rest earth pressures during secondary compression. Other workers recognized that conventional predictive models for peat settlement were lacking and applied the concept of secondary consolidation to peat soils (Fox and others, 1992). This article fostered an interesting discussion by Mesri and others (1994), wherein they presented test data and asserted that the secondary compression index of peats increases significantly with time. This bodes ill for embankments founded upon such materials, especially levees built over such deposits that are subject to changes in ground-water level.

Secondary compression is widely recognized to occur in marine clays and estuary deposits, such as the San Francisco Bay Mud. Even before Mesri's (1973) first article appeared, Margason and Arrango (1972) had developed test data for secondary compression of San Francisco Young Bay Mud while studying a bay fill site in Emeryville, along the east shore of San Francisco Bay. Based on long-term tests, like those presented in figure 7, it would appear that secondary consolidation may begin in many of the deeper urban embankments in coastal California if these fills become sufficiently saturated (possibly due to gradual breakdown of subdrain systems, over watering or extreme precipitation and/or shallow ground-water infusion).

## FLUID WITHDRAWAL

Fluid withdrawal from the ground can often trigger mass consolidation of unconsolidated sedimentary formations, such as alluvium and lacustrine (lake) deposits. In the 1930s and 1940s, the withdrawal of petroleum in the Wilmington–San Pedro–Long Beach areas of southern California caused widespread areal settlement within specific oil fields (Harris and Harlow, 1947). By 1965, portions of Terminal Island in San Pedro settled as much as 29 ft (Mayuga and Allen, 1966). Between 1910 and 1970, ground-water withdrawal in coastal basins like that occupied by San Jose experienced upwards of 13 ft of subsidence (at North First Street and St. James) (Tolman and Poland, 1940; Green, 1964; McArthur, 1981). During this same period, subsidence in the San Joaquin Valley ascribable to agricultural ground-water withdrawal was documented to be in excess of 27 ft (Poland and others, 1975). Other authors (this volume) address this subject in much greater detail.

From a soil mechanics view, the mechanism causing ground settlement via fluid withdrawal is two-fold: (1) pore fluids between soil grains can no longer share in carrying vertical loads, and (2) the removal of pore fluids increases the dead load being borne by the skeletal soil fabric. The change in in-situ porewater pressures due to fluid withdrawal, both within aquifers and in the aquicludes, is a prominent driving mechanism in large-scale land subsidence. For many years the most puzzling part of subsidence assessments was estimating the physical properties of hydraulic conductivity and compressibility of buried units at considerable depth. Many settlement assessments were largely empirical, working numbers to correspond with the observed settlements (Carrillo, 1948). Sufficient pore-water pressure measurements were eventually made as part of the Inter-Agency Committee studies in the west San Joaquin

Valley that resulted in meaningful geotechnical data on deep aquicludes and aquifers (Johnson and others, 1968). By the mid-1970s, predictive models of withdrawal-induced subsidence became sufficiently sophisticated so as to accurately predict the observed settlements (Sharp and Domenico, 1976).

In 1982 Holzer (1982a) wrote and edited a series of articles presenting the theories and experiences with assessing fluid-withdrawal-induced ground subsidence. Later, Holzer (1991) prepared a concise, but authoritative, overview of the historical development and contributions made up until that time with respect to various types of land subsidence. This article has provided geotechnical engineers with the most up-to-date compilation of fluid-withdrawal-induced subsidence.

In many arid areas of California deep ground-water withdrawal has caused widespread ground settlement that is usually accompanied by numerous open ground fissures (Holzer, 1982b). Such phenomena have been described in the geologic literature describing ground fissures in the Salt River basin around Phoenix, Las Vegas, parts of Utah, and many California desert communities, such as Rancho California (Jachens and Holzer, 1982), Palm Desert, Palm Springs, and Rancho Murrieta areas of Riverside County. Fissuring associated with land subsidence is described by many authors (this volume). This mode of arid land subsidence is in contrast to collapsing soils that occur within the vadose zone close to the ground surface (discussed in a following section).

Subsidence also occurs when the ground-water table is lowered in areas containing organic-rich soils, like peat (Bacchus, 1997, this volume; Stephens and others, 1982). Known in the literature as "biologic oxidation" (Holzer, 1991), this phenomenon occurs when organic-rich soils, such as peat beds, are dewatered. When exposed to air, peat oxidizes and the organic debris is then able to decompose rapidly. In the Sacramento-San Joaquin Delta, the peatlands were initially drained beginning in the 1850s (Leconte, 1882). During the succeeding century as much as 12–14 ft of subsidence has occurred in parts of the delta (Miller, 1966). Deverel and others (1997, this volume) compare rates of peat oxidation to land subsidence rates in the Sacramento-San Joaquin Delta. Prior to this recent work, a considerable body of English geotechnical literature described this form of settlement in the Fens area of England (Darby, 1940; Russell, 1957).

Fluid withdrawal from discrete well points for purposes of dewatering construction excavations can also lead to localized settlement of adjacent structures. In California law, persons who construct excavations at least 9 ft deep have explicit liability for any ground movements felt by adjacent structures during or shortly following construction (California Civil Code Section 832.4; CCEB, 1982; Olshansky and Rogers, 1987).

## COLLAPSING SOILS

Partly saturated native soils, such as silty sands, sandy silts, and clayey sands, in arid and semiarid areas may be subject to collapse upon initial saturation. This phenomenon was initially recognized by Cassagrande (1932). The most susceptible soils appear to be wind-blown silt and sand mixtures in areas where potential evaporation greatly exceeds rainfall. Debris flow deposits on alluvial fans in arid climes have long been recognized as prime candidate deposits for collapse (Bull, 1964). These deposits are also susceptible to collapse via another surface weathering process known as "sluicing" or "sieving" (Hooke, 1967; Rollins and Rogers, 1994). Debris flows are usually deposited with very high initial void ratios (void ratios in excess of 1.0 are not uncommon).

Jennings and Knight (1957) were the first to suggest an odometer apparatus to test for collapse potential, working with South African soils. Similar testing was then being carried out by the U.S. Bureau of Reclamation in assessing observed settlement in Nebraska, Nevada, eastern Washington, and central California (Holtz and Hilf, 1961). The problem became increasingly recognized because of the research and field tests carried out by the

Inter-Agency Committee in California's lower San Joaquin Valley for routing the California Aqueduct (Lucas and James, 1976; Bean, 1997, this volume). Even though substantive precautions were taken during construction, the canal continues to subside (Prokopovich, 1984a, b; 1989).

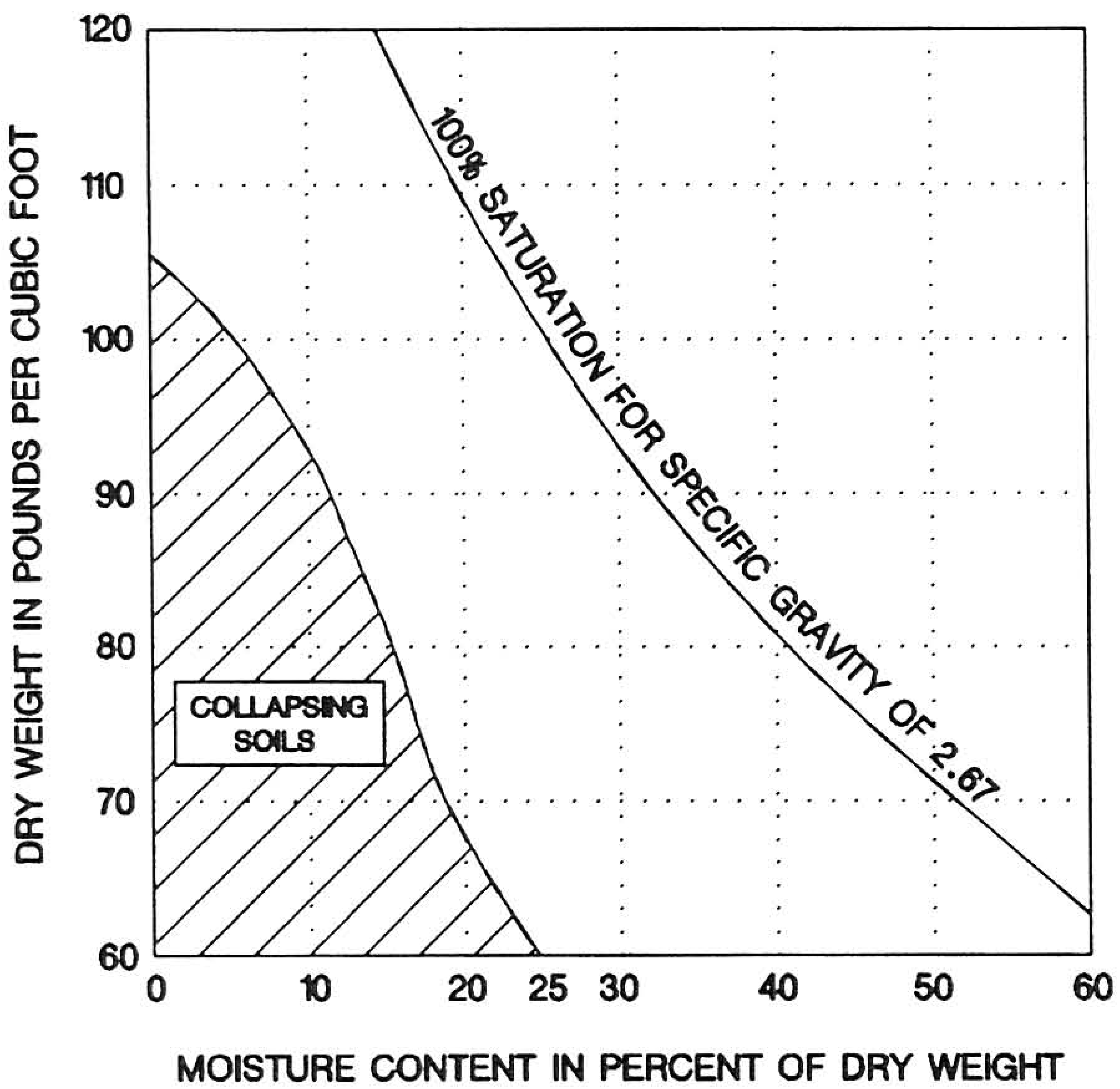



**Figure 9. Observed range of dry density and moisture content for naturally collapsing soils, as plotted on a standard ASTM compaction test diagram, taken from Evans (1985). Soils with low unit densities and less than 25 percent moisture by weight have been found to be most susceptible to collapse upon wetting in the Palm Springs area of California. Most of these soils were deposited in debris fans emanating from steep mountain fronts.**

Geotechnical engineers working outside of the western San Joaquin Valley paid little attention to the problem until Dudley's 1970 article in the ASCE *Geotechnical Journal*; this article recognized the widespread potential for collapsing soils in the more temperate climes of the southern California coastline. Dudley (1970) suggested a test methodology, using a double odometer, that has been common practice in southern California since the early 1970s. A few years later, Barden and others (1973) published a geology article outlining the various mechanisms thought to be responsible for collapse in partially saturated soils.

The key factor identifying soils with collapse potential is a loose bulky arrangement of soil particles, commonly seen as low in-situ bulk density (fig. 9). These soils also exhibit high void ratios and generally possess little or no cohesion, or interparticle cementation (geologically youthful sands and silts that have not yet undergone any meaningful manner of diagenesis). Common binders include salts (mostly carbonate and gypsum), clay films, silt, or water tension (fig. 10). When free water is introduced into the soil pores, salts may dissolve, clay and silt films soften, and there is a loss of soil suction. Cassagrande's basic concepts were eventually confirmed through research by Alwail and others (1994) using scanning electron microscopy. As the interparticle binders are lost or softened, the larger soil particles (usually sand grains) are free to slide against other, causing the structure to collapse on itself (fig. 10). The solution of gypsum beneath structures and irrigated areas is a well recognized problem in arid areas, such as Las Vegas, Nevada, and White Sands, New Mexico.

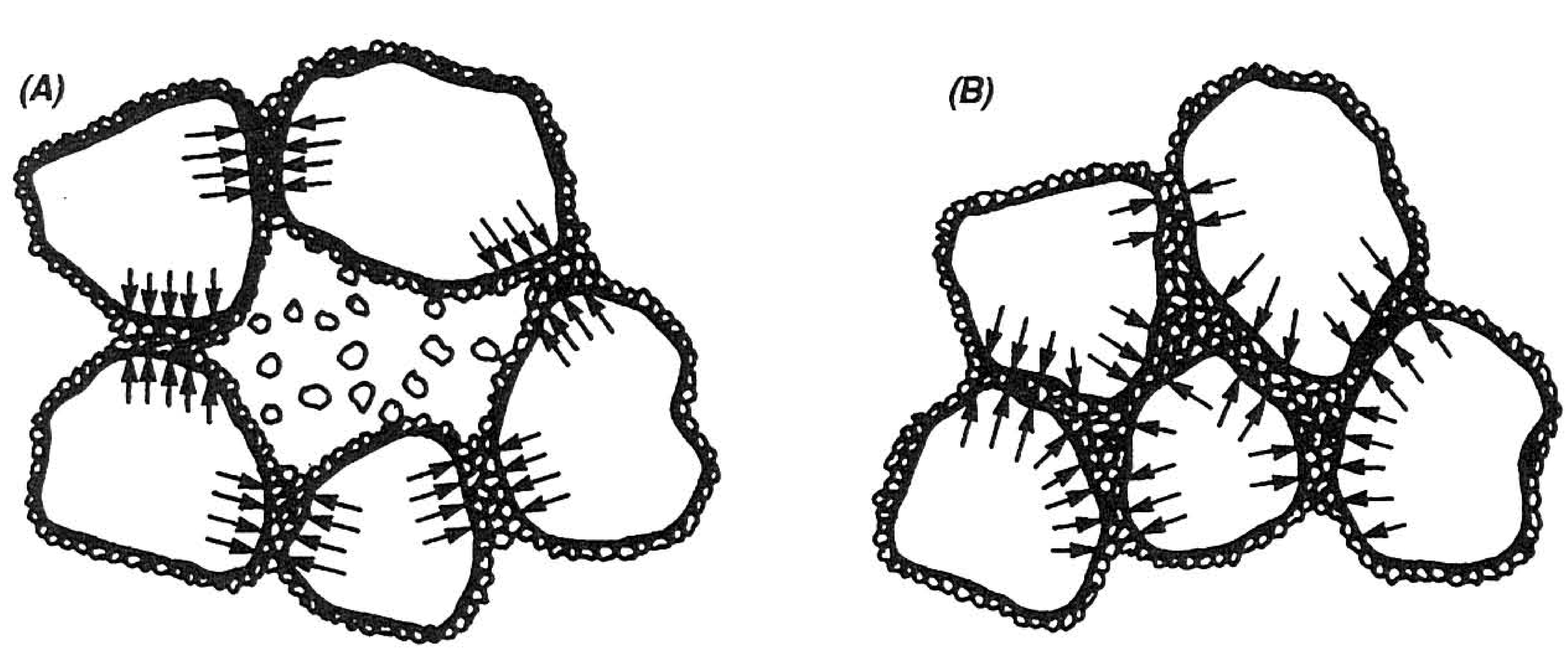



**Figure 10. Silt/clay microstructure suggested by Cassagrande (1932) (*A*) before and (*B*) after inundation (simple addition of free moisture). This basic mechanism is still believed to be responsible for hydrocompression of compacted fills as well as collapse of naturally deposited soils.**

In the late 1980s considerable attention began to be given to collapsing soils and the various mechanisms responsible for their occurrence. The prediction of collapse potential shown in figure 9 was empirically derived by Evans based on actual test data in the Palm Springs-Palm Desert area in the early 1980s (L. T. Evans, Converse Consultants, 1985, written communication). Within a decade numerous researchers sought to define the problem theoretically (for example, see Nowatzkiand others, 1989; and Karakouzian and Roullier, 1993). Among the most important articles to appear were those of Beckwith and Hansen (1989) and Houston and others (1988), that traced the phenomena in the greater Phoenix area and presented predictive procedures for their assessment. Soil collapse also caused poor performance of pavement sections in arid areas (Houston, 1988), including Albuquerque, New Mexico (Hansen and others, 1989). In the past several years additional work has continued to emanate from Arizona State University (El-Ehwany and Houston, 1990; Houston and El-Ehwany, 1991; Houston, 1992). Houston and El-Ehwany's 1991 article spawned considerable discussion. A rational testing procedure for assessing collapse potential was eventually presented by Houston and others (1993), and will likely gain ASTM certification sometime in the late 1990s.

Collapsing soils are very common in the arid and subarid regions of the American west (Beckwith and Hansen, 1989), but are also recognized in temperate coastal plains, such as Redondo Beach, Long Beach, Seal Beach, Huntington Beach, and Playa del Rey, California. Gypsum solution is widely recognized in most arid areas, especially around Las Vegas (Karakouzian and Roullier, 1993), but has also been reported in Rapid City, South Dakota (Rahn and Davis, 1996). Landscape watering and inadvertent concentration of collected runoff are the most common triggering mechanisms for collapsing soils in residential areas. Houston and Houston (1989) presented work on what types of mitigation measures have been employed when collapse potential is identified.

## HYDROCOMPRESSION

Hydrocompression is a new engineering term introduced by Brandon and others (1990) to describe observed settlement of compacted fill embankments upon their initial absorption of free moisture. The term differed slightly with that of "hydroconsolidation" introduced by Krynine and Judd (1957), and "hydrocompaction" introduced by Prokopovich (1963) and Lofgren (1965). Until recently, "hydrocompression" had been used interchangeably with the term "collapsing soils" or "field collapse" (Houston and others, 1988). These terms are now generally applied to the same phenomenon in naturally deposited soils, as opposed to compacted or engineered fills.

Within just the past 6 years geotechnical engineers have come to recognize and respect hydrocompression of compacted fill embankments. Soil mixtures of clayey sand and sandy clays appear most susceptible to hydrocompression, and the percent change in volume is clearly a function of overburden pressure (Lawton and others, 1989; Brandon and others, 1990). Clayey soils tend to swell when wetted at low applied stresses and compress when wetted at high stresses, as depicted in figure 11. Basma and Tuncer (1993) subsequently alleged that well-graded soils (different grain sizes) were more susceptible to hydrocompression-induced collapse than poorly sorted (all one grain size) mixtures of granular soils. This subject was discussed further by Alwail and others (1994).

Hydrocompression and hydroswell commonly affect compacted fill embankments in the urban metropolitan areas of California (Lawton and others, 1989, 1991a, b; Brandon and others, 1990; Noorany and others, 1992; Rogers, 1992a; Vicente and others, 1994; Noorany and Stanley, 1994; Kropp and others, 1994). The amount of swell or compression appears to depend on six principal factors:

(1) Fill soil mineralogy (such as expansivity)
(2) Moisture content at the time of initial compaction

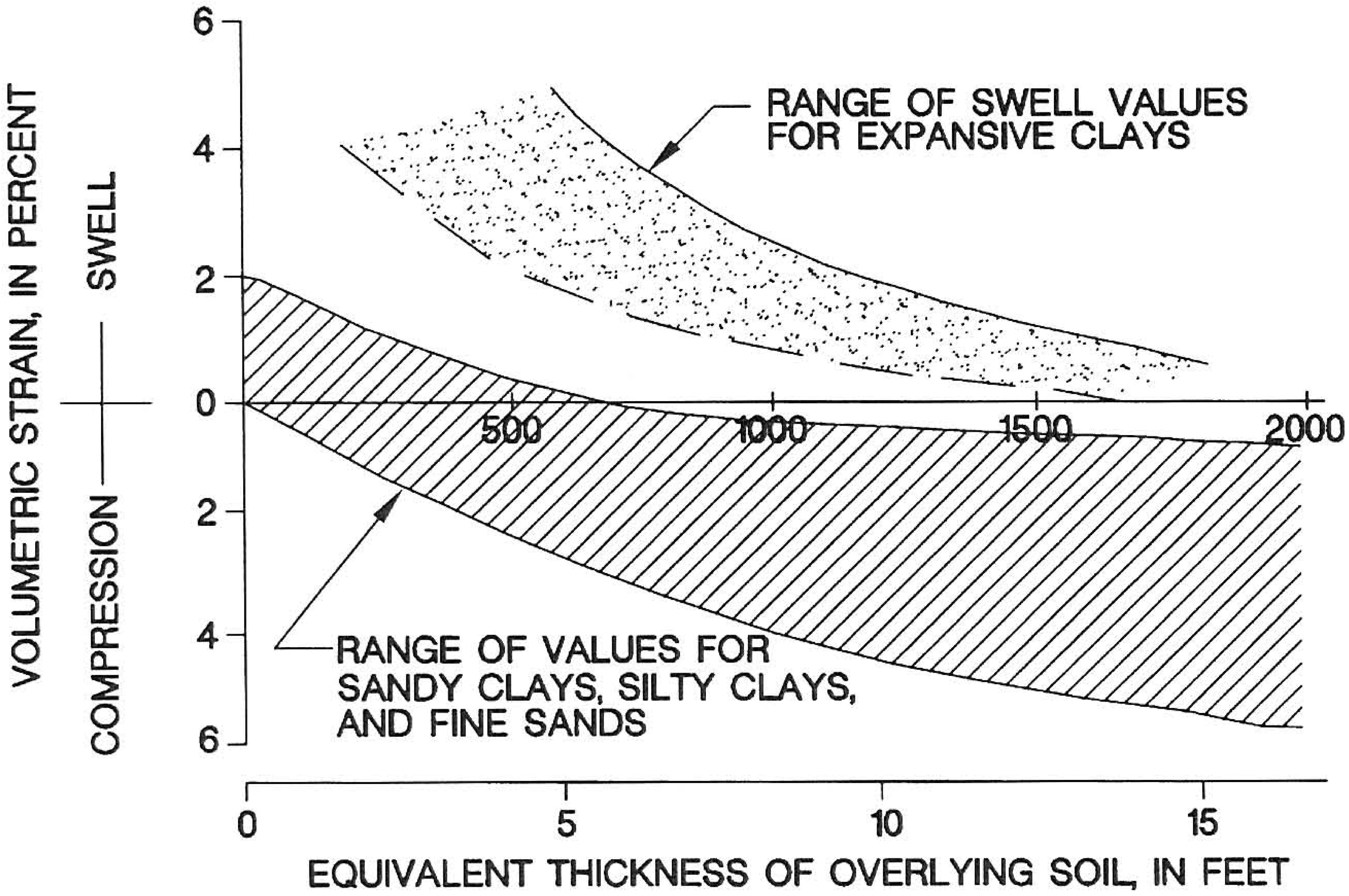



**Figure 11. Range of swell and compression recorded in odometer tests of expansive clays, silty/sandy clays and nonexpansive fine-grained sands. Inundation of the samples began at 50-percent saturation.**

(3) Bulk density of the compacted soil at the time of placement

(4) Amount of time the fill embankment is allowed to dry or desiccate before landscape watering or rainfall begins to affect the fill

(5) Amount of fill allowed to swell before additional surcharging (adding of load)

(6) Potential presence of buried fluid permeability boundaries, such as sudden changes in soil type within the embankment, that can serve to concentrate seepage and thereby cause differential performance

These items are the author's conclusions; not everyone who has studied the problem agrees on the methods of testing or analysis to assess hydrocompression. Lawton and others (1989, 1992) used double odometer tests (as did their predecessors, Jennings and Knight, 1957; and Dudley, 1970), wherein one sample could be wetted, then loaded, wetted, and loaded again, so as to construct a family of data points relating measured volume change with increasing normal force after wetting. These articles generated no small degree of controversy. Noorany (1992) maintained that one cannot employ double odometer tests to replicate actual field situations, where the fill can be loaded for many years prior to the onset of wetting (and wetting fronts may repeatedly migrate through a mass). Noorany believed that the stress path engendered by loading exerts a significant influence on the test results (like Lawton and others, he also cites the work by Justo and others, 1984). The engineering geology literature reached similar conclusions with regards to employing the single odometer test regimen (Reznik, 1994). Note that much of Noorany's experience included expansive soils that would be more sensitive to the load factors and wetting mechanisms he discussed whereas those evaluated by Lawton and others tended to be granular soils whose response to wetting cycles is not always as complex as that of clayey mixtures.

Lawton, Fragaszy, and Hardcastle's 1989 article stirred comment from other quarters (Day, 1991; McRae, 1991; Lawton and others, 1991a, b), as did that of Lawton, Fragaszy, and Hetherington (1992). That article contained the most complete bibliography on hydrocompression to date. Vilar (1994) asserted that compacted fills seldom ever reach absolute saturation, but that increases in moisture content after compaction were often ascribable to matrix suction effects (Escario, 1973; Escario and Saez, 1973). In their response to discussions, Lawton and others (1991a, b) cited the phenomenon of partial collapse associated with partial saturation, conceding to this aspect of the problem brought out by others.

Thornthwaite (1948) introduced the concept that soils subject to steady infiltration tend to absorb moisture until "field capacity," a quasi-equilibrium condition, is achieved. Soils within the rooted zone can lose moisture through evapotranspiration until the wilting point is reached, below that point the soil will lose no further moisture. In 1957, Thornthwaite and Mather (1957) built upon this concept to introduce water balance assessments that estimate infiltration as the difference between precipitation and runoff minus interflow (gravity seepage out of the zone of infiltration back to the ground surface). Seasonal losses via evapotranspiration could then be manipulated to ascertain whether net recharge of the soil mass should occur below the rooted zone. The Post-Tensioning Institute (1980) and Lytton (1994) had applied the concepts of field capacity and wilting point with soil suction to predict expansive soil behavior. Rogers (1992a) and others asserted that most fill soils obey these basic tenets of agricultural soil science in that they tend to absorb moisture until the quasi- "field capacity" of Thornthwaite and Mather (1957) is achieved.

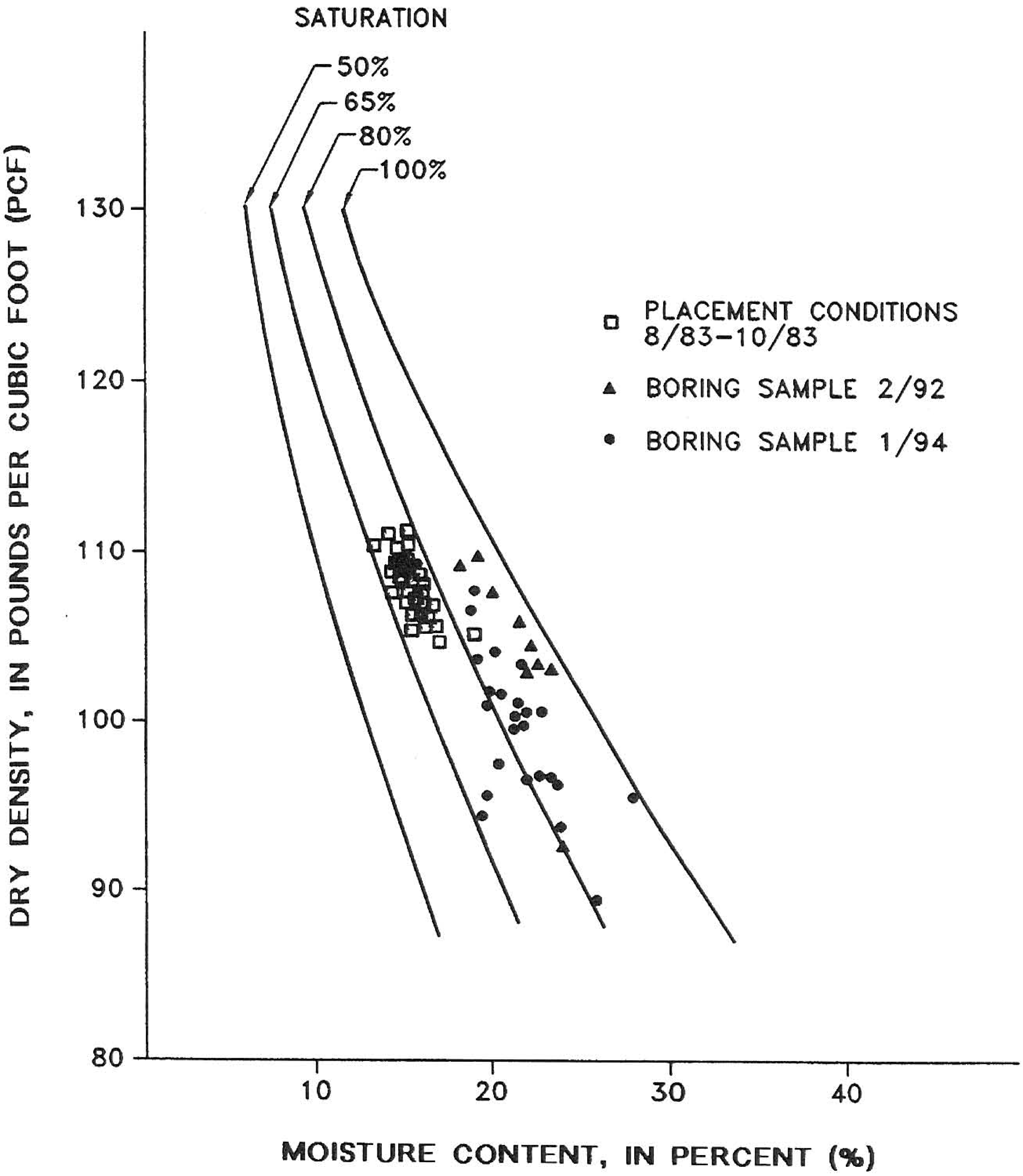
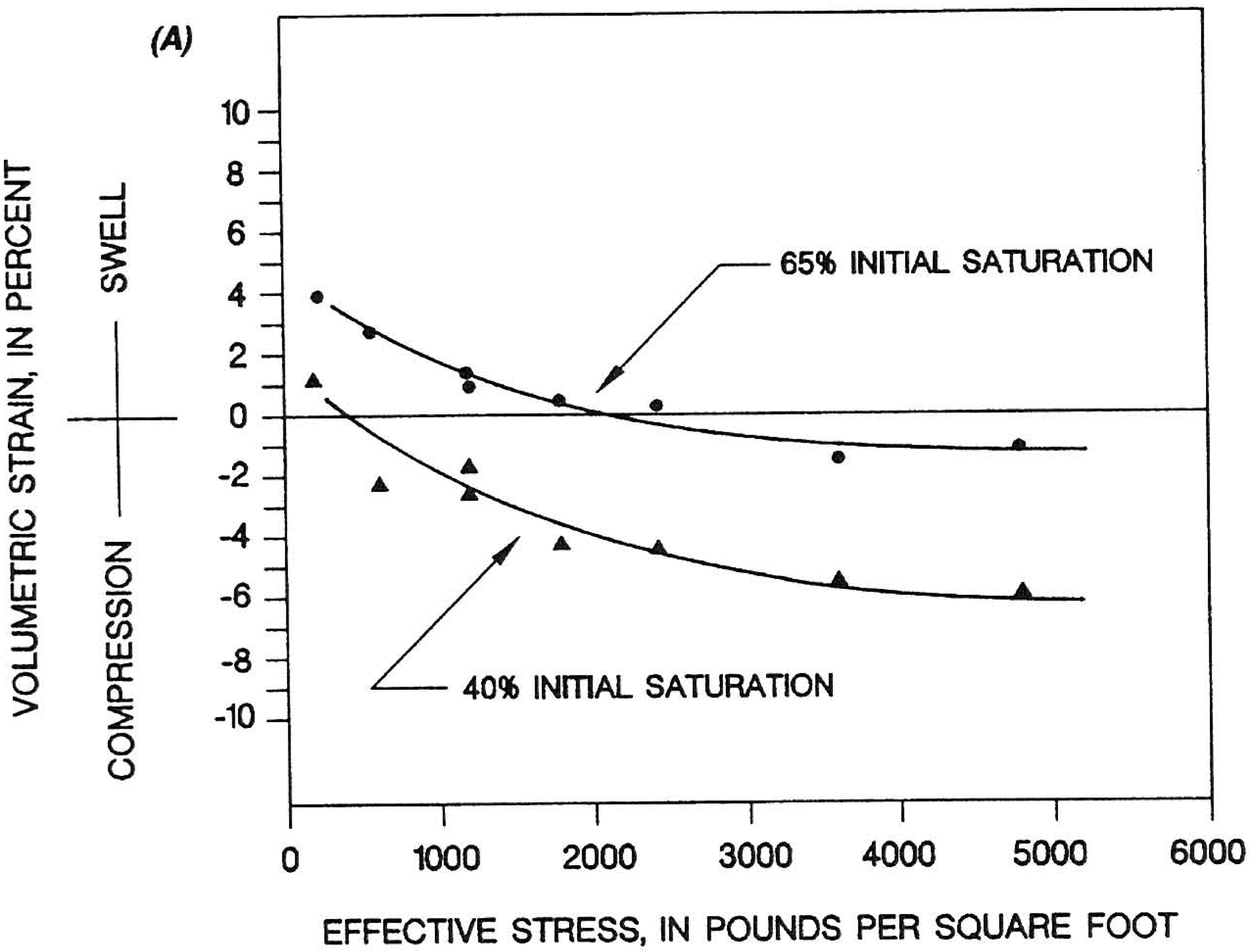
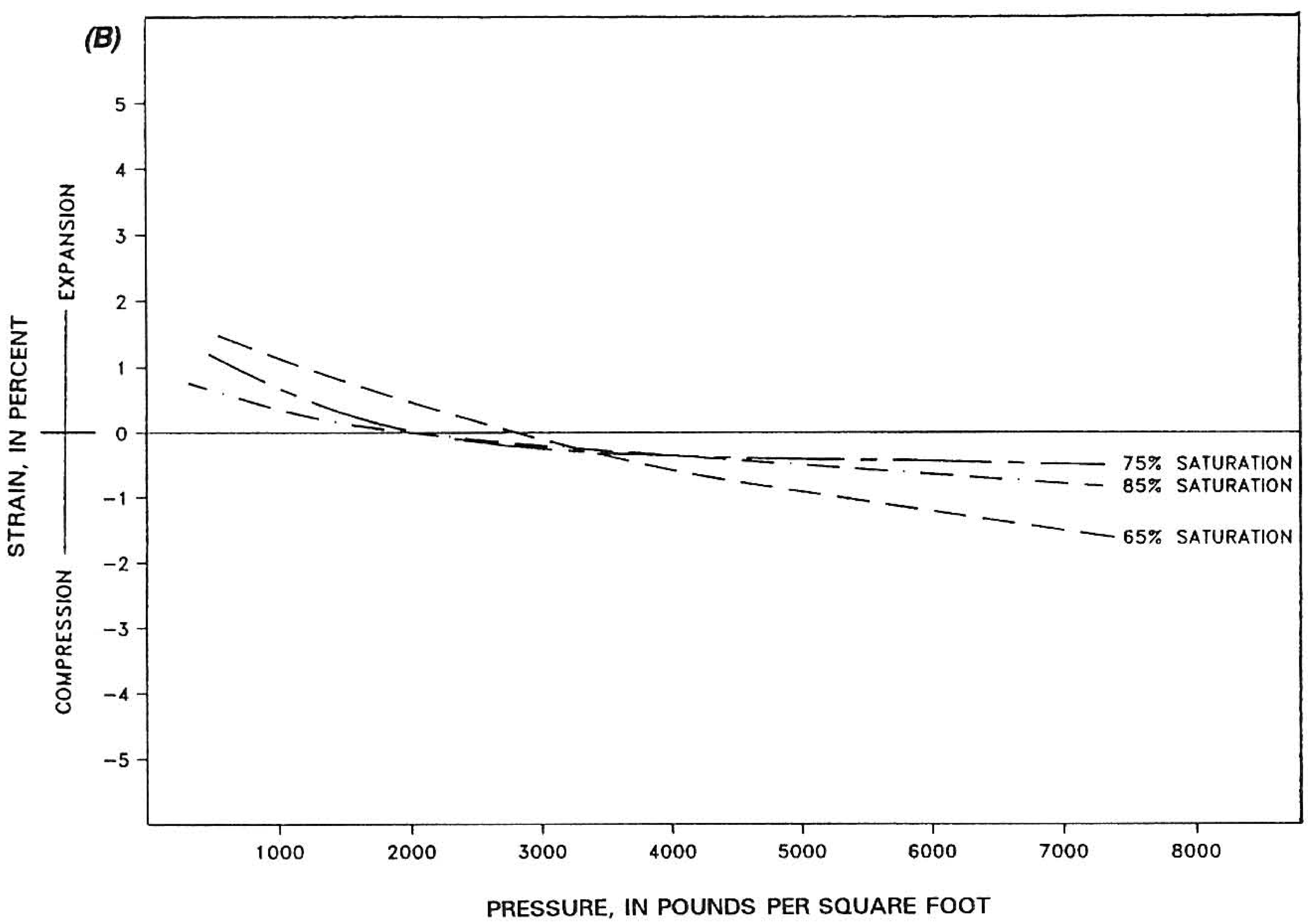



**Figure 12. Comparison of soil moisture and density at time of compaction and approximately 10 years later. As fill soils absorb moisture and swell, their bulk density is decreased. Moisture absorption in fills compacted near optimum moisture content is typically around 5–7 percent over the first 10 years, but can be greater. Lines of equivalent saturation are presented in relation to the theoretical line of zero air voids, that corresponds to complete saturation. Fill soils seldom sustain a saturated condition, generally reaching a point of quasi-equilibrium somewhere between 80 and 95 percent saturation.**

In compacted fills containing an appreciable silt or clay fraction, this quasi-equilibrium state of moisture absorption (field capacity) generally corresponds to something between 80 and 95 percent saturation (fig. 12). Most fill soils reach this "field capacity" as a long-term quasi-equilibrium condition, provided the soils are more-or-less continuously irrigated and sufficiently insulated from evapotranspiration losses (generally, that fraction of the fill greater than 10 ft deep). During periods of sustained precipitation, pore pressure "waves" (transient saturation) will infiltrate and travel through the fill, temporarily raising moisture levels (and pore pressures) to saturation, but never keeping it saturated. However, in a number of documented cases, discrete zones of saturation may persist in certain zones of the fill, such as areas of permeability contrast with low head differential, such as the bottom of fills above their basal contact with underlying bedrock, or close to the ground surface where near-constant recharge occurs from landscape watering and rainfall/runoff concentration (Rogers, 1992a).




(A)




(B)

**Figure 13. Variation of hydrocompression/hydroswell with initial water content and increasing confinement of (*A*) sandy silt fill material. Note how the beginning moisture content influences the amount of measured compression, with the wetter fills having much less tendency to hydrocompress. (*B*) Fill materials derived from the expansive Capistrano and Monterey formations in Orange County, California. These lines represent the mean loci of a considerable body of data. Note how the percent compression decreases with increasing initial moisture content, although not in a linear fashion.**

Figure 11 presents hydroswell/hydrocompression data for both expansive and nonexpansive soils, compacted at 65 percent of saturation (slightly dry of optimum moisture content; see figs. 1 and 2). Note that expansive clay will swell at shallow depths due to low confinement. When fills containing expansive clay are deeper than 15–40 ft (depending on soil plasticity), they usually consolidate upon initial saturation, and thereby may have less tendency to hydrocompress due to subsequent load surcharge than more lightly loaded zones that are lying above. Compaction of the deeper sections of large fill embankments at higher than optimum moisture contents could be expected to lessen the tendency toward hydrocompression-induced settlement.

Precise evaluation of hydrocompression can be complicated by postconstruction drying, due to subaerial exposure in climates with long dry spells, like California. If the fill is allowed to dry, moisture contents in the uppermost 5–10 ft of material may be lowered. The effect of such drying on a sandy silt fill would be to exacerbate hydrocompression within the affected zone (fig. 13*A*). Figure 13*B* presents a typical example of an expansive soil mixture, where the lines of equal initial percent saturation may overlap each other with increasing confinement; this scenario is not as simple as the situations depicted in figures 11 and 13*A* because soils under greater and greater levels of confinement will compress, and thereby have less tendency to undergo hydrocompression when absorbing moisture.

## HYDROSWELL PHENOMENA

"Hydroswell" is a colloquial term used to describe the swelling of expansive clay embankments that occurs upon initial saturation. By definition, expansive soils will tend to swell, thereby increasing their volume, with the addition of moisture. Swelling can be observed in odometer tests, such as those presented in figures 11, 13, and 14. The amount of hydroswell is a function of three components: (1) soil mineralogy and plasticity, (2) initial water content (that influences available suction), and (3) the confining pressure acting upon the soil (Seed and others, 1962). All other factors being equal, the amount of measured hydroswell is generally a function of effective normal stress (confining pressure), as shown in figure 14. The swell potential of an expansive soil, therefore, is intimately tied to the effective confining pressure exerted upon the soil and its initial water content. In pavement design, thick layers of gravel subbase are used to provide dead load confinement, thereby diminishing the swell potential of the underlying soil. Because of this relationship between confinement and swell potential, shallow expansive soils can be expected to swell more than the same soil buried at greater depths.

It has long been recognized that the postconstruction behavior of compacted fills is influenced by the initial soil moisture content, at the time of fill placement (Seed and others, 1962; Lytton, 1994), as shown in figure 14. This is why most geotechnical engineers recommend that expansive soils be emplaced at 2 percent over optimum moisture content, whenever possible (Seed and others, 1962). After construction is complete, the moisture content of expansive soils in pavement subgrades (Monismith, 1990) and embankments (Noorany and Stanley, 1994; Rogers, 1992a) usually increases and bulk density usually decreases.

In 1992, Day asserted that there were three main factors responsible for the degree of swell experienced by a compacted clay: the surcharge (confining) pressure, compacted dry density, and the moisture content at the time of compaction. But, Day (1992a) also asserted that the method of compaction, curing time, sample size, and duration of the swell test also accounted for differences in measured values of swell. Day (1992a) went on to explore the relation between the degree

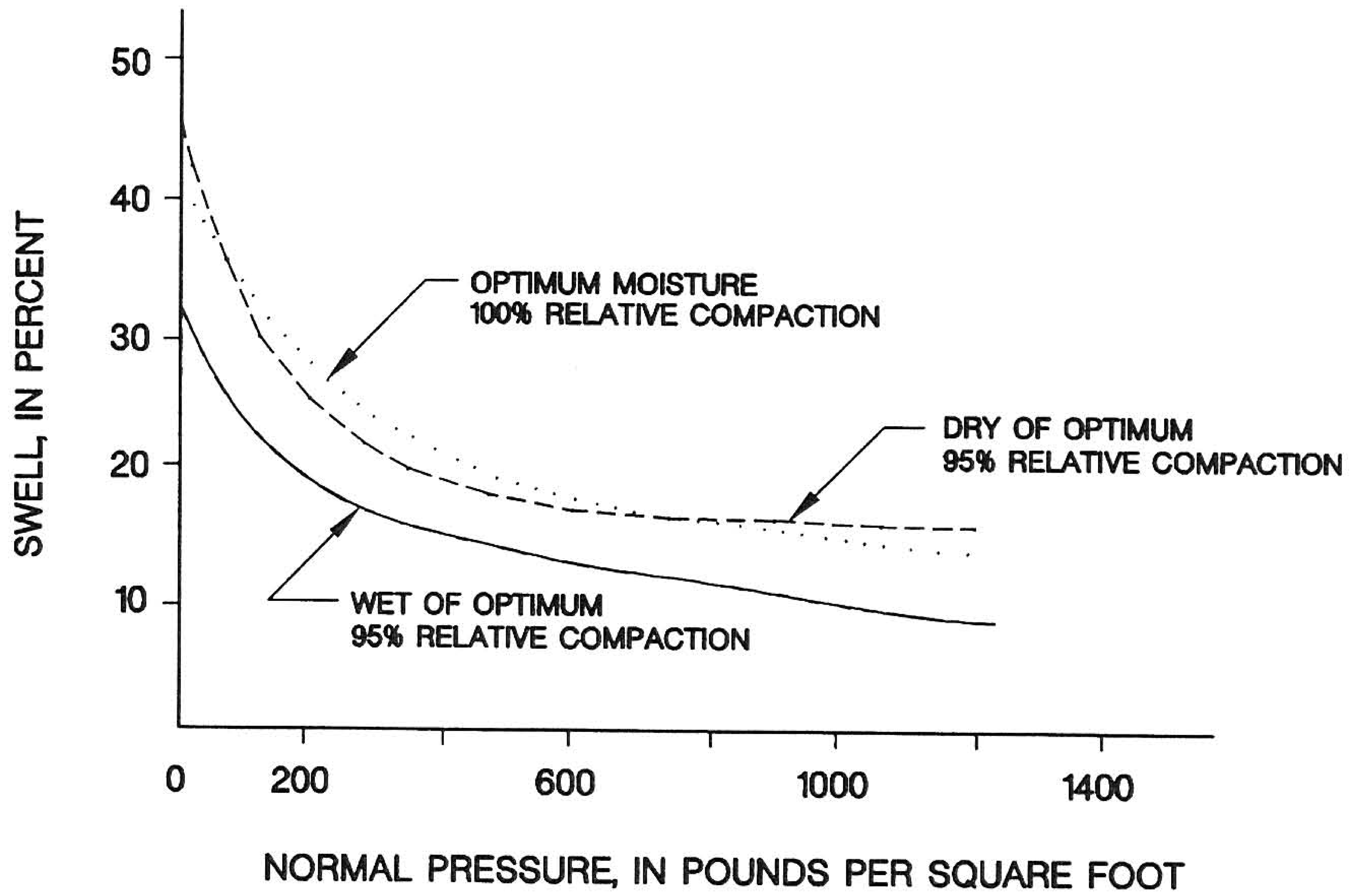



**Figure 14. Relation between swell of a sandy clay and confining pressure, with varying initial water content. Soil samples were compacted to 95 percent relative compaction dry and wet of optimum moisture content (fig. 2), as well 100 percent compaction (at optimum moisture content). Taken from Rogers (1992a). Note how a wet, dispersed structure (fig. 2) results in less swell. For that reason most engineers recommend that expansive soils be compacted with a moisture content at least 2 percent higher than optimum.**

of saturation and swell in compacted clay. Much of his results mirror those presented in figure 14. Engineers recognize a phenomenon termed "primary swell," that is, that cycle of swelling that generally occurs on the compacted soil's first exposure to free water, where absorption occurs via osmotic, capillary, and suction mechanisms. Phatak and Kanitkar (1994) presented examples of plotting percent swell on log-log plots to better identify the end of the primary swell cycle. This distinction is important to recognize to draw conclusions about how much additional swell may occur because the after-swell volume is greater than the initial volume, making the calculation of full saturation somewhat problematic. Day (1994) closed discussions of his 1992 article by paraphrasing Chen (1988) when he stated that additional swell cannot occur without introducing additional moisture, ignoring load rebound or absorption of water vapor (that does occur beneath slabs and pavement; see Tucker and Poor, 1978).

The ability of expansive soils to drain themselves past the "wilting point" of Thornthwaite and Mather (1957), and thereby desiccate, then reenter another cycle of swelling is one of the vexing problems associated with structures founded on expansive soils. This attribute of expansive soils makes the prediction of future behavior difficult in that clay can "remember" its own stress history, overconsolidate when desiccated (Mayne and Kulhawy, 1982), and repeatedly change volume when exposed to varying moisture levels.

When embankments comprised of expansive clay fill are subjected to long-term wetting, their response is usually fivefold:

(1) The uppermost 15–40 ft of the fill usually swells, exerting vertical uplift forces on flat-lying pads and lateral pressures in proximity of slopes (Noorany and others, 1992).

(2) Expansive soils buried more than 15–40 ft deep tend to compress and consolidate upon absorption of moisture, up to a point of quasi-moisture equilibrium, commonly referred to as "field capacity." This absorption of moisture within the vadose zone can be quite rapid, generally within 5–10 years after the onset of landscape watering (Noorany and Stanley, 1994).

(3) If expansive clays are allowed to dry out, subsequent cycles of wetting will cause a repeat of the swelling process that may result in appreciable shrink-swell related damage (Chen, 1988).

(4) The expansive clay soil will lose shear strength with long-term saturation (Morganstern and Eigenbrod, 1974; Day and Axten, 1990; Stark and Duncan, 1990; Rogers, 1992a).

(5) If water levels are allowed to cycle up and down in an embankment, the load cycling may cause a net decrease in soil cohesion, thereby affecting soil shear strength and the overall stability of an embankment (Stark and Duncan, 1990).

The net result of hydroswell is most obvious in proximity to slopes (Noorany and others, 1992), as presented diagrammatically in figure 15. When near-surface expansive soils are wetted, the resulting swell will generally be directed upward on level lot pads and outward on slopes. The horizontal component of this outward movement may cause tension to be

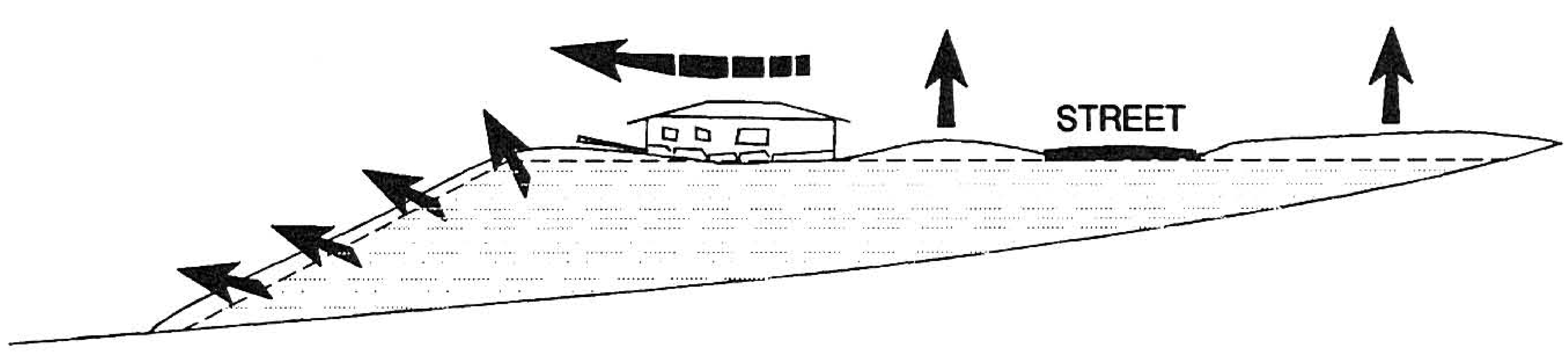



**Figure 15. Outward swell of sidehill embankment fill, due to hydroswell of expansive clay. Outward-directed strain can serve to pull structures apart that are situated in proximity to the affected slope. Asymmetric landscape watering can also serve to exacerbate such tendencies, causing damage to landscape improvements and structures.**

accumulated in structures founded in proximity of the slope face.

Not everyone in the engineering community has agreed with the assertions of Noorany and others (1992, 1994). Jensen and Kasim (1994) pointed out that the soft Tertiary sediments that comprise the source materials for most northern California fills are subject to air and water slaking, and they questioned the scale of Noorany's test specimens. Jensen and Kasim also presented a corollary example using Terzaghi's diffusion equation, taken from finite element analyses for earthquake-induced deformations of earthen embankments. They pointed out that the bulk and shear soil moduli to model partially saturated and saturated soils are data that are generally lacking in hydrocompression assessments to date. However, Noorany (San Diego State University, 1996, oral communication) demonstrated that he can measure soil deformability parameters on unsaturated soil specimens, and that his theoretical predictions have been verified by measurements of field performance.

We can expect continued research in the area of long-term behavior of partially-saturated embankments (Fredlund and Rahardjo, 1993). One of the most troubling aspects of performing such analyses is the relative paucity of reliable experimental data on the deformational properties of partially saturated soils. Prior to research work by Noorany and the author, volumetric strains were only measured from pore water expelled from saturated specimens under triaxial loading. In addition, in-situ instrumentation of large urban fills (as well as landslides) suggests that pore pressure "waves" move through the masses during periods of extended precipitation, temporarily changing the degree of saturation and the deformability properties (Iverson and Major, 1987; Baum and Fleming, 1991). Those deep fills that are saturated have shown marked sequences of additional compression/swell-induced settlement following sustained periods of precipitation. The fills then "returned" to quasi-field capacity water contents and relative "dormancy." This makes the absolute prediction of "no more settlement" a somewhat risky venture in that one cannot control whether or not additional pulses of moisture travel through the fill. The long-term effects of such pore pressure "cycles" needs to be explored further.

Recently published primary references on expansive soils include Chen's second edition of *Foundations on Expansive Soils* (1988), Nelson and Miller's *Expansive Soils* (1992) and Lytton's 1994 article. Other important references are included in the proceedings of the 7th International Conference on Expansive Soils held in Dallas, Texas, in August 1992. Expansion of shale bedrock can also cause structural distress greater than that ascribable to the adjoining expansive soil regolith (Meehan and others, 1975). Meehan and Karp (1994) addressed the various aspects of residential distress associated with construction upon expansive soils and how a portion of that may actually be ascribed to structural details.

## PSEUDO-SUBSIDENCE

Other mechanisms may work to create the impression that land subsidence is occurring owing to changes in soil-moisture content. Two of the most common errors in interpretation of land-surface deformation are pseudo-settlement ascribable to heaving of overconsolidated shale, and dynamic consolidation of embankments, usually caused by earthquakes. Either of these mechanisms can cause significant changes in the void ratio and bulk density that may affect the future response of the land surface to variations in subsurface moisture content.

### Pseudo-Settlement Owing to Shale Heave

The removal of loads by excavation may cause reductions in effective confinement that promote apparent upward movement, or swelling, due to stress relief. This phenomena has long been recognized in areas where shale has been unloaded (Hollingworth and others, 1944). Shale rebound was initially reported

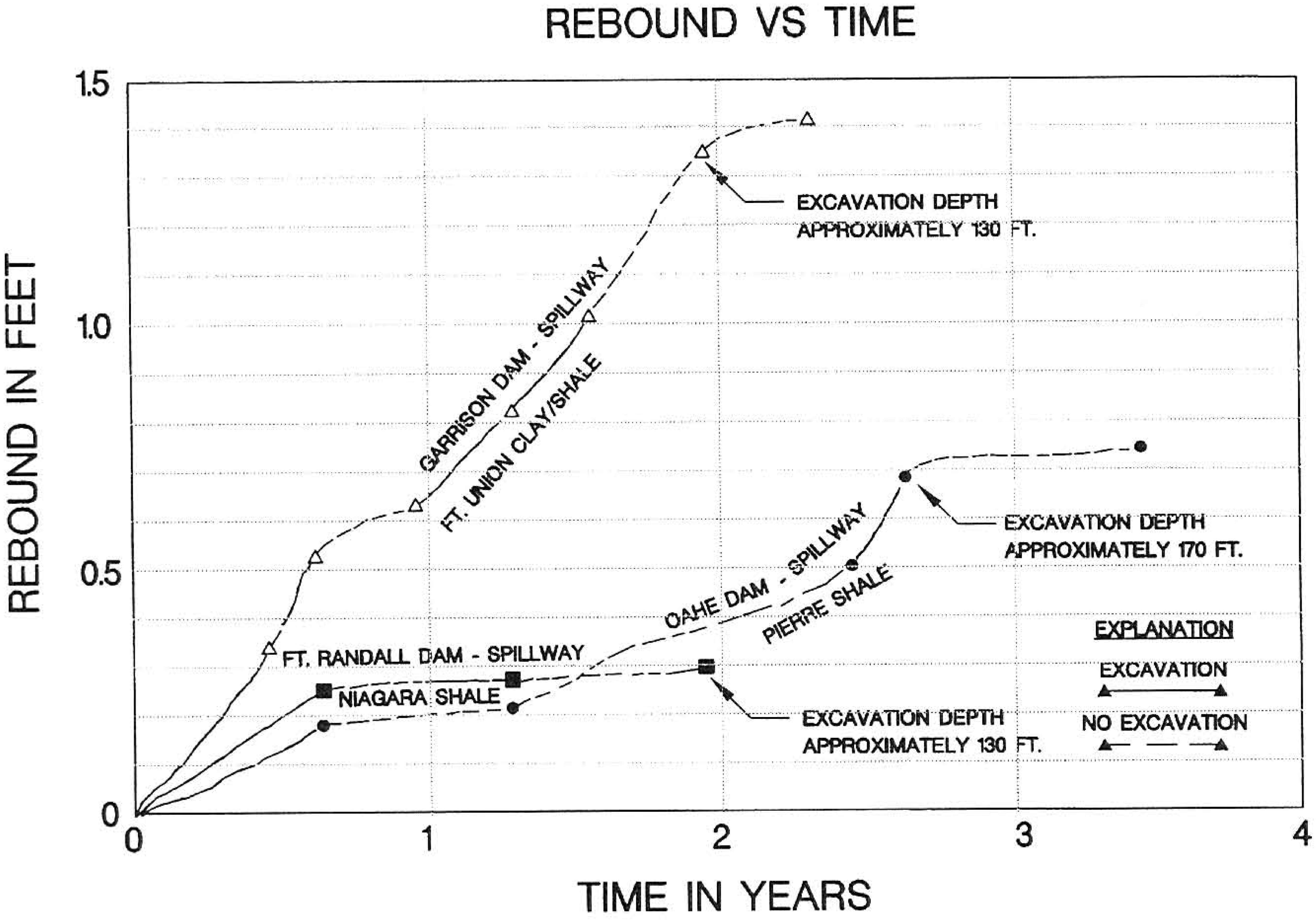



**Figure 16. Measured rebound of different shales after excavation from Garrison Dam, Ft. Randall Dam, and Oahe Dam, taken from Trantina (1964). Note how rebound was measured both during and following excavation. This same phenomenon was described earlier by Crosby (1939) working on excavations for dams in New Mexico and West Virginia. In many instances, settlement is alleged when, in fact, differential swell is occurring.**

upon in excavations in interbedded shale and limestone made at Tygart Dam in West Virginia and later at Conchas Dam in New Mexico, as reported by Crosby (1939). Cretaceous shales containing thin beds of volcanic ash altered to the swelling clay mineral montmorillonite have long been recognized for their ability to rebound after initial excavation (Smith and Redlinger, 1953; Peterson, 1958). Other types of shale are susceptible to this phenomena (Trantina, 1964; Ferguson, 1967; Twidale, 1974; Meehan and others, 1975; Young, 1988). Price (1966) suggested that rebound in shale was ascribable to the release of stored elastic strain energy, a theorem that has been generally accepted. Figure 16 presents some representative values of shale rebound measured in excavations for Corps of Engineers' dams in the Missouri River basin. Moran (1989) presented five models by which shale heave occurs, including load removal, gypsum growth, sulfate alteration, biochemical expansion, and water absorption. Recently, Dhowian (1990) presented a simplified model for predicting heave of expansive shale; his model was based on his experience in the Middle East.

## Dynamic Consolidation

Dynamic consolidation can occur whenever a soil or embankment mass is subjected to strong seismic shaking. Earthquake-induced settlement of dams, levees, and highway and urban embankments has long been recognized. Newmark's (1965) pioneering work suggested a method by which a rational deformational analysis could be performed. Newmark's method required specification of the dynamic loading induced by a model earthquake. Using this ground motion input, a two-step analysis is then performed. The first step is to obtain the embankment response to the input ground motion, and the second step consists of making displacement calculations of potential sliding masses. The U.S. Bureau of Reclamation has prepared a simplified procedure for performing the deformational analysis on computer (Makdisi and Seed, 1978). The difficult part of the Newmark method lies in defining the dynamic properties of existing embankments when such data may not exist.

Virtually any embankment subjected to strong seismic motion can suffer some measure of permanent deformation. The amount of lateral embankment deformation, or "lurching," depends on the plasticity of the embankment soils (Seed, 1973). Plastic soils tend to undergo greater total deformations, and stiff soils tend to rupture, leaving ground cracks. Ground cracks can damage overlying

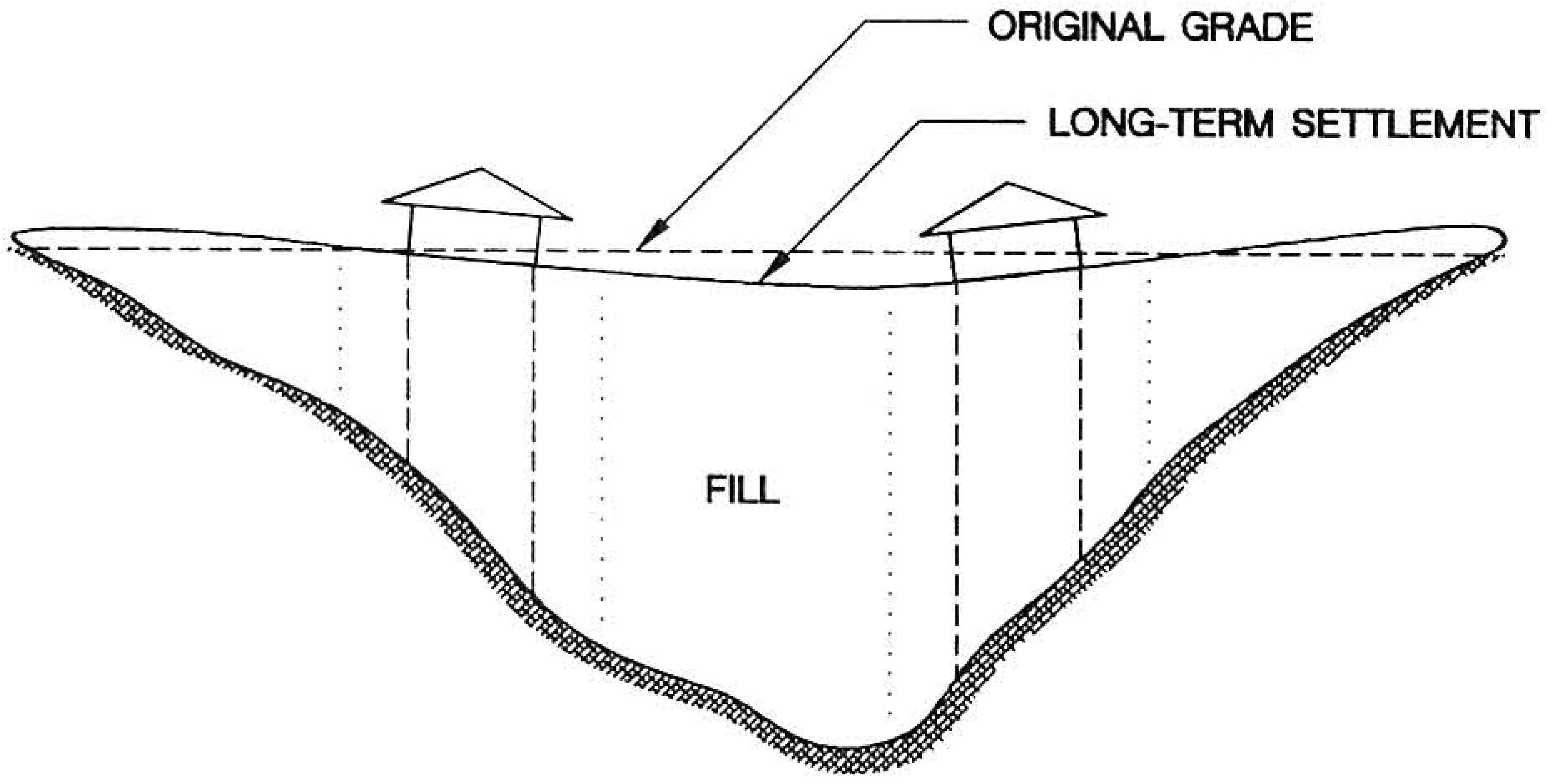



**Figure 17. Schematic representation of dynamically induced compaction of a valley fill. Classic settlement of the deep fill causes extension of near-surface elements. Taken from Rogers (1992b).**

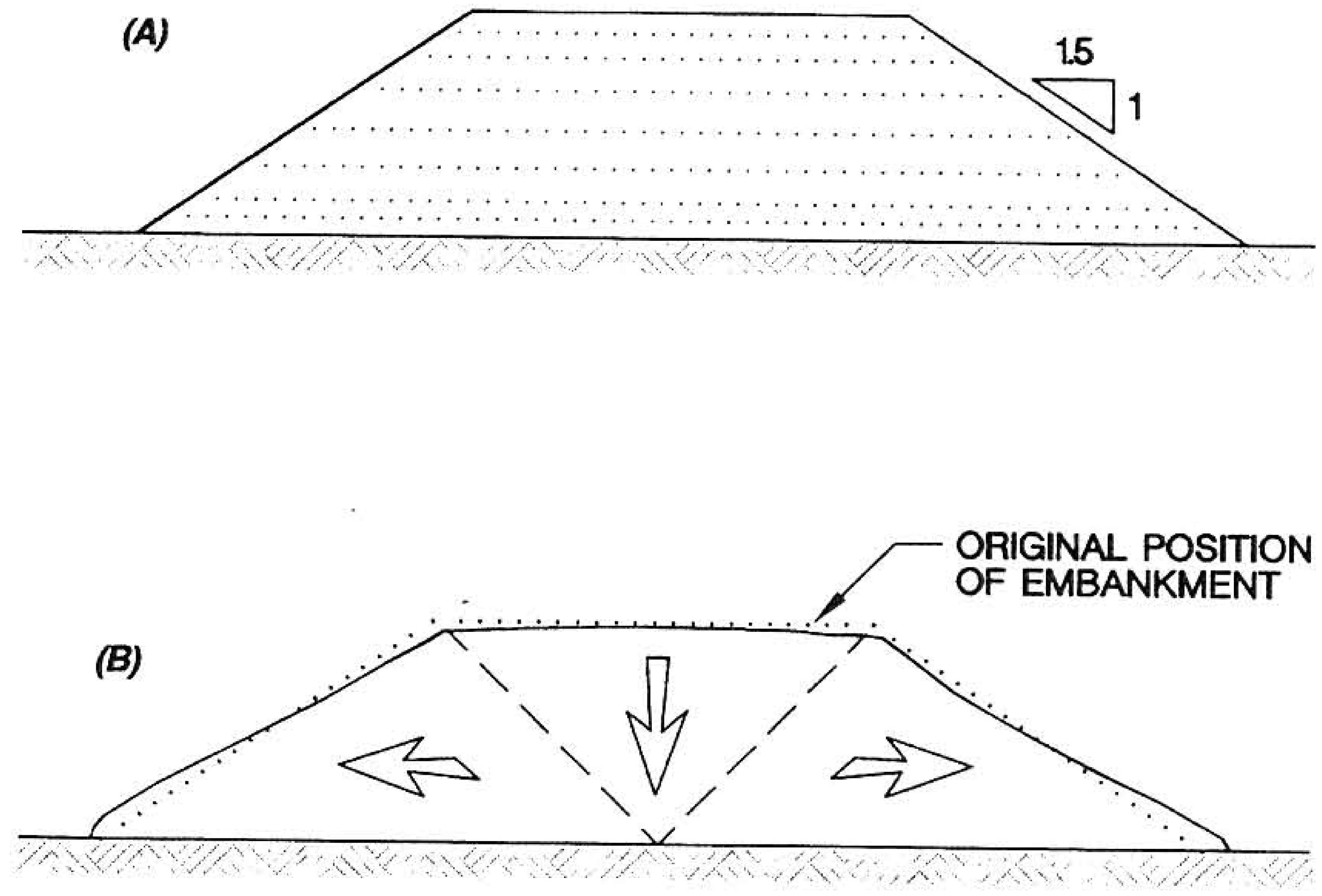



**Figure 18. Cross section of a highway bridge abutment approach fill, as (*A*) initially constructed with 1. 5:1 side slopes and (*B*) showing the reaction wedges that develop in response to vertical acceleration during an earthquake, with the roadway appearing to settle relative to the nonyielding bridge structure. Such settlement may not be due to particle consolidation as much as lateral movement.**

structures and pose grave danger to water-retention structures.

In granular embankments comprised chiefly of silts, sand, or gravels, seismic shaking can be expected to cause volumetric consolidation (fig. 17). This lowering of the soil's void ratio usually leads to immediate consolidation, although many workers suspect that lateral bulging is responsible for much of the observed settlement, as is shown in figure 18. The lower the water table, the greater the observed deformation (although heightened water tables may engender liquefaction, a more serious problem). The amount of dynamically induced settlement varies according to the soil's bulk density prior to the quake. In 1959, the upstream shell of Hebgen Dam in Montana settled 3.4–5.4 ft (on an embankment 110–140 ft high) during a magnitude (M) 7.5 earthquake. This rockfill had been sidecast (loosely dumped, without benefit of mechanical compaction) into the reservoir pool when the dam was originally constructed in 1915 adjacent to a reinforced concrete core wall (fig. 19). This manner of uncompacted placement could be expected to produce a low-density fill, susceptible to dynamic consolidation.

In large urban fill embankments, granular soils have been mechanically compacted in near-horizontal layers, called "lifts," which seldom exceed 8 in thick. Mechanically compacted embankments tend to fare better in earthquakes, depending on topographic position. For instance, during the M 6.9 Northridge earthquake near Los Angeles, the Hansen Flood Control Basin embankment (the world's first dry compacted embankment in 1944) settled between 1 and 3 in, but embankments of similar depth situated high in the nearby Santa Monica Mountains settled nearly 12 in. This variance appears to be due to embankment shape and topographic position; steep-sided ridgelines experiencing greater accelerations than lower slopes on less steep gradients. This factor, commonly referred to as "topographic enhancement," has been widely recognized since the 1971 San Fernando earthquake (Rimer, 1973; Chang, 1976 ).

## CONCLUSIONS

Given the available data being collected on long-term performance of compacted fill embankments, the following conclusions suggest how fill construction methodologies might be modified in the coming decade.

- Every effort should be made to compact fills at the highest water content possible, hopefully in the range of 2–4 percent over optimum moisture content (as defined in fig. 2).

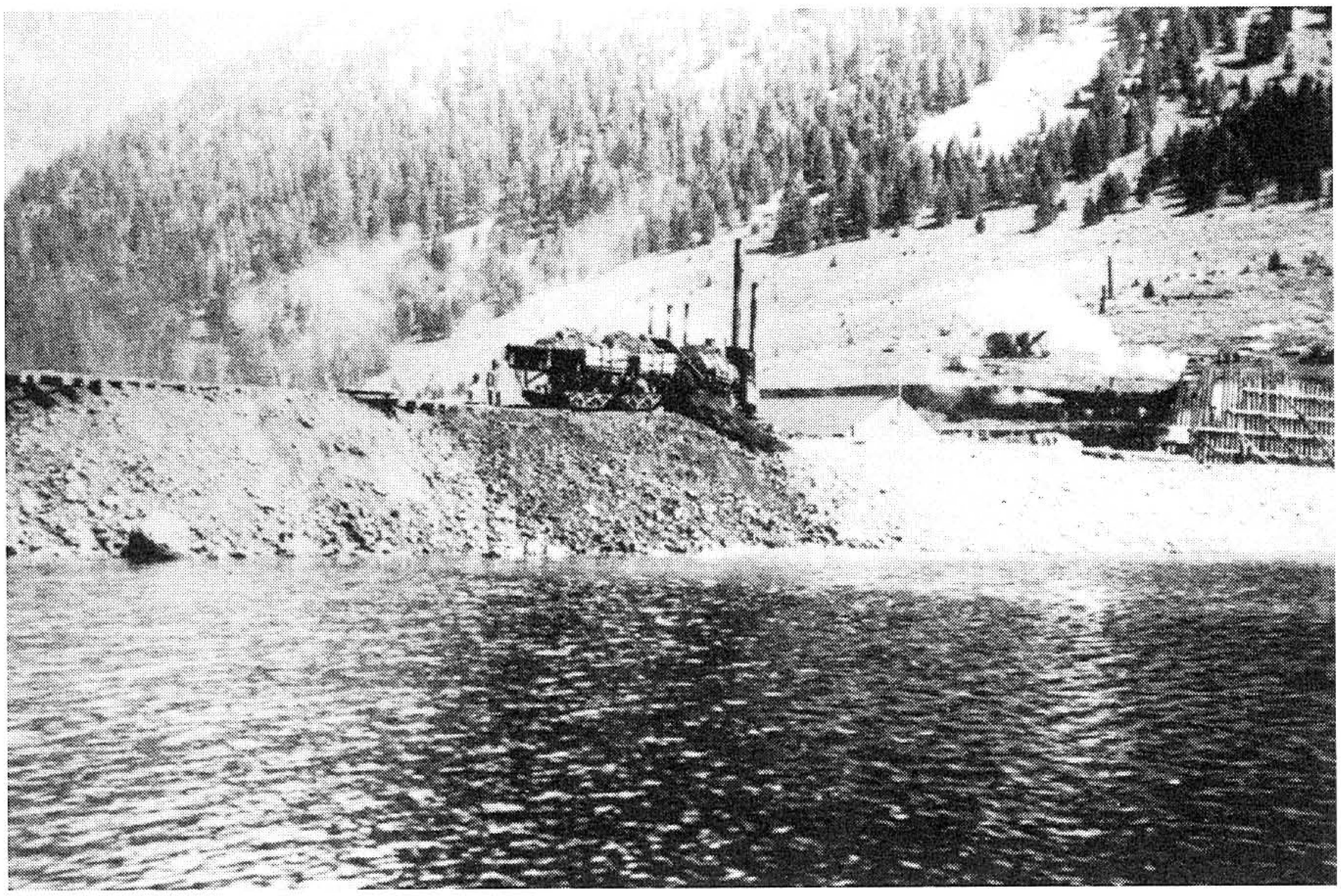


Figure 19. Photograph taken in 1915 during construction of Hebgen Dam in Montana. Note how rock fill is being loosed from side-dumping rail cars into the reservoir pool. Formwork for the concrete core wall can be seen at the far right. This embankment subsequently settled up to 5.3 ft against the core wall during the magnitude 7.5 West Montana earthquake in August 1959 (Sherrard, 1959; James and Kiersch, 1991).

Test data conclusively suggest that the higher the percent saturation, the lower the likelihood of future hydrocompression problems.

• In fills more than 15–20 ft deep, the building pad area should be overexcavated so as to create a semilevel fill prism beneath the vertical projection of the proposed structure's limits (fig. 20). Performance data suggest that fill differentials of less than 15 percent perform within tolerable limits of differential settlement.

• After a fill is initially compacted at 50–85 percent saturation, its strength can only be expected to decrease. Long-term assessments of slope stability and surficial creep should be based upon drained shear testing of soils at their field capacity, something generally between 85 and 95 percent saturation. Soils tested at these higher water contents will yield lower strengths than those measured at compaction water contents.

• Structures sensitive to long-term movements that are founded adjacent to fill slopes more than 20 ft high should be built upon drilled pier or caisson foundation elements designed to resist surficial slope creep engendered by hydroswell and creep. The steeper the slope, the greater the

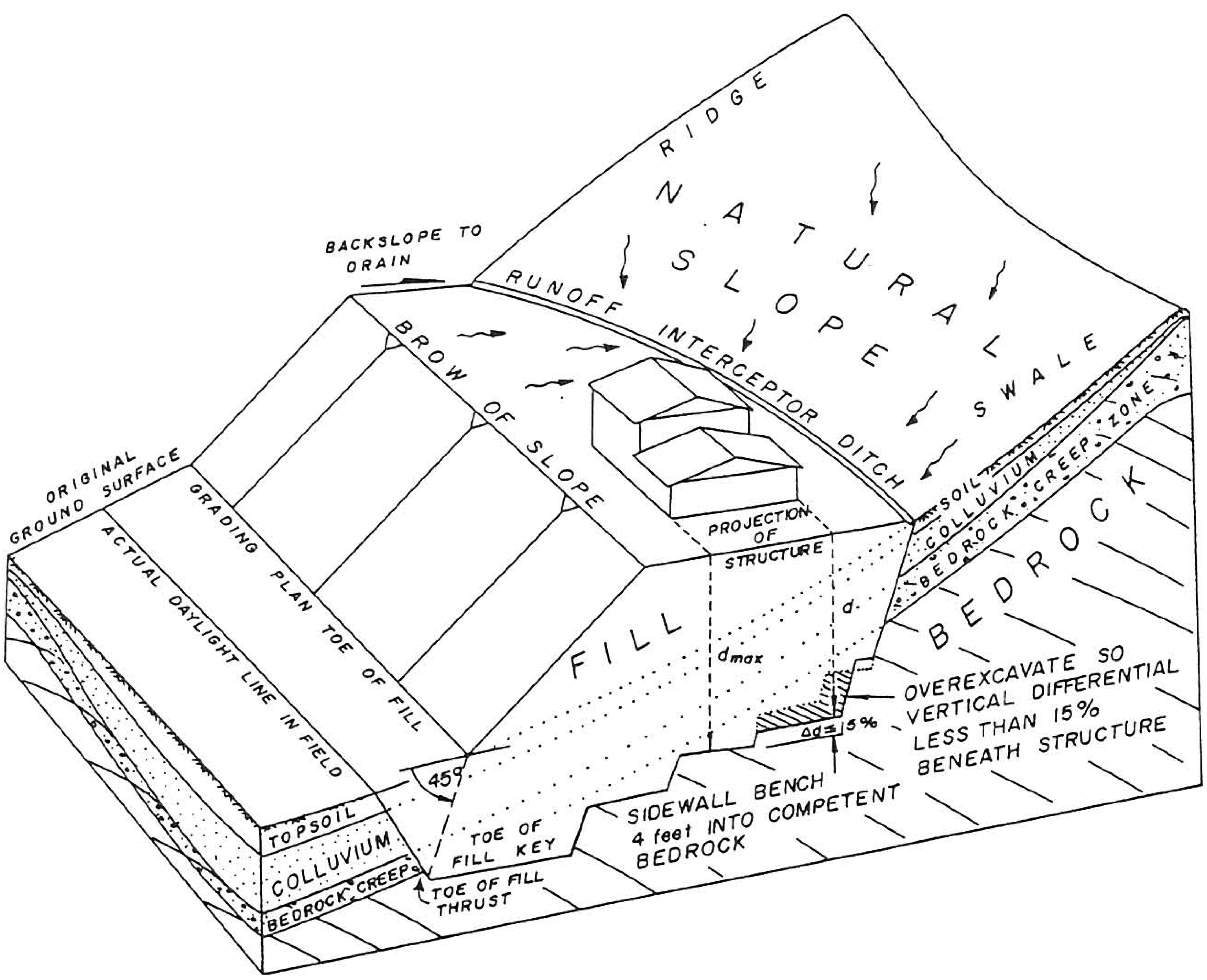



Figure 20. Overexcavation standards for embankments recommended by Rogers (1992a), based on an assessment of more than 500 cases of differential settlement in California. This model is based on a model developed by Scullin (1983) in Orange County, California, in the late 1960s.

imposed lateral loads that can be expected within the near-surface zone. Dropping slope gradients to 3:1 inclinations would help, but not completely forestall such movement. Less sensitive elements, such as free-standing fences, posts, and pilasters, could be restrained with some manner of tensile tie anchors.

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