2

Ground Subsidence, Collapse, and Heave

2.1 Introduction

2.1.1 General

Origins

The hazardous vertical ground movements of subsidence, collapse, and heave, for the most part, are the results of human activities that change an environmental condition. Natural occurrences, such as earthquakes and tectonic movements, also affect the surface from time to time.

Significance

Subsidence, collapse, and heave are less disastrous than either slope failures or earthquakes in terms of lives lost, but the total property damage that results each year probably exceeds that of the other hazards. A positive prediction of their occurrence is usually very difficult, and uncertainties always exist, although the conditions favorable to their development are readily recognizable.

2.1.2 The Hazards

A summary of hazardous vertical ground movements, their causes, and important effects is given in Table 2.1.

2.1.3 Scope and Objectives

Scope

Ground movements considered in this chapter are caused by some internal change within the subsurface such as the extraction of fluids or solids, solution of rock or a cementing agent in soils, erosion, or physicochemical changes. Movements brought about by the application of surface loads from construction activity (that is, ground settlements resulting from embankments, buildings, etc.) will not be considered here.

Objectives

The objectives are to provide the basis for recognizing the potential for surface movements, and for preventing or controlling the effects.

TABLE 2.1
Summary of Hazardous Vertical Ground Movements

Movement	Description	Causes	Important Effects
Regional subsidence	Downward movement of ground surface over large area	Seismic activity,[a] groundwater extraction, oil and gas extraction	Flooding, growth faults, structure distortion
Ground collapse	Sudden downward movement of ground surface over limited area	Subsurface mining, limestone cavity growth, piping cavities in soils, leaching of cementing agents	Structure destruction, structure distortion
Soil subsidence	Downward movement of ground surface over limited area	Construction dewatering, compression under load applied externally, desiccation and shrinkage	Structure distortion
Ground heave	Upward movement of ground surface	Expansion of clays and rocks, release of residual stresses, tectonic activity,[b] ground freezing	Structure distortion, weakening of clay shale slopes

[a] See Section 3.3.3.
[b] See Section 3.3.3, Appendix A.

2.2 Groundwater and Oil Extraction

2.2.1 Subsurface Effects

Groundwater Withdrawal

Aquifer Compaction

Lowering the groundwater level reduces the buoyant effect of water, thereby increasing the effective weight of the soil within the depth through which the groundwater has been lowered. For example, for a fully saturated soil, the buoyant force of water is 62.4 pcf (1 t/m^3) and if the water table is lowered 100 ft (30 m), the increase in effective stress on the underlying soils will be 3.0 tsf (30 t/m^3), a significant amount. If the prestress in the soils is exceeded, compression occurs and the surface subsides. In an evaluation of the effect on layered strata of sands and clays, the change in piezometric level in each compressible stratum is assessed to permit a determination of the change in effective stress in the stratum. Compression in sands is essentially immediate; cohesive soils exhibit a time delay as they drain slowly during consolidation. Settlements are computed for the change in effective stress in each clay stratum from laboratory consolidation test data.

The amount of subsidence, therefore, is a function of the decrease in the piezometric level, which determines the increase in overburden pressures and the compressibility of the strata. For clay soils the subsidence is a function of time.

Construction Dewatering

Lowering the groundwater for construction projects has the same effect as “aquifer compaction,” that is, it compresses soil strata because of an increase in effective overburden stress.

Oil Extraction

Oil extraction differs from groundwater extraction mainly because much greater depths are involved, and therefore much greater pressures. Oil (or gas) extraction results in a reduction of pore-fluid pressures, which permits a transfer of overburden pressures to the intergranular skeleton of the strata.

In the Wilmington oil field, Long Beach, California, Allen (1973) cites "compaction as taking place primarily by sand grain arrangement, plastic flow of soft materials such as micas and clays, and the breaking and sharding of grains at stressed points." Overall, about two thirds of the total compaction at the Wilmington field is attributed to the reservoir sands and about one third to the interbedded shales (Allen and Mayuga, 1969). During a period of maximum subsidence in 1951 to 1952, faulting apparently occurred at depths of 1500 to 1750 ft (450 to 520 m), shearing or damaging hundreds of oil wells.

2.2.2 Surface Effects

Regional Subsidence

General

Surface subsidence from fluid extraction is a common phenomenon and probably occurs to some degree in any location where large quantities of water, oil, or gas are removed. Short-term detection is difficult because surface movements are usually small, are distributed over large areas in the shape of a dish, and increase gradually over a span of many years.

Monitoring Surface Deflections

Traditionally, subsidence has been measured periodically using normal surveying methods. When large areas are involved the procedure is time-consuming, costly, and often incomplete. Since 1992, some major cities have been monitoring subsidence with InSAR (*Interferometric Synthetic Aperture Radar*) (Section 2.2.3). SAR images are presently obtained by the European Space Agency (ESA) satellites. Cities mapped include Houston, Phoenix, and Las Vegas in the United States, and Mexico City. The new InSAR maps provide new and significant aspects of the spatial pattern of subsidence not evident on conventional mapping (Bell et al., 2002),

Some Geographic Locations

Savannah, Georgia: The city experienced as much as 4 in. of subsidence over the 29 year period between 1933 and 1962 because of water being pumped from the Ocala limestone, apparently without detrimental effects (Davis et al., 1962).
Houston, Texas: A decline in the water table of almost 300 ft since 1890 has caused as much as 5 ft of subsidence with serious surface effects (see Section 2.2.4).
Las Vegas, Nevada, Tucson and Elroy, Arizona and the San Joaquin and Santa Clara Valleys, California: They have insignificant amounts of subsidence from groundwater withdrawal.
Mexico City: In the hundred years or so, between the mid-1800s and 1955, the city experienced as much as 6 m (20 ft) of subsidence from compression of the underlying soft soils because of groundwater extraction. By 1949, the rate was 35 cm/yr (14 in.). Surface effects have been serious (see Section 2.2.4).
London, England: A drop in the water table by as much as 200 ft has resulted in a little more than 1in. of subsidence, apparently without any detrimental effects because of the stiffness of the clays.
Long Beach, California: It has suffered as much as 30 ft of subsidence from oil extraction between 1928 and 1970 with serious effects (see Section 2.2.4).

Lake Maricaibo, Venezuela: The area underwent as much as 11 ft of subsidence between 1926 and 1954 due to oil extraction.
Po Valley, Italy: It has been affected by gas withdrawal. In the Adriatic Sea, close to the coast near Ravenna, gas withdrawal from 1971 to 1992 has resulted in 31 cm of land subsidence. Numerical predictions suggest that a residual subsidence of 10 cm may occur by 2042, 50 years after the field was abandoned (Bau et al., 1999).

Flooding, Faulting, and Other Effects

Flooding results from grade lowering and has been a serious problem in coastal cities such as Houston and Long Beach in the United States, and Venice in Italy. In Venice, although subsidence from groundwater withdrawal has been reported to be only 5.5 cm/year, the total amount combined with abnormally high tides has been enough to cause the city to be inundated periodically. The art works and architecture of the city have been damaged as a result. Flood incidence also increases in interior basins where stream gradients are affected by subsidence.

Faulting or *growth faults* occur around the periphery of subsided areas. Although displacements are relatively small, they can be sufficient to cause distress in structures and underground storm drains and sanitary sewers, and sudden drops in roadways. Oil extraction can cause movement along existing major faults.

Differential movement over large distances affects canal flows, such as in the San Joaquin Valley of California and over short distances causes distortion of structures, as in Mexico City.

Grade lowering can also result in the loss of head room under bridges in coastal cities and affect boat traffic, as in Houston (ENR, 1977).

Local Subsidence from Construction Dewatering

Drawdown of the water table during construction can cause surface subsidence for some distance from the dewatering system. Differential settlements reflect the cone of depression. The differential settlements can be quite large, especially when highly compressible peat or other organic soils are present, and the effect on adjacent structures can be damaging.

During the construction of the Rotterdam Tunnel, wellpoints were installed to relieve uplift pressures in a sand stratum that underlay soft clay and peat. The groundwater level in observation wells, penetrating into the sand, at times showed a drop in water level of 42 ft. Settlements were greatest next to the line of wellpoints: 20 in. at a distance of 3 ft and 3 in. at a distance of 32 ft. The water level was lowered for about 2.9 yr and caused an effective stress increase as high as 1.3 tsf (Tschebotarioff, 1973).

2.2.3 Physiographic Occurrence

General

Although subsidence can occur in any location where large quantities of fluids are extracted, its effects are felt most severely in coastal areas and inland basins. When groundwater depletion substantially exceeds recharge, the water table drops and subsidence occurs.

Coastal Areas

Many examples of coastal cities subsiding and suffering flooding can be found in the literature, and any withdrawal from beneath coastal cities with low elevations in reference to sea level must be performed with caution.

Interior Basins

In the semiarid to arid regions of the western United States, the basins are often filled with hundred of meters of sediments, which serve as natural underground reservoirs for the periodic rainfall and runoff from surrounding mountains. Subsidence can reach significant amounts: for example, as much as 30 ft in the San Joaquin Valley of California, 12 ft in the Santa Clara Valley in California, 15 ft in Elroy, Arizona, 6 ft in Las Vegas, Nevada, and 9 ft in Houston (Leake, 2004). Around the Tucson Basin, where the water level has dropped as much as 130 ft since 1947, it has been suggested that minor faulting is occurring and may be the reason for distress in some home foundations (Davidson, 1970, Pierce, 1972). Other effects will be increased flooding due to changes in stream gradients and the loss of canal capacity due to general basin lowering.

In 1989, the U.S. Department of Housing and Urban Development began requiring special subsidence hazard assessments for property located near subsidence features in Las Vegas (Bell et al., 1992). The requirement resulted from structural damage to a major subdivision in North Las Vegas that required the repair or displacement of more than 240 damaged or threatened homes at a cost of $12 to $13 million. The assessments included guidelines specifying detailed studies and specialized construction for all new developments within 150 m of a mapped fault.

The basin in which Mexico City is situated is filled with thick lacustrine sediments of volcanic origin, and groundwater withdrawal has resulted in serious consequences.

2.2.4 Significant Examples

Houston, Texas (Water Extraction: Flooding and Faulting)

Between 1906 and 1964, 5 ft of subsidence occurred, and reports place the subsidence at 9 ft at some locations (Civil Engineering, 1977). The cost of the subsidence, including flood damage, between 1954 and 1977 has been estimated to be $110 million and in 1977 was growing at the annual rate of $30 million (Spencer, 1977). The subsidence results in "growth" faults that cause distress in structures, large deflections of roadways, and rupture of utility lines; in flooding, resulting in homes being abandoned along Galveston Bay; and the lowering of bridges over the Houston Ship Canal.

The problem of growth faults in the Houston metropolitan area is severe. Activity has been recognized on more than 40 normal faults, which are prehistoric according to Van Siclen (1967). Major surface faults and the cumulative subsidence between 1906 and 1964 are given in Figure 2.1, and a profile of subsidence and groundwater decline for a distance of about 14 mi is given in Figure 2.2. The drop in water level of almost 300 ft causes an increase in overburden pressure of about 9 tsf, which is believed to be causing downward movement along the old faults as clay beds interbedded with sand aquifers consolidate (Castle and Youd, 1972).

Holdahl et al. (1991) report that the area has been divided into two zones, west and east of downtown Houston. In 1987, west zone subsidence was ranging up to 72 mm/year (2.8 in./year), but after 1978 the east zone has experienced 60 to 90% decreases in subsidence rates due to regulated groundwater withdrawal and the use of canals to carry water from Lake Houston. The east zone is the industrial zone. Subsidence patterns in Houston are being monitored with InSAR (Section 2.2.2).

Mexico City (Water Extraction: Subsidence and Foundation Problems)

Geologic Conditions

The basin of the valley of Mexico City, 2240 m above sea level, has been filled with 60 to 80 m of Pleistocene soils including interbedded sands, sands and gravels, and

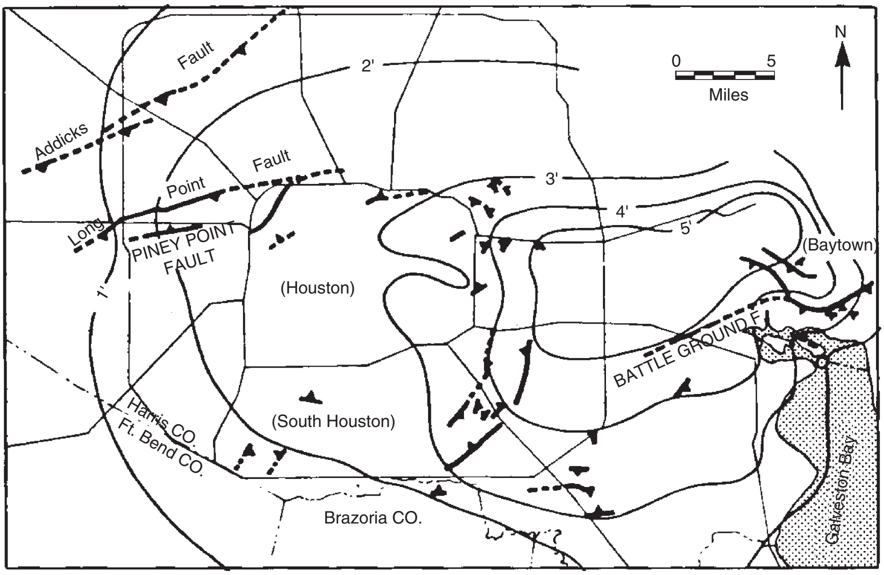


FIGURE 2.1
Surface faulting and cumulative subsidence (in ft) in the Houston area between 1906 and 1964. (From Castle, R. O. and Youd, T. L., *Bull. Assoc. Eng. Geol.*, 9, 1972. With permission.)

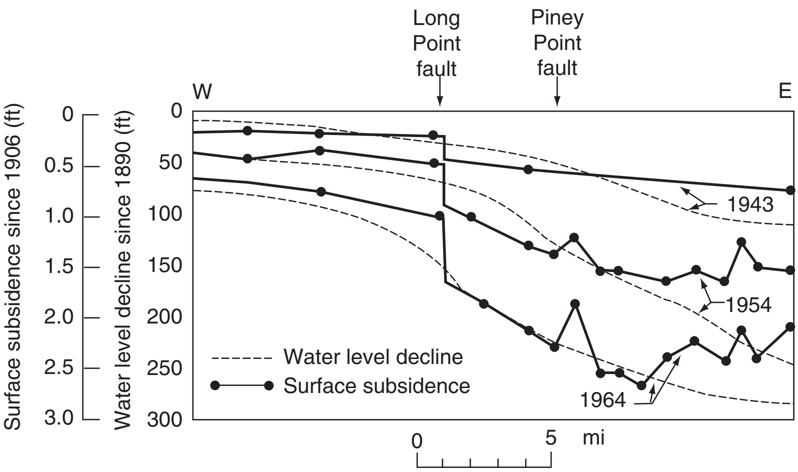


FIGURE 2.2
Profiles of subsidence and groundwater decline along a section trending due west from Houston. (From Castle, R. O. and Youd, T. L., *Bull. Assoc. Eng. Geol.*, 9, 1972. With permission.)

lacustrine volcanic clay, which overlie a thick deposit of compact sand and gravel. The soils to depths of about 33 m are very soft to medium-stiff clays. Void ratios as high as 13 and water contents as high as 400% indicate the very high compressibility of the soils. The clays are interbedded with thin sand layers, and thick sand strata are found at depths of 33, 45, and 73 m. Prior to the 1930s, groundwater beneath the city was recharged naturally.

Ground Subsidence

In the 100 years prior to 1955, a large number of water wells were installed between depths of 50 and 500 m in the sand and sand and gravel layers. Subsequent to the 1930s, groundwater withdrawal began to exceed natural recharge. The wells caused a large reduction in piezometric head, especially below 28 m, but the surface water table remained unaltered because of the impervious shallow clay formation. A downward hydraulic gradient was induced because of the difference in piezometric levels between the ground surface and the water-bearing layers at greater depths. The flow of the descending water across the highly compressible silty clay deposits increased the effective stresses, produced consolidation of the weak clays, and thus caused surface subsidence (Zeevaert, 1972).

In some places, as much as 7.6 m of subsidence had occurred in 90 years, with about 5 m occurring between 1940 and 1970. The maximum rate, with respect to a reference sand stratum at a depth of 48 m, was 35 cm/year, and was reached in 1949. About 80 to 85% of the subsidence is attributed to the soils above the 50 m depth. In 1955, the mayor of the city passed a decree prohibiting all pumping from beneath the city, and the rate of lowering of the piezometric levels and the corresponding rate of subsidence decreased considerably. As of 2001, however, subsidence was still continuing at rates of about 3 to 7 cm/year (Rudolph, 2001), and groundwater was being pumped. Even after pumping ceases, the compressible soils will continue consolidating under the increased effective stresses. Attempts are being made to bring water in from other watersheds. Subsidence patterns in Mexico City are being monitored with InSAR (Section 2.2.2).

Foundation Problems

Before the well shutdown program, many of the wells that were poorly sealed through the upper thin sand strata drew water from these strata, causing dish-shaped depressions to form around them. The result was the development of severe differential settlements, causing tilting of adjacent buildings and breakage of underground utilities. Flooding has become a problem as the city now lies 2 m below nearby Lake Texcoco (Rudolph, 2001).

A different problem occurs around the perimeter of the basin. As the water table drops, the weak bentonitic clays shrink by desiccation, resulting in surface cracks opening to as much as a meter in width and 15 m in depth. When the cracks open beneath structures, serious damage results.

The major problems, however, have been encountered within the city. Buildings supported on piles can be particularly troublesome. End-bearing piles are usually driven into the sand stratum at 33 m for support. As the ground surface tends to settle away from the building because of the subsidence, the load of the overburden soils is transferred to the piles through "negative skin friction" or "downdrag." If the piles do not have sufficient capacity to support both the building load and the downdrag load, settlements of the structure result. On the other hand, if the pile capacities are adequate, the structure will not settle but the subsiding adjacent ground will settle away from the building. When this is anticipated, utilities are installed with flexible connections and allowances are made in the first-floor design to permit the sidewalks and roadways to move downward with respect to the building.

Modern design attempts to provide foundations that enable a structure to settle at about the same rate as the ground subsides (Zeevaert, 1972). The "friction-pile compensated foundation" is designed such that downdrag and consolidation will cause the building to settle at the same rate as the ground subsidence. The Tower Latino Americano, 43 stories high, is supported on a combination of end-bearing piles and a compensated raft foundation (Zeevaert, 1957). The piles were driven into the sand stratum at 33 m, and a 13-m-deep excavation was made for the raft which had the effect of removing a substantial overburden load, subsequently replaced by the building load. The building was completed in 1951 and the settlements as of 1957 occurring from consolidation of the clay strata were as predicted, or about 10 cm/year.

Long Beach, California (Oil Extraction: Subsidence and Flooding)

Geologic Conditions

The area is underlain by 2000 ft of "unconsolidated" sediments of late Pliocene, Pleistocene, and Holocene age, beneath which are 4600 ft of oil-producing Pliocene and Miocene formations including sandstone, siltstone, and shale.

Ground subsidence began to attract attention in Long Beach during 1938 to 1939 when the extraction of oil began from the Wilmington field located primarily in the city (see site location map, Figure 2.3). A peak subsidence rate of over 20 in/year was reached in 1951 to 1952 (Section 2.2.1), By 1973, subsidence in the center of a large bowl-shaped area had reached 30 ft vertically, with horizontal movements as great as 13 ft. Flood protection for the city, which is now below sea level in many areas, is provided by extensive diking and concrete retaining walls. Deep-well recharging by salt-water injection has halted subsidence and some rebound of the land surface has occurred.

Baldwin Hills Reservoir Failure (Oil Extraction: Faulting)

Event

On December 14, 1963, the Baldwin Hills Reservoir, a pumped storage reservoir located in Los Angeles (Figure 2.3), failed and released a disastrous flood onto communities downstream (Jansen et al., 1967).

Background

The dam was located close to the Inglewood oil field and the Inglewood fault passed within 500 ft of the west rim of the reservoir. The Inglewood fault is part of the major Newport–Inglewood fault system (Figure 2.3). During construction excavation in 1948, two minor faults were found to pass through the reservoir area, but a board of consultants judged that further movement along the faults was unlikely. The dam was well instrumented with two strong-motion seismographs, tiltmeters, settlement measurement devices, and observation wells.

Surface Movements

Between 1925 and 1962, at a point about 0.6 mi west of the dam, about 10 ft of subsidence and about 6 in. of horizontal movement occurred. In 1957, cracks began to appear in the area around the dam. Six years later, failure occurred suddenly and the narrow breach in the dam was found to be directly over a small fault (Figure 2.4). It was judged that 6 in. of movement had occurred along the fault, which ruptured the lining of the reservoir and permitted the sudden release of water. It appears likely that subsidence from oil extraction caused the fault displacement, since there had been no significant seismic activity in the area for at least the prior month.

2.2.5 Subsidence Prevention and Control

Groundwater Extraction

General

Subsidence from groundwater extraction cannot be avoided if withdrawal exceeds recharge, resulting in significant lowering of the water table or a reduction in piezometric levels at depth, and if the subsurface strata are compressible.

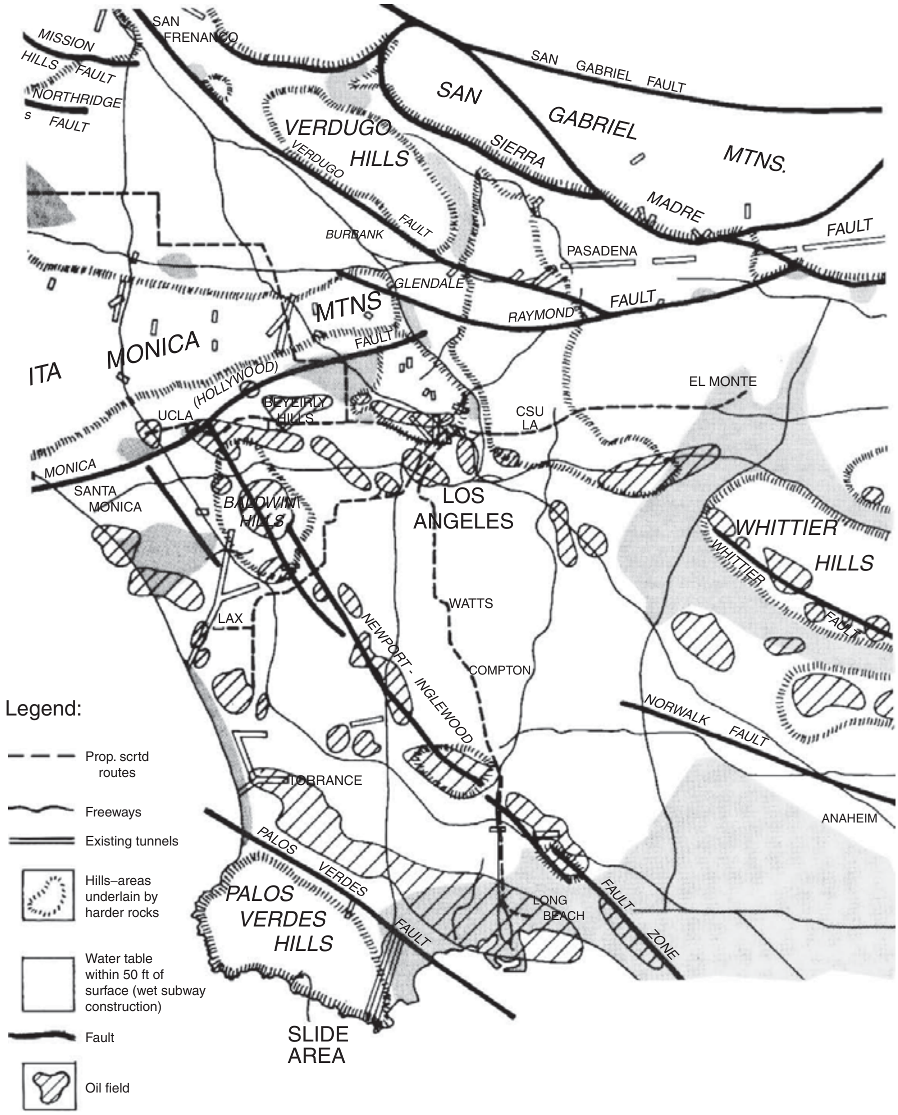


FIGURE 2.3
Water table, faults, and oil fields in the Los Angeles Basin. (From Proctor, R. J., *Geology, Seismicity and Environmental Impact,* Special Publication Association Engineering Geology, University Publishers, Los Angeles, 1973, pp. 187–193. With permission.)

Prediction

Prior to the development of a groundwater resource, studies should be made to determine the water-balance relationship and estimate magnitudes of subsidence. The water-balance relationship is the rate of natural recharge compared with the anticipated maximum rate of withdrawal. If recharge equals withdrawal, the water table will not drop and subsidence will not occur. If withdrawal significantly exceeds recharge, the water table will be lowered.

Estimates of the subsidence to be anticipated for various water-level drops are made to determine the maximum overdraft possible before surface settlement begins to be troublesome and cause flooding and faulting. By using concepts of soil mechanics it is possible to

FIGURE 2.4
Pumped storage reservoir failure 1968, Baldwin Hills, California. Failure occurred suddenly over a small fault.

compute estimates of the surface settlements for various well-field layouts, including differential deflections and both immediate and long-term time rates of settlements.

The increase in the flood hazard is a function of runoff and drainage into a basin or the proximity to large water bodies and their relative elevations, including tidal effects. The growth of faults is difficult to assess both in location and magnitude of displacement. In general, locations will be controlled by the locations of relict faults. New faults associated with subsidence are normally concentrated in a concentric pattern around the periphery of the subsiding area. The center of the area can be expected to occur over the location of wells where withdrawal is heaviest.

Prevention

Only control of overextraction prevents subsidence. Approximate predictions as to when the water table will drop to the danger level can be based on withdrawal, precipitation, and recharge data. By this time, the municipality must have provisions for an alternate water supply to avoid the consequences of overdraft.

Control

Where subsidence from withdrawal is already troublesome, the obvious solution is to stop withdrawal. In the case of Mexico City, however, underlying soft clays continued to consolidate for many years, even after withdrawal ceased, although at a much reduced rate. Artificial recharging will aid the water balance ratio.

Recharging by pumping into an aquifer requires temporary surface storage. The Santa Clara Valley Water District (San Jose, California) has been storing storm water for recharge pumping for many years (ENR, 1980). In west Texas and New Mexico, dams are to be built to impound flood waters that are to be used as pumped-groundwater recharge (ENR, 1980).

Where the locale lacks terrain suitable for water storage, deep-well recharging is not a viable scheme, and recharge is permitted to occur naturally. Venice considered a number of recharge schemes, in addition to a program to cap the city wells begun in 1965. Apparently, natural recharge is occurring and measurements indicate that the city appears to be rising at the rate of about 1 mm every 5 years (*Civil Engineering*, 1975).

Oil and Gas Extraction

Prediction of subsidence from oil and gas extraction is difficult with respect to both magnitude and time. Therefore, it is prudent to monitor surface movements and to have contingency plans for the time when subsidence approaches troublesome amounts.

Control by deep-well recharging appears to be the most practical solution for oil and gas fields. At Long Beach, water injection into the oil reservoirs was begun in 1956; subsidence has essentially halted and about 8 mi^2 of land area has rebounded, in some areas by as much as 1 ft (Allen, 1973; Testa, 1991).

Construction Dewatering

Prediction and Control

Before the installation of a construction dewatering system in an area where adjacent structures may be affected, a study should be made of the anticipated drop in water level as a function of distance, and settlements to be anticipated should be computed considering building foundations and soil conditions. Peat and other organic soils are particularly susceptible to compression. In many cases, condition surveys are made of structures and all signs of existing distress recorded as a precaution against future damage claims. Before dewatering, a monitoring system is installed to permit observations of water level and building movements during construction operations. The predicted settlements may indicate that preventive measures are required.

Prevention

Prevention of subsidence and the subsequent settlement of a structure are best achieved by placing an impervious barrier between the dewatering system and the structure, such as a slurry wall. Groundwater recharge to maintain water levels in the area of settlement-sensitive structures is considered to be less reliable.

Surcharging

Surcharging of weak compressible layers is a positive application of construction dewatering. If a clay stratum, for example, lies beneath a thickness of sands adequate to apply significant load when dewatered, substantial prestress can be achieved if the water table remains lowered for a long enough time. Placing a preload on the surface adds to the system's effectiveness.

2.3 Subsurface Mining

2.3.1 Subsidence Occurrence

General

Extraction of materials such as coal, salt, sulfur, and gypsum from "soft" rocks often results in ground subsidence during the mining operation or, at times, many years after operations have ceased. Subsidence can also occur during hard rock mining and tunneling operations.

In the United States, ground subsidence from mining operations has occurred in about 30 states, with the major areas located in Pennsylvania, Kansas, Missouri, Oklahoma, Montana, New Jersey, and Washington (Civil Engineering, 1978). Especially troublesome in terms of damage to surface structures are the Scranton-Wilkes-Barre and Pittsburgh areas of Pennsylvania. The approximate extent of coal fields in the eastern United States in the Pennsylvanian Formations is given in Figure 2.5.

Other troublesome areas: The midlands of England where coal has been, and is still being, mined. Paris, France, and surrounding towns have suffered surface collapse, which at times has swallowed houses, over the old underground limestone and gypsum quarries that were the source of building stone for the city in the 18th century (Arnould, 1970). In the Paris area, collapse has been intensified by groundwater pumping (see Section 2.4.2).

Metal Mining

During the 1700s and 1800s various ores, including iron (magnetite), copper, zinc, and lead, were extracted from relatively shallow underground mines in New Jersey and Pennsylvania. In recent years, the deterioration of these workings has resulted in the formation of surface sinkholes. Over 400 abandoned iron mines in Northern New Jersey have been identified (Cohen et al., 1996). A collapse resulting in a 30-m-diameter sinkhole jeopardizing a major road was reported. The Schuyler Copper Mine in North Arlington, New Jersey, is estimated to contain up to 55 shafts located in a 20-acre area within the town (Trevits and Cohen, 1996). In 1989, a collapse occurred and a large hole opened in the backyard of a home, and depressions formed at other shaft sites.

In Chester County, Pennsylvania, numerous shallow zinc and lead mines were developed in the 1800s. In general, the metals were deposited in thin, near-vertical formations. Mining proceeded along drifts that, in many cases, were extended upward toward the surface leaving shallow caps of overburden. The area periodically suffers the development of small sinkholes, particularly after heavy rains.

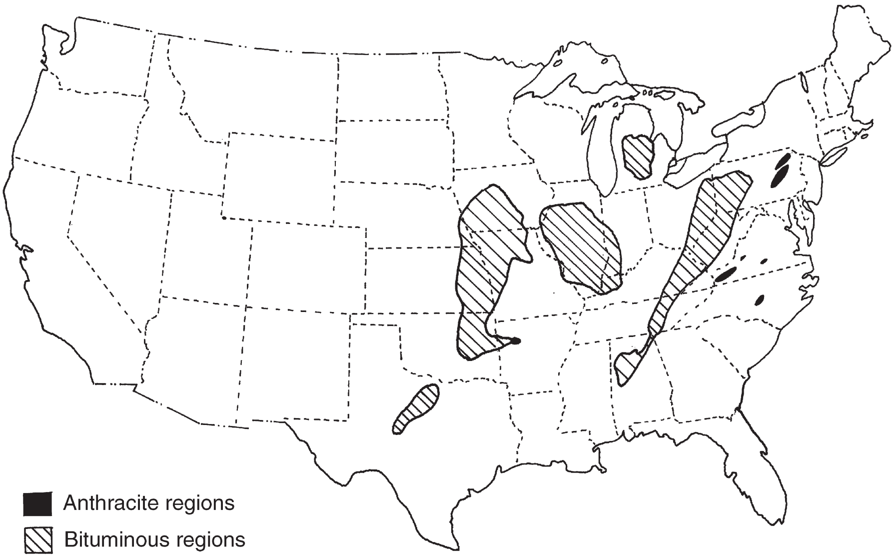


FIGURE 2.5
Approximate extent of coal fields of the Pennsylvanian formations in the eastern United States. (From Averitt, P., *USGS Bull, 1275,* U.S. Govt., Printing Office, Washington, DC, 1967.)

Salt Mines

In the United States, salt is mined from the subsurface in New York, Kansas, Michigan, Utah, Texas, Louisiana, and California.

Kansas

In Kansas, natural sinkholes, such as Lake Inman in McPherson County, are common on the solution front along the eastern edge of the Hutchinson Salt Member. Solution mining has resulted in subsidence in and around the city of Hutchinson. In 1974, a sinkhole 300 ft in diameter occurred at the Cargill Plant, southeast of the city.

In Russell County, I-70 crosses two active sinkholes that have been causing the highway to subside since its construction in the mid-1960s. In the area, sandstone and shale overlie a 270-ft-thick salt bed 1300 ft below the surface. It is believed that old, improperly plugged, abandoned oil wells allowed fresh water to pass through the salt bed resulting in dissolution of the salt and the creation of a large void. Subsidence has been of the order of 4 to 5 in./year, and in recent years a nearby bridge has subsided over another sinkhole. Over the years, the roadway grade has been raised and repaved. In 1978, a sinkhole 75 ft across and 100 ft deep, centered about an abandoned oil well, opened up about 20 mi from the I-70 sinks. Plugging old wells and performing cement injections have failed to stop the subsidence (Croxton, 2002).

New York

On March 12, 1994, a 650×650 ft panel 1180 ft under the Genesse Valley in Retsof, New York, failed catastrophically generating a magnitude 3.6 seismic event. A similar size panel failed a month later, and groundwater immediately entered the mine. Two sinkholes, 700 ft across and up to 75 ft deep, opened over the failed mine panels (William, 1995). Surface subsidence over a large area in the valley due to (1) dissolution of pillars and (2) dewatering of valley-fill sediments was occurring at rates measured in inches to feet per year in 1995. The salt bed, about 30 ft thick, was mined for many years by the room and pillar method where 40 to 50% of the salt was left in place. The collapse occurred in an area of the mine where smaller pillars, known as yield pillars, were used to maintain the mine roof (MSHA, 1995). Gowan and Trader (2000) suggest that an anomalous buildup of natural gas and brine pressure above the collapse area contributed to the failure.

Coal Mining

From the aspects of frequency of occurrence and the effects on surface structures, coal mining appears to be the most important subsurface mining operation. Mine collapse results in irregular vertical displacement, tilting, and horizontal strains at the surface, all resulting in the distortion of structures as illustrated in Figure 2.6. The incidence and severity of subsidence are a function of the coal bed depth, its thickness, percent of material extracted, tensile strength of the overburden, and the strength of pillars and other roof supports. InSAR is being used in Poland and other locations in Europe to monitor subsidence from coal mining.

General methods of extraction and typical subsidence characteristics include:

- The longwall panel method is used in Europe, and is finding increasing use in the United States. It involves complete removal of the coal. Where the mine is relatively shallow and the overlying materials weak, collapse of the mine, and surface subsidence progress with the mining operation. Subsidence is in the form of a large trough which mirrors, but is larger than, the plan dimensions of the mine.
- The room and pillar method has been used in the United States. Pillars are left in place to support the roof, but subsequent operations rob the pillars, weakening support. Collapse is often long term. Subsidence takes the forms of sinkholes (pits) or troughs (sags).

Most states have prepared detailed "Codes" relating to underground mining; for example, in Pennsylvania, Code 89.142a adopted in 1998 refers to "Subsidence Control: Performance Standards."

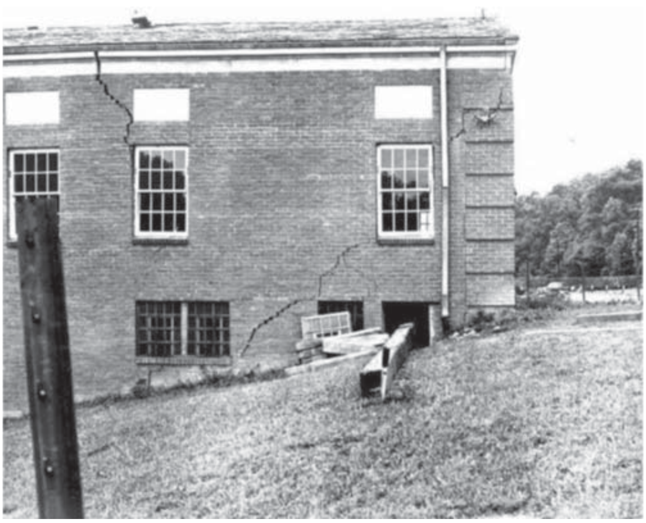


FIGURE 2.6
Building damaged by mine subsidence in Pittsburgh. (Photo courtesy of Richard E. Gray.)

2.3.2 Longwall Panel Extraction

Extraction Procedures

Most of the coal currently produced in Pennsylvania uses the longwall mining method (PaDEP, 2002). Mining begins with the excavation of hallways supported by pillars along the sides of the panel to be mined, generally 600 to 7000 ft or more in width (Figure 2.7). The hallways provide for access, ventilation, and the removal of the mined coal. Along the face of the panel a cutting head moves back and forth, excavating the coal, which falls onto a conveyor belt. A system of hydraulic roof supports prevents the mine from collapsing. As the excavation proceeds, the supports move with it and the mine roof behind the supports is permitted to collapse.

National Coal Board (NCB) Studies

Based on surveys of 157 collieries, the NCB of Great Britain (NCB, 1963, 1966) developed a number of empirical relationships for the prediction of the vertical component of surface displacements s and the horizontal component of surface strain e, associated with the trough-shaped excavation of the longwall panel method of coal extraction, illustrated in Figure 2.8.

Surveys were made over coal seams that were inclined up to 25° from the horizontal, were about 3 to 15 ft in thickness m, and ranged in depth h from 100 to 2600 ft. Face or panel width w varied from 100 to 1500 ft, and the panel width to depth ratio w/h varied from 0.05 to 4.0. Panel widths were averaged if the panel sides were nonparallel. The foregoing conditions assume that there is no zone of special support within the panel areas. The physical relationships are illustrated in Figure 2.8.

Subsidence Characteristics

Angle of the Draw

The area where the coal is mined is referred to as the "panel." Extraction results in surface subsidence termed "subsidence trough" or "basin." The width by which the subsidence trough exceeds the panel width is the "angle of the draw."

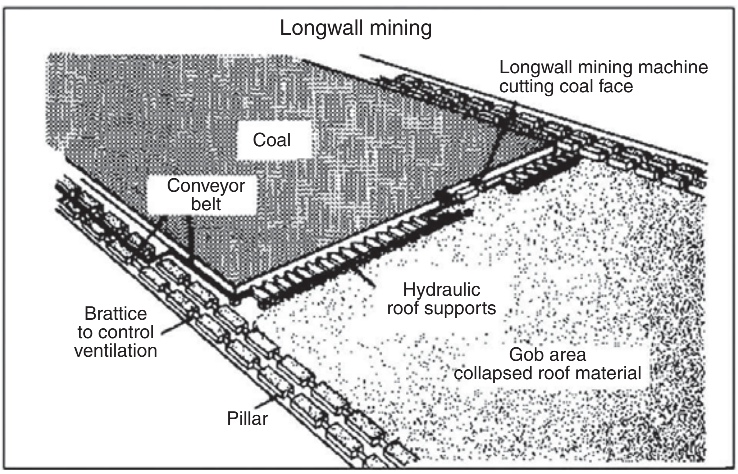


FIGURE 2.7
Longwall panel mining of coal beds. (From PaDEP, Commonwealth of Pennsylvania, Department of Environmental Protection, Bureau of Mining and Reclamation, Contract No. BMR-00-02, 2002. With permission.)

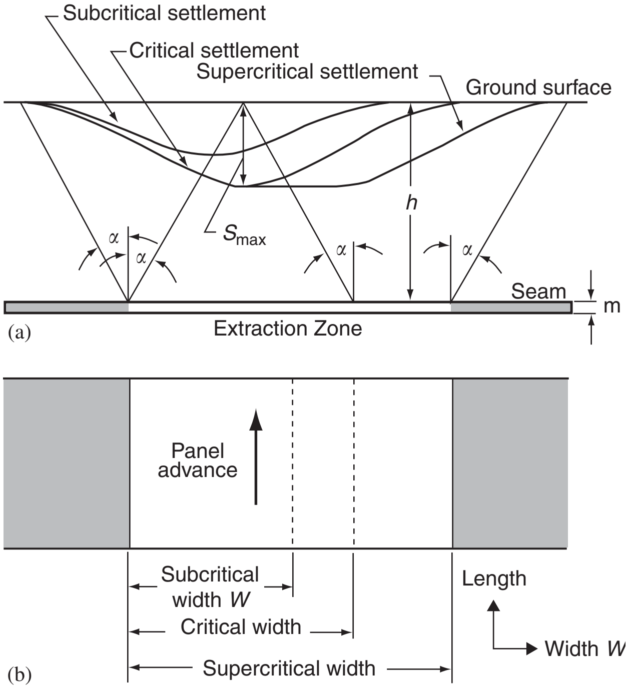


FIGURE 2.8
Mine subsidence vs. critical depth concept for longwall panel extraction developed by the National Coal Board of Great Britain: (a) section and (b) plan. (After Mabry, R. E., Proceedings of ASCE, 14th Symposium on Rock Mechanics, University Park, Pennsylvania, 1973.)

Most of the subsidence in the NCB studies was found to occur within a day or two of extraction in a dish-shaped pattern over an area bounded by lines projected upward from the limits of the collapsed area at the angle α as shown in Figure 2.8. The angle of the draw was found to be in the range of 25 to 35° for beds dipping up to 25°. The angle of the draw, however, could be controlled by defects in the overlying rock, such as a fault zone providing weak rock, and by surface topography. Residual subsidence may occur for several months to a year after mining (PaDEP, 2002).

Greatest maximum subsidence Smax possible was found to be approximately equal to 90% of the seam thickness, occurring at values of panel width w/depth h (w/h) greater than about 1.2. At values of $w/h < 0.2$, the maximum subsidence was less than 10% of the seam thickness.

Critical width is the panel width required to effect maximum subsidence (Figure 2.8); as the panel width is extended into the zone of supercritical width, additional vertical subsidence does not occur, but the width of the subsiding area increases accordingly. Detailed discussion and an extensive reference list are given in Voight and Pariseau (1970).

2.3.3 Room and Pillar Method (Also "Breast and Heading" Method)

Extraction

Early Operations

In the anthracite mines of the Scranton-Wilkes-Barre and Pittsburgh areas of Pennsylvania, "first mining" proceeded historically by driving openings, called breasts, in the up-dip direction within each vein, which were then connected at frequent intervals with heading openings. The usual width of the mine openings ranged from 15 to 25 ft and the distance between center lines of adjacent breasts varied generally from 50 to 80 ft with the shorter distances in the near-surface veins and the greater distances in the deeper veins. Pillars of coal were left in place to support the roof and the room-pillar configurations were extremely variable. One example is given in Figure 2.9.

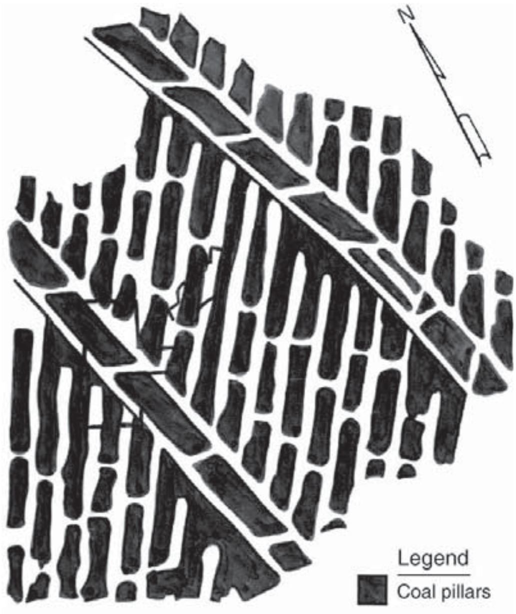


FIGURE 2.9
Plan of coal mine room and pillar layout, Westmoreland County, Pennsylvania. (From Gray, R. E. and Meyers, J. F., *Proc. ASCE J. Soil Mech. Found. Eng. Div.*, 96, 1970. With permission.)

"Robbing" occurred subsequent to first mining and consisted of removing the top and bottom benches of coal in thick veins and trimming coal from pillar sides.

Collapse occurs with time after mine closure. The coal is often associated with beds of clay shale, which soften and lose strength under sealed, humid mine conditions. Eventually failure of the pillars occurs, the roof collapses, and the load transfer to adjacent pillars causes them to collapse. If the mined area is large enough and the roof thin enough, subsidence of the surface results.

Old mines may contain pillars of adequate size and condition to provide roof support or robbed pillars, weakened and in danger of collapse. On the other hand, collapse may have already occurred.

Modern Operations

Two major coal seams of the Pennsylvanian underlie the Pittsburgh area, each having an average thickness of 6 ft: the Pittsburgh coal and the Upper Freeport coal. The Pittsburgh seam is shallow, generally within 200 ft of the surface, and has essentially been worked out. In 1970, the Upper Freeport seam was being worked to the north and east of the city where it lay at depths of 300 to 600 ft (Gray and Meyers, 1970).

In the new mines complete extraction is normally achieved. A system of entries and cross-entries is driven initially to the farthest reaches of the mine before extensive mining. Rooms are driven off the entries to the end of the mine, and when this is reached, a second, or retreat phase, is undertaken. Starting at the end of the mine, pillars are removed and the roof is permitted to fall.

In active mines where surface development is desired either no extraction or partial extraction proceeds within a zone beneath a structure determined by the "angle of the draw" (Gray and Meyers, 1970). The area of either no extraction or partial extraction is determined by taking an area 5 m or more in width around the proposed structure and projecting it downward at an angle of 15 to 25° from the vertical to the level of the mine (angle of the draw). With partial extraction, where 50% of coal pillars remain in place there is a small risk of surface subsidence, whereas with no extraction there should be no risk.

Mine Collapse Mechanisms

General

Three possible mechanisms which cause mine collapse are roof failure, pillar failure, or pillar foundation failure.

Roof Failure

Roof stability depends upon the development of an arch in the roof stratum, which in turn depends on the competency of the rock in relation to span width. In weak, fractured sedimentary rocks, this is often a very difficult problem to assess, since a detailed knowledge of the engineering properties and structural defects of the rock is required, and complete information on these conditions is difficult and costly to obtain. If the roof does have defects affecting its capability, it is likely that it will fail during mining operations and not at some later date, as is the case with pillars.

Roof support does become important when the pillars weaken or collapse, causing the span length to increase, which in turn increases the loads on the pillars. Either roof or pillars may then collapse.

Pillar Failure

The capability of a pillar to support the roof is a function of the compressive strength of the coal, the cross-sectional area of the pillar, the roof load, and the strength of the floor and roof. The cross-sectional area of the pillar may be reduced in time by weathering and spalling of its walls, as shown in Figure 2.10, to a point where it cannot support the roof and failure occurs.

FIGURE 2.10
Spalling of a coal pillar in a mine room (Pittsburgh). (Photo courtesy of Richard E. Gray.)

Pillar Punching

A common cause of mine collapse appears to be the punching of the pillar into either the roof or the floor stratum. Associated with coal beds, clay shale strata are often left exposed in the mine roof or floor. Under conditions of high humidity or a flooded floor in a closed mine, the clay shales soften and lose their supporting capacity. The pillar fails by punching into the weakened shales, and the roof load is transferred to adjacent pillars, which in turn fail, resulting in a lateral progression of failures. If the progression involves a sufficiently large area, surface subsidence can result, depending upon the type and thickness of the overlying materials.

Earthquake Forces

In January 1966, during the construction of a large single-story building in Belleville, Illinois, settlements began to occur under a section of the building, causing cracking. It was determined that the settlements may have started in late October or early November 1965. An earthquake was reported in Belleville on October 20, 1965. The site was located over an old mine in a coal seam 6 to 8 ft thick at a depth of 130 ft, which was closed initially in about 1935, then reworked from 1940 to 1943. Mansur and Skouby (1970) considered that building settlements were the result of pillar collapse and mine closure initiated by the earthquake. Some investigators, however, consider that the collapsing mine was the shock recorded in Belleville.

Subsidence over Abandoned Mines

Two types of subsidence occurring above abandoned mines have been classified; sinkholes (pits) and troughs (sags). The subsidence form usually relates to mine depth and geologic conditions.

Sinkholes (Pits)

A sinkhole is a depression on the ground surface resulting from the collapse of overburden into a limited mine opening, such as a room or entry. They usually develop where the cover over the mine is less than 50 to 100 ft. Competent strata above the mine will limit sinkhole development, but sinkholes developed over a mine in Illinois where the overburden was 164 ft deep.

Troughs

Where a pillar or pillars fail by crushing or punching into the mine roof, troughs develop. Subsidence troughs usually resemble those that form above active mines but often do not conform to the mine boundaries. Trough diameters above abandoned mines in the Northern Appalachian Coal Field commonly measure 1.5 to 2.5 times the overburden thickness.

2.3.4 Strength Properties of Coal

General

Pillar capacity analysis requires data on the strength properties of coal. A wide range of values has been obtained by investigators either by testing specimens in the laboratory, or by back-analysis in which the strength required to support an existing roof is calculated for conditions where failure has not occurred. For the determination of the stability of a working mine, the strength of fresh rock specimens governs, whereas for problems involving stability after a lapse of many years, the strength of weathered specimens of the pillar and its roof and floor support pertain.

Typical coals contain a system of orthogonal discontinuities consisting of horizontal bedding planes and two sets of vertical cracks called "cleats," which are roughly perpendicular to each other as shown in Figure 2.11. This pattern makes the recovery of undisturbed specimens difficult. Some unconfined compression strength values for coals from various locations are given in Table 2.2.

Triaxial Compression Tests

A series of triaxial compression tests was performed in the laboratory on specimens from Eire, Colorado; Sesser, Illinois; and Bruceton, Pennsylvania, by Ko and Gerstle (1973). Confining pressures of 50, 100, 250, and 600 psi were used, with the maximum pressures considered as the overburden pressure on a coal seam at a depth of 600 ft. While confining pressure was held constant, each specimen was loaded axially to failure while strains and stresses were recorded. The test specimens were oriented on three perpendicular axes (α, β, and γ), and the failure load was plotted as a function of the confining pressure P_c to obtain the family of curves presented in Figure 2.12.

It was believed by the investigators that the *proportional limit* represents the load level at which microcracking commences in the coal. The safe limit is an arbitrary limit on the applied load set by the investigators to avoid internal microcracking.

2.3.5 Investigation of Existing Mines

Data Collection

Collection of existing data is a very important phase of investigation for projects that are to be constructed over mines. The data should include information on local geology, local

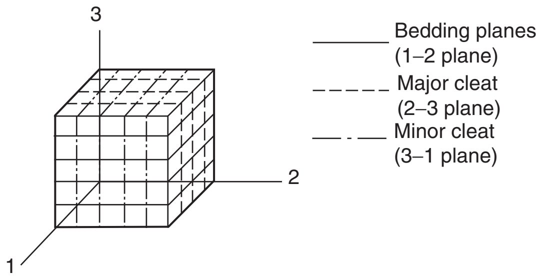


FIGURE 2.11
Bedding and cleat orientation of coal and the principal material axis. (From Ko, A. Y. and Gerstle, K. H., *Proceedings of ASCE, 14th Symposium on Rock Mechanics,* University Park, Pennsylvania, 1973, pp. 157–188. With permission.)

TABLE 2.2
Unconfined Compression Strength Values for Coal

Specimen Source	Description	Strength kg/cm^2	Comments	Reference
South Africa	2 ft cube	19	Failure sudden	Voight and Pariseau (1970)
Not given	Not given	50–500	None	Fanner (1968)
Pittsburgh coal	Sound pillar	~57	Pillar with firm bearing. H=3 m, sides, 4.8×4.8 m	Greenwald et al. (1941)
Anthracite from Pennsylvania	1 in. cubes	200,[a] 404[a]	First crack appears Crushing strength	Mabry (1973) from Griffith and Conner (1912)

[a] Values selected by Mabry (1973), from those obtained during a comprehensive study of the strength of 116 cubic specimens reported by Griffith and Conner (1912).

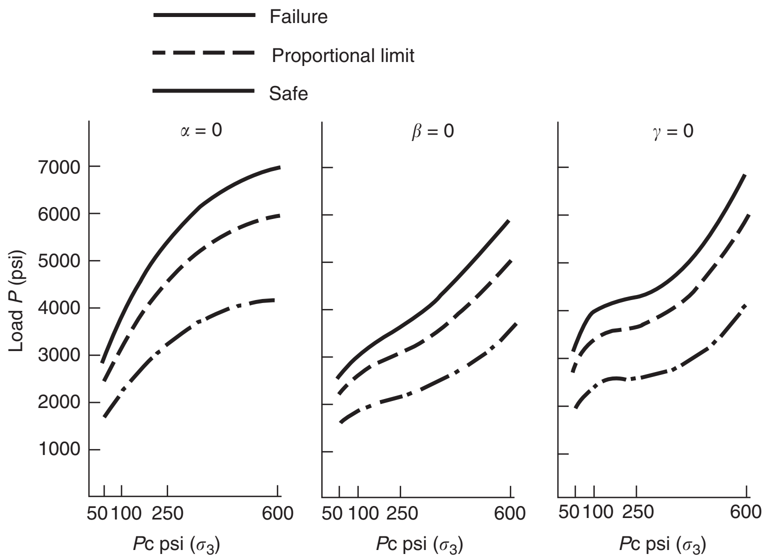


FIGURE 2.12
Triaxial compression tests results from coal specimens. (From Ko, A. Y. and Gerstle, K. H., *Proceedings of ASCE, 14th Symposium on Rock Mechanics,* University Park, Pennsylvania, 1973, pp. 157–188. With permission.)

subsidence history, and mining operations beneath the site. Many states and municipalities have extensive catalogs of abandoned mine maps. For example, the Ohio Division of Geological Survey has maps for 4138 mines, most of which are coal mines.

If possible, data on mining operations beneath the site should be obtained from the mining company that performed the extraction. The data should include information on the mine limits, percent extraction, depth or depths of seams, pillar dimensions, and the closure date. Other important data that may be available include pillar conditions, roof and floor conditions, flooding incidence, the amount of collapse that has already occurred, and accessibility to the mine.

Explorations

Exploration scopes will vary depending upon the comprehensiveness of the existing data and the accessibility of the mine for examination. Actual inspection of mine conditions is extremely important, but often not possible in old mines.

Preliminary explorations where mine locations and collapse conditions are unknown or uncertain may include the use of:

- Gravimeters to detect anomalies indicating openings.
- Rotary probes, if closely spaced, to detect cavities and indicate collapse conditions.
- Borehole cameras to photograph conditions, and borehole TV cameras, some of which are equipped with a zoom lens with an attached high-intensity light, to inspect mines remotely.
- Acoustical emissions devices, where mines are in an active collapse state, to locate the collapse area and monitor its growth.
- Electrical resistivity using the pole–dipole method was used in a study in Scranton, Pennsylvania, where the abandoned coal seam was 45 to 75 ft below the surface. Three signatures were interpreted: intact rock, caved rock, and voids. Test borings confirmed the interpretations.

Detailed study of conditions requires core borings to obtain cores of the roof, floor, and pillars, permitting an evaluation of their condition by examination and laboratory testing.

Finite-Element Method Application

General

The prediction of surface distortions beneath a proposed building site caused by the possible collapse of old mines by use of the finite element method is described by Mabry (1973). The site is in the northern anthracite region of Pennsylvania near Wilkes-Barre, and is underlain by four coal beds at depths ranging from 260 to 600 ft, with various percentages of extraction *R*, as illustrated in Figure 2.13.

Finite Element Model

Pillar analysis revealed low safety factors against crushing and the distinct possibility of subsidence. To evaluate the potential subsidence magnitude, a finite-element model was prepared incorporating the geometry of the rock strata and mines to a depth of 700 ft, and engineering properties including density, strength, and deformation moduli of the intervening rock and coal strata. The finite element mesh is given in Figure 2.14.

Analysis and Conclusions

Gravity stresses were imposed and the ground surface subsidence due to the initial mining in the veins was determined. Subsequent analysis was made of the future surface distortions, such as those would be generated by pillar weathering and eventual crushing. Pillar weathering was simulated by reducing the joint stiffness, and pillar collapse or yield was simulated by setting the joint stiffness in the appropriate intervals of the coal seams to zero. After pillar weathering in the coal seams was evaluated by changing joint stiffness values, the intervals of the veins were "collapsed" in ascending order of computed safety factors for several extraction ratios. The results are summarized in Table 2.3.

After an evaluation of all of the available information, it was the judgment of the investigator that the more realistic case for plant design was Case 2 (Table 2.3), and that the probability of Cases 3 and 4 developing during the life of the structure was very low.

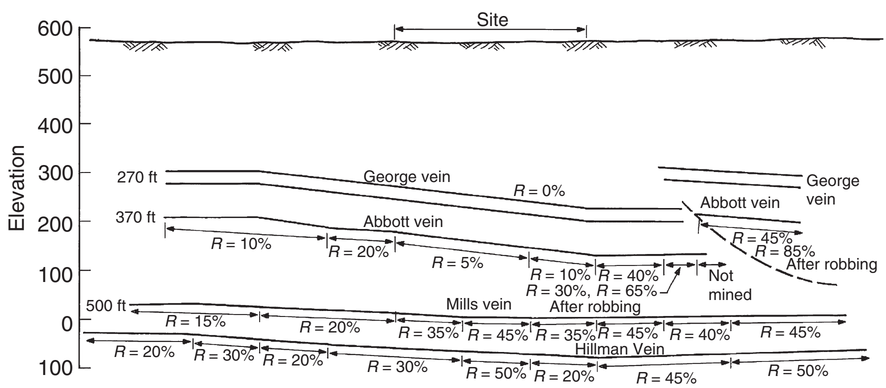


FIGURE 2.13
Section illustrating coal mines and percent extraction *R* beneath a proposed construction site near Wilkes-Barre, Pennsylvania. (From Mabry, R. E., *Proceedings of ASCE, 14th Symposium on Rock Mechanics,* University Park, Pennsylvania, 1973. With permission.)

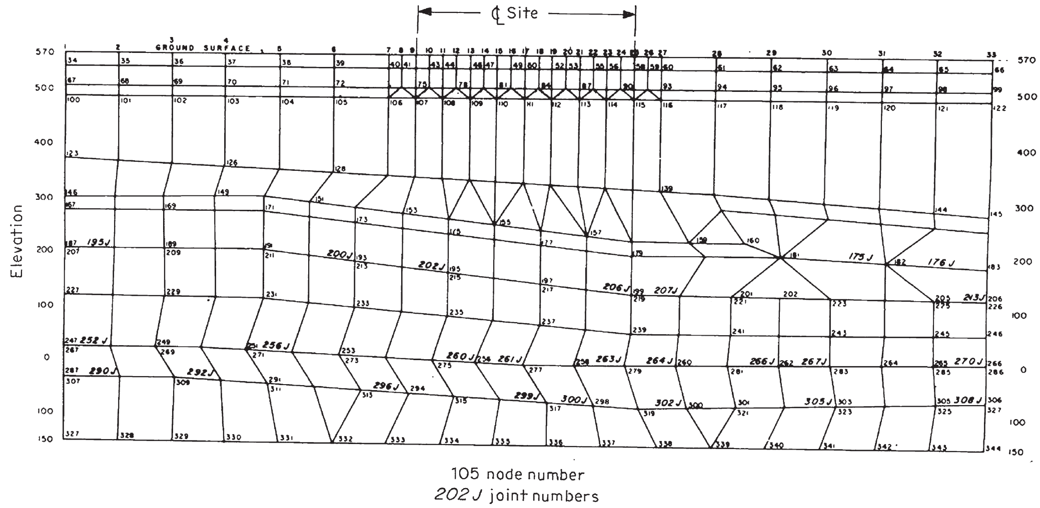


FIGURE 2.14
Finite-element mesh for coal mine conditions beneath site near Wilkes-Barre, Pennsylvania. (Mabry, R. E., *Proceedings of ASCE, 14th Symposium on Rock Mechanics,* University Park, Pennsylvania, 1973. With permission.)

TABLE 2.3
Summary of Finite-Element Analysis of Coal Mine Study[a]

Condition	Safety Factor against Pillar Failure	Cumulative Settlement (cm)	Maximum Distortion within Plant Site: Angular Rotation (rad)	Maximum Distortion within Plant Site: Horizontal Strain (+ = tension)
Weathering in all veins		1.0	20.0×10^{-6}	$\pm12.0=10^{-6}$
Collapse in Hillman, R=50%	0.80	4.0	3.6×10^{-4}	$+1.7\times10^{-4}$
Collapse in Mills and Hillman, R=40%	0.89 to 0.99	29.0	3.0×10^{-3}	$+10.1\times10^{-4}$
Collapse in Mills, R=30%	1.1	63.2	4.8×10^{-3}	$+19.2\pm10^{-4}$

[a] From Mabry, R. E., *Proceedings of ASCE, 14th Symposium on Rock Mechanics*, University Park, Pennsylvania, June 1972. With permission.

2.3.6 Subsidence Prevention and Control and Foundation Support

New Mines

In general, new mines should be excavated on the basis of either total extraction, permitting collapse to occur during mining operations (if not detrimental to existing overlying structures), or partial extraction, leaving sufficient pillar sections to prevent collapse and resulting subsidence at some future date. Legget (1972) cites the case where the harbor area of the city of Duisburg, West Germany, was purposely lowered 1.75 m by careful, progressive longwall mining of coal seams beneath the city, without damage to overlying structures.

Old Mines

Solutions are based on predicted distortions and their probability of occurrence.

Case 1

No or small surface distortions are anticipated when conditions include adequate pillar support, or complete collapse has occurred, or the mined coal seam is at substantial depth overlain by competent rock. Foundations may include mats, doubly reinforced continuous footings, or articulated or flexible design to allow compensation for some differential movements of structures.

Case 2

Large distortions are anticipated or small distortions cannot be tolerated, when pillar support is questionable, collapse has not occurred, and the mine is at relatively shallow depths. Solutions may include:

- Relocate project to a trouble-free area.
- Provide mine roof support with construction of piers in the mine or installation of grout columns, or completely grout all mine openings from the surface within the confines of a grout curtain installed around the site periphery.
- Install drilled piers from the surface to beneath the mine floor.

2.4 Solution of Rock

2.4.1 General

Significance

Ground subsidence and *collapse* in soluble rock masses can result from nature's activities, at times aided by humans, or from human-induced fluid or solid extraction. Calcareous rocks, such as limestone, dolomite, gypsum, halite, and anhydrite, are subject to solution by water, which causes the formation of cavities of many shapes and sizes. Under certain conditions, the ground surface over these cavities subsides or even collapses, in the latter case forming sinkholes.

The Hazard

Geographic distribution is widespread, and there are many examples in the literature of damage to structures and even deaths caused by ground collapse over soluble rocks. Examples are the destruction of homes in central Florida (Sowers, 1975) (Figure 2.15); the sudden settlement of a seven-story garage in Knoxville, Tennessee (ENR, 1978); and a foundation and structural failure in an Akron, Ohio, department store that resulted in 1 dead and 10 injured (ENR, 1969). Collapses resulting in substantial damage and in some cases deaths have also been reported for locations near Johannesburg and Paris (see Section 2.4.3). Subsidence and sinkholes associated with the removal of halite have been reported for areas around Detroit, Michigan; Windsor, Ontario; and Hutchinson, Kansas.

Collapse incidence is much less than that for slope failures, but nevertheless the recognition of its potential is important, especially since the potential may be increasing in a given area. Collapse does occur as a natural phenomenon, but the incidence increases substantially in any given area with an increase in groundwater withdrawal.

2.4.2 Solution Phenomenon and Development

Characteristics of Limestone Formations

General

Limestone, the most common rock experiencing cavity development, is widely distributed throughout the world, and is exposed in large areas of the United States, as shown in Figure 2.16. The occurrence, structure, and geomorphology of carbonate rocks are briefly summarized in this section.

FIGURE 2.15
Collapse of two houses into a funnel-shaped sink in Bartow, Florida in 1967. Cause was ravelling of medium to fine sand into chimney-like cavities in limestone at depths of 50 to 80 ft below the surface. (Photo courtesy of George F. Sowers.)

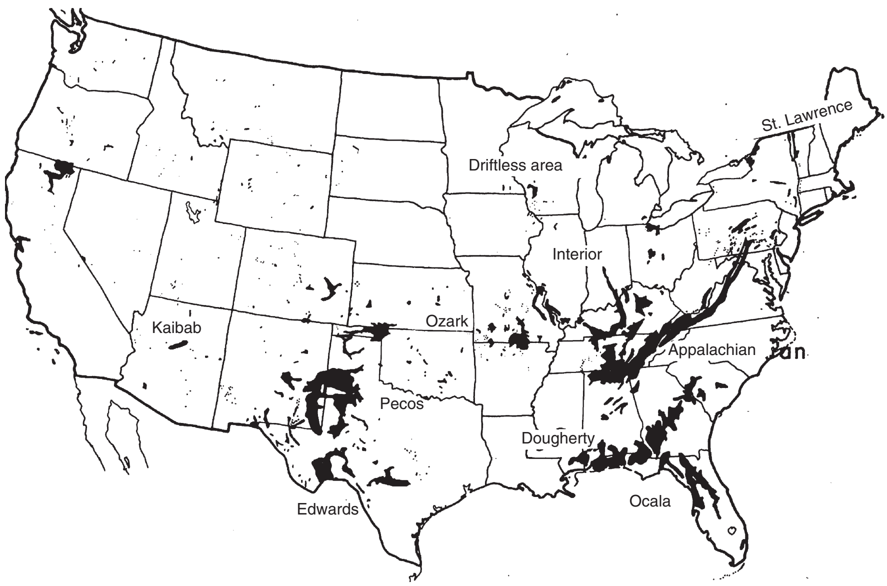


FIGURE 2.16
Distribution of karst regions in the United States. (Compiled by William E. Davies; from White, W. B., *Encyclopedia of Geomorphology*, Dowden Hutchinson & Ross Publ., Stroudsburg, Pennsylvania, 1968, pp. 1036–1039. With permission.)

Rock Purity and Cavity Growth

Purer limestone, normally found as thick beds of dense, well-indurated rock, is the most susceptible to cavity growth. At least 60% of the rock must be made up of carbonate materials for karst development, and a purity of 90% or more is required for full development (Corbel, 1959).

Impure limestone is characteristically thinly bedded and interbedded with shale and is resistant to solution.

Jointing: Groundwater moves in the rock along the joints, which are usually the result of strain energy release (residual from early compression) that occurs during uplift and rebound subsequent to unloading by erosion. This dominant origin causes most joints to be normal to the bedding planes. Major joints, cutting several beds, usually occur in parallel sets and frequently two sets intersect, commonly at about 60°, forming a conjugate joint system.

Cavity Growth, Subsidence, and Collapse

Solution

Groundwater moving through the joint system at depth and rainfall entering the joint system from above result in the solution of the rock. As rainwater passes through the surface organic layer, it becomes a weak acid that readily attacks the limestone. Solution activity is much greater, therefore, in humid climates with heavy vegetation than in dry climates with thin vegetation.

Geologic Conditions and Cavity Growth Form

Horizontal beds develop cavities vertically and horizontally along the joints, which grow into caverns as the solution progresses. Cavern growth is usually upward; surface subsidence occurs when the roof begins to deflect, or when broken rock in the cavern provides partial support, preventing a total collapse. When a cavern roof lacks adequate arch to support overburden pressures, collapse occurs and a sinkhole is formed.

Horizontal beds overlain by thick granular overburden are also subject to sudden raveling into cavities developing along joints. An example of a large sink developing under these conditions is given in Figure 2.15. Sowers (1975) states, "Raveling failures are the most widespread and probably the most dangerous of all the subsidence phenomena that are associated with limestone." The author considers this statement to apply to conditions like those in central Florida, that is, relatively thick deposits of granular alluvium overlying limestone undergoing cavity development from its surface.

Dipping beds develop cavities along joint dips, as shown in Figure 2.17, creating a very irregular rock surface, characterized by pinnacles. As the cavity grows, the overburden moves into the void, forming a soil arch. With further growth the arch collapses and a sinkhole results. In granular soils, the soil may suddenly enter a cavity by raveling, wherein the arch migrates rapidly to the surface, finally collapsing. A pinnacled rock surface is typical of eastern Pennsylvania, as illustrated in Figure 2.18. In addition to sinkholes resulting from soil raveling into cavities, the very irregular rock surface poses difficult foundation conditions.

Natural Rate of Cavity Growth

It has been estimated that the rainfall in the sinkhole region of Kentucky will dissolve a layer of limestone 1 cm thick in 66 years (Flint et al., 1969). Terzaghi (1913), reporting on a geologic study that he made in the Gacka region of Yugoslavia, observed that solution proceeded much more rapidly in heavily forested areas than in areas covered lightly by grass or barren of vegetation. In an analysis he assumed that 60% of the annual rainfall, or 700 mm/year, entered the topsoil and that the entire amount of carbon dioxide developed

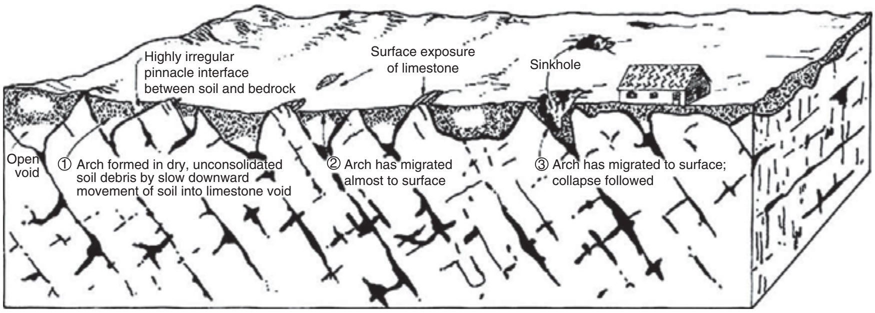


FIGURE 2.17
Hypothetical section through a carbonate valley showing stages of sinkhole development. (From Pennsylvania Geological Survey, *Engineering Characteristics of the Rocks of Pennsylvania*, Pennsylvania Geological Survey, 1972. With permission.)

FIGURE 2.18
Very irregular rock surface (pinnacles) and cavities in quarry wall. House in upper left gives scale (Ledger dolomite, Upper Merion, Pennsylvania).

in the topsoil was used up in the process of solution, which removed the limestone at the rate of 0.5 mm/year, or 1 cm in 20 years.

Collapse Causes Summarized

Collapse of limestone cavities can result from:

- Increase in arch span from cavity growth until the strength is insufficient to support the overburden weight.
- Increase in overburden weight over the arch by increased saturation from rainfall or other sources, or from groundwater lowering, which removes the buoyant force of water.
- Entry of granular soils by raveling into a cavity near the rock surface.
- Applications of load to the surface from structures, fills, etc.

Geomorphic Features of Karst

Karst refers in general to the characteristic, readily recognizable terrain features that develop in the purer limestone. The important characteristics of karst topography are its predominantly vertical and underground drainage, lack of surface drainage systems, and the development of circular depressions and sinks. At times, streams flow a short distance and suddenly disappear into the ground.

Youthful karst is characterized by numerous sinkholes and depressions as well as deranged and intermittent drainage.

Mature karst is characteristic of humid tropical climates. The landform consists of numerous rounded, steep-sided hills ("haystacks" or "pepinos").

Buried karst is illustrated by the ERTS image of Florida, showing numerous lakes that have filled subsidence depressions. In the Orlando region, the limestone is often buried under 60 to 100 ft of alluvium.

Groundwater Pumping Effects

Significance

Groundwater withdrawal greatly accelerates cavity growth in soluble rocks, and lowering of the water table increases overburden pressures. The latter activity, which substantially increases the load on a naturally formed arch, is probably the major cause of ground subsidence and collapse in limestone regions (Prokopovich, 1976). Even if groundwater withdrawal is controlled with the objective of maintaining a water balance and preserving the natural water table, the water table drops during severe and extensive droughts and collapse activity increases significantly, as occurred in central Florida during the spring of 1981.

Examples

Pierson, Florida: The sink illustrated in Figure 2.19, 20 m in diameter and 13 m deep, formed suddenly in December 1973 after 3 days of continuous pumping from nearby irrigation wells. The limestone is about 30 m in depth.

Johannesburg, South Africa: A large pumping program was begun in 1960 to dewater an area, underlain by up to 1000 m of Transvaal dolomite and dolomitic limestone, for gold mining operations near Johannesburg. In December 1962, a large sinkhole developed suddenly under the crushing plant adjacent to one of the mining shafts, swallowed the entire plant, and took 29 lives. In 1964, the lives of five persons were lost when their home suddenly fell into a rapidly developing sinkhole. Between 1962 and 1966, eight sinkholes larger than 50 m in diameter and 30 m in depth had formed in the mine area (Jennings, 1966).

Paris, France: Groundwater withdrawal has increased the solution rate and cavern growth in old gypsum quarries beneath the city and some suburban towns (Arnould, 1970). Ground collapse has occurred, causing homes to be lost and industrial buildings to be damaged. In one case, it was estimated that pumping water from gypsum at the rate of about 85 ft^3/min removed 136 lb of solids per hour.

Hershey, Pennsylvania: Increased dewatering for a quarry operation caused groundwater levels to drop over an area of 5000 acres and soon resulted in the appearance of over 100 sinkholes (Foose, 1953). The original groundwater levels were essentially restored after the quarry company sealed their quarry area by grouting.

Round Rock, Texas: The Edwards limestone outcrops in the area and is known to be cavernous in some locations. During a study for new development, interpretation of stereo-pairs of air photos disclosed several sink holes and depressions in one area of the

FIGURE 2.19
Sink 65 ft in diameter and 42 ft deep formed suddenly in December 1973 in Pierson, Florida, after 3 days of continuous pumping from nearby irrigation wells. The limestone is about 100 ft in depth. (Photo: Courtesy of George F. Sowers.)

site as illustrated in Figure 2.20 (Hunt, 1973). During reconnaissance of the site in 1972, the discovery of a sink 20 ft wide and 6 ft deep (Figure 2.21) that did not appear on the aerial photos, dated 1969, indicated recent collapse activity. Since the Edwards overlies the major aquifer in the area and groundwater withdrawals have increased along with area development, it may be that the incidence of collapse activity is increasing.

2.4.3 Investigation

Preliminary Phases

Data Collection

The existing geologic data are gathered to provide information on regional rock types and their solubility, bedding orientation and jointing, overburden types and thickness, and local aquifers and groundwater withdrawal.

Landform Analysis

Topographic maps and remote-sensing imagery are interpreted by landform analysis techniques. Some indicators of cavernous rock are:

- *Surface drainage*: Lack of second- and third-order streams; intermittent streams and deranged drainage; and streams ending suddenly.
- *Landform*: Sinks and depressions or numerous dome-shaped, steep-sided hills with sinks in between.
- *Photo tone*: Soils in slight depressions formed over cavity development will have slightly higher moisture contents than those in adjacent areas and will show as slightly darker tones on black-and-white aerial photos such as Figure 2.20. Infrared is also useful in detecting karstic features because of differences in soil moisture.

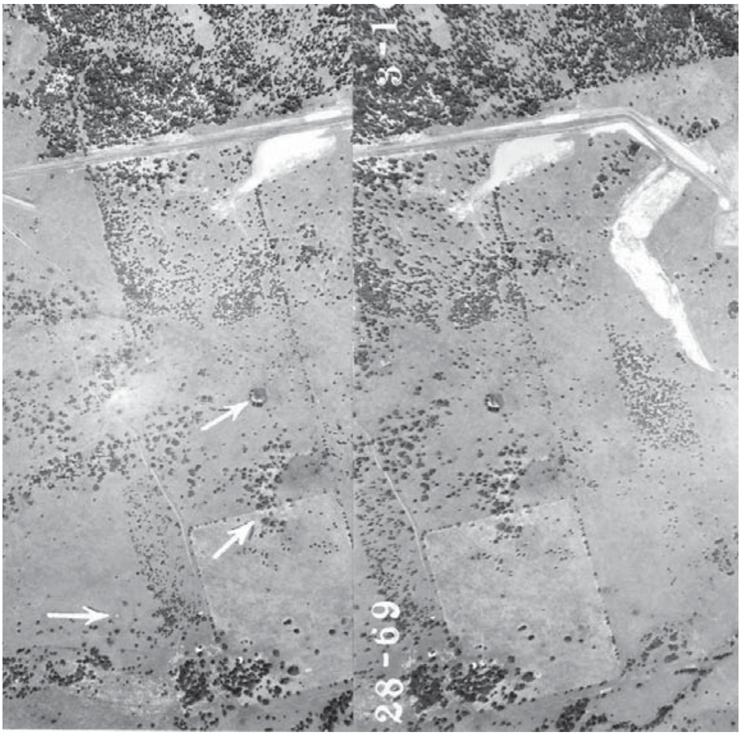


FIGURE 2.20
Stereo-pair of aerial photos of area near Round Rock, Texas, showing sinks and depressions from partial collapse of shallow limestone. (From Hunt, R. E., *Bull. Assoc. Eng. Geol.,* 10, 1973. With permission.)

FIGURE 2.21
Recent sink, not shown in aerial photos dated 1969 (Figure 2.19). Photo taken in 1972. (From Hunt, R. E., *Bull. Assoc. Eng. Geol.,* 10, 1973. With permission.)

Preliminary Evaluation

From a preliminary evaluation of the data, judgments are formulated regarding the potential for or the existence of cavity development from natural or human causes, and the possible locations, type, and size of ground subsidence and collapse. From this data base explorations are programmed.

Explorations

General

The location of all important cavities and the determination of their size and extent is a very difficult and usually impossible task. Explorations should never proceed without completion of data collection and landform analysis.

Geophysics

Explorations with geophysical methods can provide useful information, but the degrees of reliability vary. Seismic refraction surveys may result in little more than "averaging" the depth to the limestone surface if it is highly irregular. Seismic-direct cross-hole surveys may indicate the presence of cavities if they are large. Electrical resistivity, at times, has indicated shallow cavity development.

Gravimeter surveys were found to be more useful than other geophysical methods (seismic cross-hole surveys or electrical resistivity) in a study for a nuclear power plant in northwestern Ohio (Millet and Moorhouse, 1973).

Ground-probing radar (GPR) may disclose cavities, but interpretation of the images is difficult.

Test Borings and Pits

Test and core borings are programmed to explore anomalies detected by geophysical explorations and terrain analysis as well as to accomplish their normal purposes. Voids are disclosed by the sudden drop of the drilling tools and loss of drilling fluid. The material in a sinkhole is usually very loose compared with the surrounding materials, and may overlie highly fractured rock where a roof has collapsed. One should be suspicious when low SPT N values, such as 2 or 3, are encountered immediately above the rock surface, following significantly higher N values recorded in the overlying strata.

Test pits are useful to allow examination of the bedrock surface. Although the normal backhoe reach is limited to 10 to 15 ft, on important projects, such as for dams or construction with heavy foundation loads, deeper excavations, perhaps requiring dewatering, may be warranted.

Rotary Probes and Percussion Drilling

If cavity presence has been confirmed, it is usually prudent to make either core borings, rotary probes, or pneumatic percussion drill holes at the location of each footing before final design, or before construction. The objective is to confirm that an adequate thickness of competent rock is present beneath each foundation.

Percussion drilling with air track rigs is a low-cost and efficient method of exploring for cavities and the rock surface, especially in dipping limestone formations. Core borings usually drill 20 to 40 ft/day, whereas air track probes readily drill 300 to 500 ft/day. During investigation several core borings should be drilled immediately adjacent to air track holes for rock-quality correlations.

Proof testing with air drills is an alternate to rotary probes at each footing location, particularly for drilled piers where installations are relatively deep. Proof testing the bottom of each pier founded on rock with air drills is much less costly than rotary probes or core borings.

Evaluation and Analysis

Basic Elements

The following elements should be considered:

- Rock bedding, i.e., horizontal vs. dipping.
- Overburden thickness and properties and thickness variations, which are likely to be substantial.
- Bedrock surface characteristics, i.e., weathered, sound, relatively sound and smooth with cavities following joint patterns from the rock surface, or highly irregular in configuration and soundness (see Figure 2.17).
- Cavities within the rock mass — location, size, and shape.
- Arch characteristics, i.e., thickness, span, soundness, and joint characteristics and properties.
- Groundwater depth and withdrawal conditions, i.e., present vs. future potential.

Analysis

Analysis proceeds in accordance with a normal foundation study (except where foundations may overlie a rock arch, then rock-mechanics principles are applied to evaluate the minimum roof thickness required to provide adequate support to foundations). A generous safety factor is applied to allow for unknown rock properties.

2.4.4 Support of Surface Structures

Avoid the High Hazard Condition

Project relocation should be considered where cavities are large and at relatively shallow depths, or where soluble rock is deep but overlain by soils subject to raveling. The decision is based on the degree of hazard presented, which is directly related to the occurrence of groundwater withdrawal or its likelihood in the future. Groundwater withdrawal represents very high hazard conditions. In fact, the probability and effects of groundwater withdrawal are the most important considerations in evaluating sites underlain by soluble rock.

Foundation Treatments

Dental Concrete

Cavities that can be exposed by excavation can be cleaned of soil and filled with lean concrete, which provides suitable support for shallow foundations.

Grouting

Deep cavities that cannot be reached by excavation are often filled by grout injection, but the uncertainty will exist that not all cavities and fractures have been filled, even if check explorations are made subsequent to the grouting operations. Grouting has the important advantage of impeding groundwater movement and therefore cavity growth, even where pumping is anticipated.

Deep Foundations

Deep, heavily loaded foundations, or those supporting settlement-sensitive structures, when founded on, or rock-socketed in, soluble rock, should be proof-tested, whether grouted or not (see discussion of explorations in Section 2.4.3).

2.5 Soil Subsidence and Collapse

2.5.1 General

Causes

Subsidence in soils results from two general categories of causes:

1. Compression refers to the volume reduction occurring under applied stress from grain rearrangement in cohesionless soil or consolidation in a cohesive soil. The phenomenon is very common and always occurs to some degree under foundation loadings.
2. Collapse is the consequence of a sudden closure of voids, or a void, and is the subject of this chapter. Collapsible or metastable soils undergo a sudden decrease in volume when internal structural support is lost; piping soils are susceptible to the formation of large cavities, which are subject to collapse.

The Hazard

Subsidence from compression or soil collapse is a relatively minor hazard, resulting in structural distortions from differential settlements.

Piping erosion forms seepage channels in earth dams and slopes and in severe cases results in the collapse of the piping tunnel, which can affect the stability of an earth dam or a natural or cut slope.

2.5.2 Collapsible or Metastable Soils

Collapse Mechanisms

Temporary Internal Soil Support

Internal soil support, which is considered to provide temporary strength, is derived from a number of sources. Included are capillary tension, which provides temporary strength in partially saturated fine-grained cohesionless soils; cementing agents, which may include iron oxide, calcium carbonate, or clay in the clay welding, of grains; and other agents, which include silt bonds, clay bonds, and clay bridges, as illustrated in Figure 2.22.

Collapse Causes

Wetting destroys capillary bonds, leaches out cementing agents, or softens clay bonds and bridges in an open structure. Local shallow wetting occurs from surface flooding or broken pipelines, and subsidence can be substantial and nonuniform. Intense, deep local wetting from the discharge of industrial effluents or irrigation can also result in substantial and nonuniform subsidence. A slow and relatively uniform rise in the groundwater level usually results in uniform and gradual subsidence.

Increased saturation under an applied load can result in gradual settlement, or in a sudden collapse as the soil bonds are weakened.

Applied load of critical magnitude can cause a sudden collapse of the soil structure when the bonds break in a brittle type of failure, even at natural moisture content.

Susceptible Soils

Loess

Loess is a fine-grained aeolian deposit.

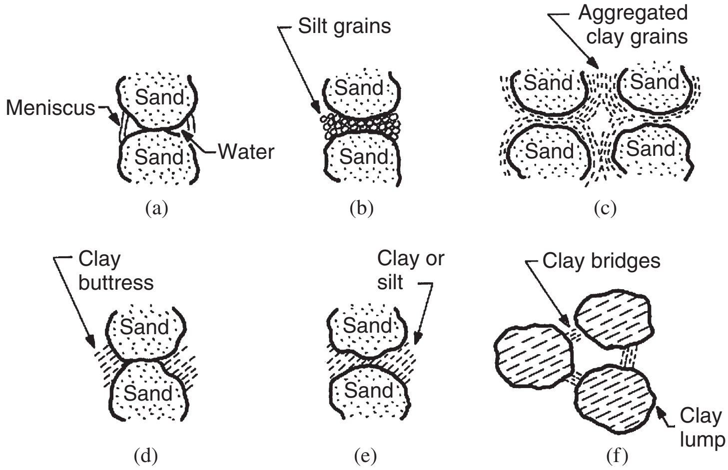


FIGURE 2.22
Typical collapsible soil structures: (a) capillary tension; (b) silt bond; (c) aggregated clay bond; (d) flocculated clay bond; (e) mudflow type of separation; (f) clay bridge structure. (From Clemence, S. P. and Finbarr, A. O., *Proceedings of ASCE*, Preprint 80–116, 1980, 22 pp. Adapted from Barden, L. et al., *Engineering Geology*, 1973, pp. 49–60.)

Valley Alluvium: Semiarid to Arid Climate

In arid climates, occasional heavy rains carry fine soils and soluble salts to the valley floor to form temporary lakes. The water evaporates rapidly leaving a loosely structured, lightly cemented deposit susceptible to subsidence and erosion. Often characteristic of these soils are numerous steep-sided gulleys as shown in the oblique aerial photo (Figure 2.23) taken near Tucson, Arizona. Tucson suffers from the collapsing soil problem, and damage to structures has been reported (Sultan, 1969).

Studies of subsidence from ground collapse were undertaken by the U.S. Bureau of Reclamation (USBR) to evaluate problems and solutions for the construction of the California aqueduct in the San Joaquin Valley of California (Curtin, 1973). Test sites were selected after extensive ground and aerial surveys of the 2000 mi^2 study area. It is interesting to note that the elevation of the entire valley had undergone *regional subsidence* and had been lowered by as much as 30 ft by groundwater withdrawal since the early 1920s.

USBR test procedures to investigate subsidence potential involved either inundating the ground surface by ponding or filling bottomless tanks with water as shown in Figure 2.24. One of the large ponds overlays 250 ft of collapsible soils. Water was applied to the pond for 484 days, during which an average settlement of 11.5 ft occurred. Benchmarks had been set at the surface and at 25 ft intervals to a depth of 150 ft. A plot of the subsidence and compaction between benchmarks as a function of time is given in Figure 2.25. It is seen that the effects of the test influenced the soils to a depth of at least 150 ft. The subsidence appears to be the summation of soil collapse plus compression from increasing overburden pressure due to saturation. The test curves show an almost immediate subsidence of 1 ft upon saturation within the upper 25 ft; thereafter, the shape of the curves follows the curve expected from normal consolidation.

San Joaquin Valley soils are described by Curtin (1973). They have a texture similar to that of loess, characterized by voids between grains held in place by clay bonds, with bubble cavities formed by entrapped air, interlaminar openings in thinly laminated sediments, and unfilled polygonal cracks and voids left by the disintegration of entrapped vegetation. The classification ranges from a poorly graded silty sand to a clay, in general with more

FIGURE 2.23
Fine-grained valley fill near Tucson, Arizona. The steep-sided gullies form in easily eroded collapsible soils.

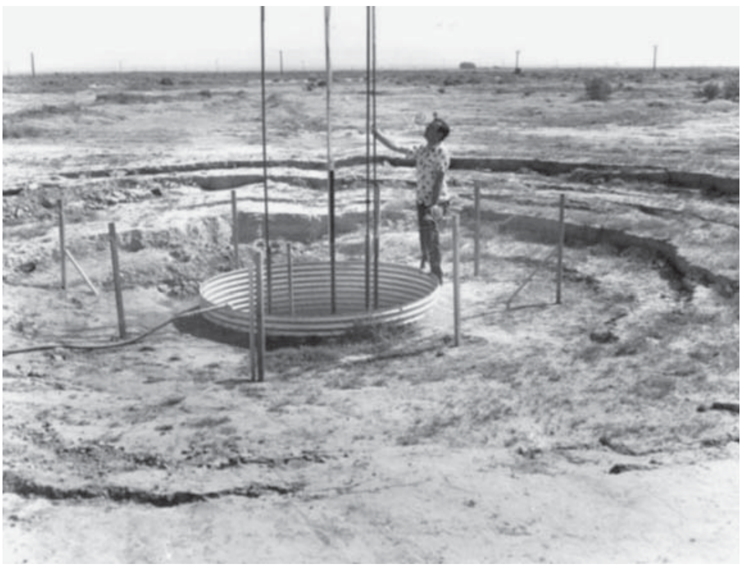


FIGURE 2.24
Subsidence after 3 months caused by ground saturation around test plot in San Joaquin Valley, California. (From Curtin, G., *Geology, Seismicity and Environmental Impact, Special Publication* Association Engineering, Geology, University Publishers, Los Angles, 1973. With permission.)

than 50% passing the 220 sieve. Dry density ranges from 57 to 110 pcf with a porosity range of 43 to 85%. The predominant clay mineral is montmorillonite, and the clay content of collapsing soils was reported to be from 3 to 30%. The observation was made that soils

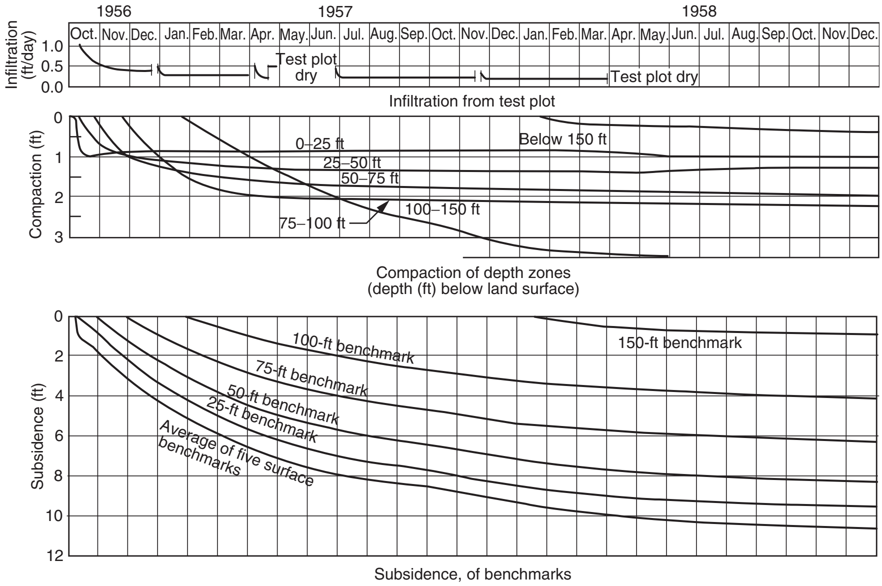


FIGURE 2.25
Subsidence measured by benchmarks, test pond B, west side of San Joaquin, California. (From Curtin, G., *Geology, Seismicity and Environmental Impact, Special Publication* Association Engineering, Geology, University Publishers, Los Angles, 1973. With permission.)

with a high clay content tended initially to swell rather than collapse. During field testing, the benchmarks on the surface first rose upon inundation as the montmorillonite swelled; soon subsidence overcame the swelling and the benchmarks moved downward. A laboratory consolidation test curve showing soil collapse upon the addition of water is given in Figure 2.26.

In describing the soils in western Fresno County, Bull (1964) notes that maximum subsidence occurs where the clay amounts to about 12% of solids; below 5% there is little subsidence and above 30% the clay swells.

Residual Soils

Collapse has been reported to occur in residual soils derived from granite in South Africa and northern Rhodesia (Brink and Kantey, 1961), and from sandstone and basalt in Brazil (Vargas, 1972).

Porous clays of Brazil (argila porosa) occur intermittently in an area of hundreds of square kilometers ranging through the central portions of the states of Sao Paulo, Parana, and Santa Catarina. The terrain consists of rolling savannah and the annual rainfall is about 1200mm (47 in.), distributed primarily during the months of December, January, and February. The remaining 9 months are relatively dry. Derived from Permian sandstone, Triassic basalt, and Tertiary sediments, the soils are generally clayey. Typical gradation curves are given in Figure 2.27, and plasticity index ranges in Figure 2.28.

The upper zone of these formations, to depths of 4 to 8 m, typically yields low SPT values, ranging from 0 to 4 blows/ft; void ratios of 1.3 to 2.0 are common. Below the “soft” (dry) upper zone a hard crust is often found. A typical boring log is given in Figure 2.29; it is to be noted that groundwater was not encountered, which is the normal condition. Some laboratory test data are also included in the figure.

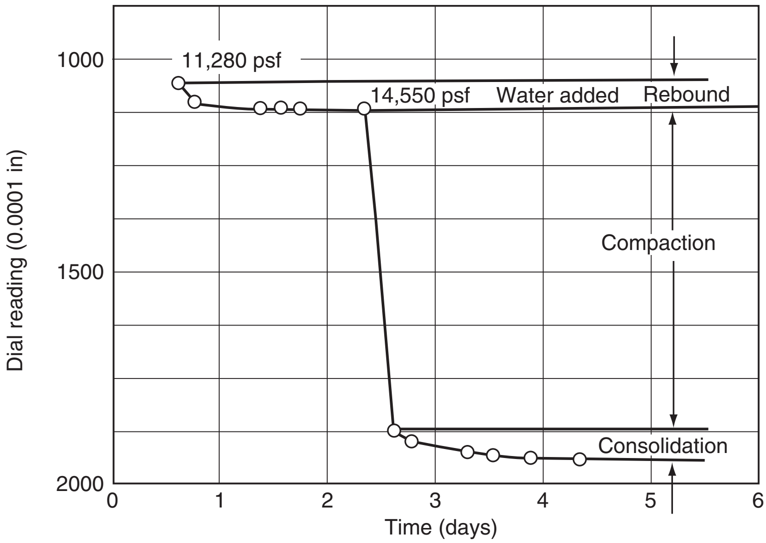


FIGURE 2.26
Laboratory consolidation test curve of compression vs. time for a collapsible soil from the San Joaquin Valley. (From Curtin, G., *Geology, Seismicity and Environmental Impact, Special Publication* Association Engineering, Geology, University Publishers, Los Angles, 1973. With permission.)

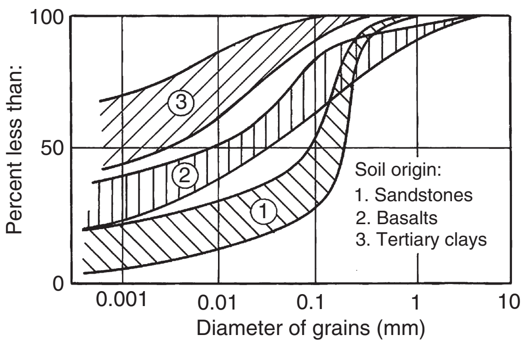


FIGURE 2.27
Gradation curves for typical porous clays of Brazil. (From Vargas, 1972.)

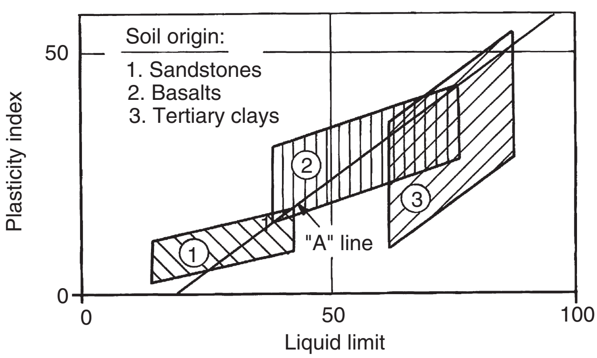


FIGURE 2.28
Relationship between plasticity index and liquid limit, porous clays of Brazil. (From Vargas, 1972.)

Although the soil is essentially a clay, its open, porous structure provides for high permeability and the rapid compression characteristics of a sand, hence the term "porous clay." The porous characteristics are the result of the leaching out of iron and other minerals that are carried by migrating water to some depth where they precipitate to form

Depth (m)	"N"	Soil type	LL	PI	w (%)	γ	e
0							
1 ■	3	Very soft to soft	45	21	32	1.25	1.94
2 ■	2	Red silty clay	46	19	31	1.19	2.01
3 ■	5	(CL)	48	24	34	1.51	1.53
4 ■	5		43	16	32	1.39	1.39
5	10	Stiff tan to brown					
6	12	Very silty clay					
7	16						
8	14						
9	11						
10	14						
11	14						
12	36	Becoming hard					
13	50						
14	31						
15	5/1 cm (refusal)						

Laboratory test data

Note:

■ – Block sample

γ_t – gm/cm^3

N – Blows per foot (SPT)

GWL – Groundwater table not encountered

FIGURE 2.29
Test boring log and laboratory test data for a porous clay derived from basalt (Araras, Sao Paulo, Brazil).

the aforementioned hard zone, which often contains limonite nodules. The effect of saturation on a consolidation test specimen is given in Figure 2.30, a plot of void ratio vs. pressure. The curve is typical of collapsing soils. In the dry condition strengths are high, and excavation walls will stand vertical for heights greater than 4 or 5 m without support in the same manner as loess.

The recognition of porous clays can often be accomplished by terrain analysis. Three factors appear to govern the development of the weak, open structure: a long relatively dry period followed by heavy summer rains, a relatively high ground elevation in rolling, hilly terrain with a moderately deep water table, and readily leachable materials. An examination of aerial photos, such as Figure 2.31, reveals unusual features for a clay soil: characteristically thin vegetation and lack of any surface drainage system, both indicative of the open porous structure. Here and there, where terrain is relatively level, bowl-shaped areas, often 3 to 4 m deep and 20 m across, with no apparent existing drainage, seem to indicate areas of possible natural collapse (Figure 2.31). These may have occurred during periods of very heavy rains, which either created ponds or fell on zones that had been very much weakened by leaching.

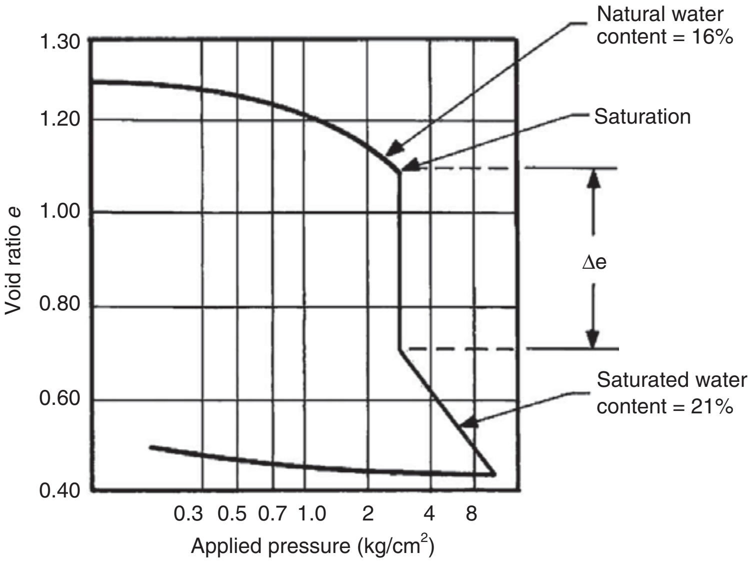


FIGURE 2.30
Effect of saturation on the pressure vs. void ratio curve of a porous clay from Brazil. (From Vargas, M., *VIII Seminario Nacional de Grandes Barragems*, São Paulo, 1972. With permission.)

FIGURE 2.31
Stereo-pair of aerial photos of area of porous clays showing probable collapse zones (state of São Paulo, Brazil). Although residual soils are clays, the lack of drainage patterns indicates high infiltration. (From Hunt, R. E. and Santiago, W. B., *Proceedings 1, Congresso Brasileiro de Geologia de Engenharia,* Rio de Janeiro, 1976, pp. 79–98. With permission.)

2.5.3 Predicting Collapse Potential

Preliminary Phases

Data Collection

A preliminary knowledge of the local geology from a literature review aids in anticipating soils with a collapse potential, since they are commonly associated with loess and other fine-grained aeolian soils, and fine-grained valley alluvium in dry climates. The susceptibility of residual soil is difficult to determine from a normal literature review. Rolling terrain with a moderately deep water table in a climate with a short wet season and a long dry season should be suspect.

Landform Analysis

Loess and valley alluvium are identified by their characteristic features. Residual soils with collapse potential may show a lack of surface drainage channels indicating rainfall infiltration rather than runoff, especially where the soils are known to be clayey. Unexplained collapse depressions may be present.

Explorations

Test Borings and Sampling

Drilling using continuous-flight augers (no drilling fluid) will yield substantially higher SPT values in collapsible soils than will drilling with water, which tends to soften the soils. A comparison of the results from both methods on a given project provides an indication of collapse potential. Undisturbed sampling is often difficult in these materials. Representative samples, suitable for laboratory testing, were obtained with the Denison core barrel in the Negev Desert of Israel.

In residual soils, a "soft" upper zone with low SPT values will be encountered when drilling fluids are used, often underlain by a hard zone or crust which may contain limonite nodules or concretions.

Test pits are useful for close examination and description of the soils in the undisturbed state, *in situ* natural density tests, and the recovery of block samples for laboratory testing.

Simple Hand Test

A hand-size block of the soil is broken into two pieces and each is trimmed until the volumes are equal. One is wetted and molded in the hand and the two volumes then compared. If the wetted volume is obviously smaller, then collapsibility may be suspected (Clemence and Finbarr, 1980).

Field Load Tests

Ground saturation by ponding or using bottomless tanks is useful for evaluating collapse and subsidence where very large leakage may occur, as through canal linings.

Full-scale or plate load tests, for the evaluation of foundation settlements, should be performed under three conditions to provide comparative data at founding level:

1. Soils at natural water content when loads are applied.
2. Loads applied while soil is at the natural water content until the anticipated foundation pressure is reached. The ground around and beneath the footing is then wetted by pouring water into auger holes. (In very dry climates and clayey soils, several days of treatment may be required to achieve an adequate level of

saturation. In all cases, natural soil densities and water contents should be measured before and after wetting).

3. Soils at the test plot are wetted prior to any loading, and then loads are applied.

Laboratory Testing

Natural Density

When the natural density of fine-grained soils at the natural moisture content is lower than normal, about 87 pcf or less, collapse susceptibility should be suspected.

Density vs. liquid limit as a collapse criterion is given by Zur and Wiseman (1973) as follows:

$D_o / D_{LL} < 1.1$, soil prone to collapse

$D_o / D_{LL} > 1.3$, soil prone to swell

where D_0 is the *in situ* dry density and D_{LL} the dry density of soil at full saturation and at moisture content equal to the liquid limit.

Normal Consolidation Test

Loads are applied to the specimen maintained at its natural water content: the specimen is wetted at the proposed foundation stress, and compression is measured as shown in Figure 2.30.

Double Consolidation Test

The double consolidation test is used to provide data for the calculation of estimates of collapse magnitude (Jennings and Knight, 1975; Clemence and Finbarr, 1980).

Two specimens of similar materials (preferably from block samples) are trimmed into consolidometer rings and placed in the apparatus under a light 0.01 tsf seating load for 24 h. Subsequently, one specimen is submerged and the other kept at its natural water content for an additional 24 h. Load applications on each specimen are then carried out in the normal manner.

The e–log p curve for each test is plotted on the same graph, along with the overburden pressure p_o and the preconsolidation pressure p_c for the saturated specimen. The curves for a normally consolidated soil ($p_c/p_o = 0.8 - 1.5$) are given in Figure 2.32, and for an overconsolidated soil ($p_c/p_o > 1.5$) in Figure 2.33. The curve for the natural water content condition is relocated to the point (e_o, p_o) given in the figures.

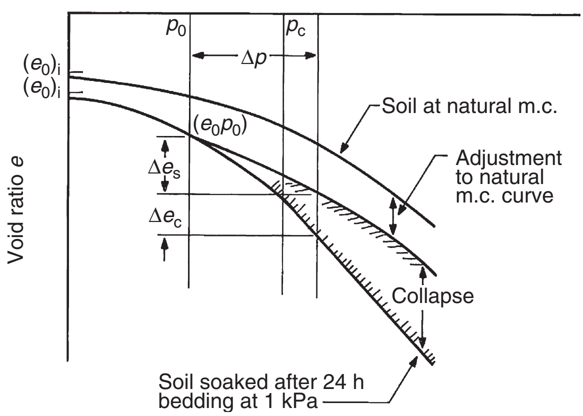


FIGURE 2.32
Double consolidation test *e–p* curves and adjustments for a normally consolidated soil. (From Clemence, S. P. and Finbarr, A. O., *Proceedings of ASCE,* Preprint 80–116, 1980, 22 pp. With permission.)

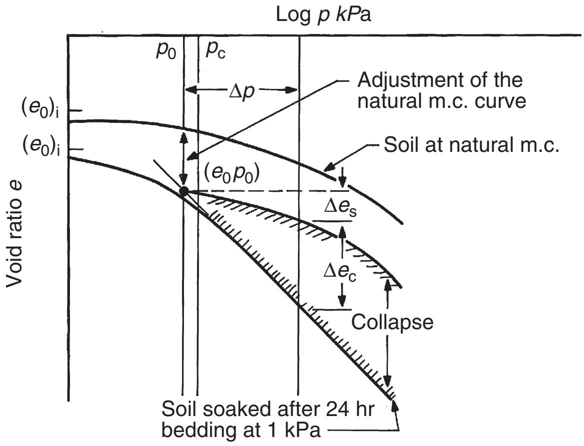


FIGURE 2.33
Double consolidation test *e–p* curves and adjustments for an overconsolidated soil. (From Clemence, S. P. and Finbarr, A. O., *Proceedings of ASCE,* Preprint 80–116, 1980, 22 pp. With permission.)

For an increase in foundation stress ΔP, and wetted soil, the settlement ρ is estimated from the expression

$$\rho = (\Delta e_s / 1 + e_o) + \Delta e_c / (1 + e_o) \tag{2.1}$$

2.5.4 Treatment and Support of Structures

Evaluate the Degree of Hazard and Risk

Degree of hazard is basically a function of the probability of significant ground wetting and of the magnitude of the potential collapse, if the critical pressure that will cause collapse at the natural water content is not approached. Sources of ground wetting have been given in Section 2.5.2 in the discussion of collapse causes.

- *Low-hazard* conditions exist, where potential collapse magnitudes are small and tolerable, or the probability for significant ground wetting is low.
- *Moderate-hazard* conditions exist, where the potential collapse magnitudes are undesirable but the probability of substantial ground wetting is low.
- *High-hazard* conditions exist, where the potential collapse magnitudes are undesirable and the probability of occurrence is high.

Degree of risk relates to the sensitivity of the structure to settlement and to the importance of the structure.

Reduce the Hazard

Prevent ground wetting and support structures on shallow foundations designed for an allowable bearing value sufficiently below the critical pressure to avoid collapse at natural water content. The critical pressure is best determined by *in situ* plate-load tests, and the allowable soil pressure is based on FS = 2 to 3, depending upon the settlement tolerances of the structure.

In some cases, such as large grain elevators, where the load and required size of a mat foundation impose bearing pressures of the order of the critical pressure, the structures have been permitted to settle as much as 1 ft (30 cm), provided that tilting is avoided. This solution has been applied to foundations on relatively uniform deposits of loess with natural water contents of the order of 13% in eastern Colorado and western Kansas. In collapsible soils derived from residual soils, however, such solutions may not be applicable because of the likelihood that a variation in properties will result in large differential settlements. Adequate site drainage should be provided to prevent ponding and all runoff should be collected and directed away from the structure. Avoid locating septic tanks and leaching fields near the structure, and construct all utilities and storm drains carefully to ensure tightness. Underground water lines near the structures have been double-piped or encased in concrete to assure protection against exfiltration. In a desert environment, watering lawns, which is commonly accomplished by flooding, should be avoided, and areas should be landscaped with natural desert vegetation.

Lime stabilization has been used in Tucson, Arizona, to treat collapsible soils that have caused detrimental settlements in a housing development (Sultan, 1969). A water-lime mixture was pumped under high pressure into 2-in.-diameter holes to depths of about 5 ft, and significant movements were arrested.

Hydrocompaction to preconsolidate the collapsible soils was the solution used by the California Department of Water Resources for the construction of the California aqueduct (Curtin, 1973). Dikes and unlined ditches were constructed and flooded along the canal route to precompact the soils at locations where collapse potential was considered high. A section of the canal being constructed over areas both precompacted and not precompacted is shown in Figure 2.34; the subsidence effects on the canal sides are evident on the photo. When hydrocompaction is used, however, the possibility of long-term settlements from the consolidation of clay soils under increased overburden pressures should be considered.

Vibroflotation was experimented with before the construction of the California aqueduct, but adequate compaction was not obtained in the fine-grained soils along the alignment.

FIGURE 2.34
Mendoza test plot showing prototype canal section along the California aqueduct. Crest width is 168ft and length is 1400ft. Note that both the lined and unlined sections of the canal are subsiding where the land was not precompacted. Concentric subsidence cracks indicate former locations of large test ponds. (From Curtin, G., *Geology, Seismicity and Environmental Impact, Special Publication* Association Engineering, Geology, University Publishers, Los Angles, 1973. With permission. Photo courtesy of California Department of Water Resources.)

Dynamic compaction involves dropping 8- to 10-ton tamping blocks from heights of 30 to 120 ft (9 to 36 m). The drops, made by a crane in carefully regulated patterns, produce high-energy shock waves that have compacted soils to depths as great as 60 ft (18 m) (ENR, 1980).

Avoid the Hazard

Settlement-sensitive structures may be supported on deep foundations that extend beyond the zone of potential collapse or, if the collapsible soils extend to limited depths, shallow foundations may be established on controlled compacted earth fill after the collapsible soils are excavated.

In Brazil, buildings constructed over collapsible residual soils are normally supported on piers or piles penetrating to the hard stratum at depths of 3 to 5 m. Floors for large industrial buildings are often supported on porous clays or small amounts of fill, and protection against ground wetting is provided. The risk of subsidence is accepted with the understanding that some relevelling by "mudjacking" may be required in the future.

2.5.5 Piping Soils and Dispersive Clays

General

Soils susceptible to piping erosion and dispersion are not a cause of large-scale subsidence. Ground collapse can occur, however, when the channels resulting from piping and dispersion grow to significant dimensions.

Piping Phenomena

Piping refers to the erosion of soils caused by groundwater flow when the flow emerges on a free face and carries particles of soil with it.

Occurrence in natural deposits results from water entering from the surface, flowing through the soil mass along pervious zones or other openings, finally to exit through the face of a stream bank or other steep slope as illustrated in Figure 2.35. As the water flows, the opening increases in size, at times reaching very large dimensions as shown in Figure 2.36,

FIGURE 2.35
Piping erosion in road cut, Tucson, Arizona. (1972 photo courtesy of Robert S. Woolworth.)

FIGURE 2.36
Tunnel about 7 m high formed from piping in colluvial–lacustrine clayey silts, near La Paz, Bolivia. The vertical slopes are about 15 m in height.

forming in the remains of the enormous ancient mudflow shown in Figure 1.55. The massive movement of the flow destroyed the original structure of the formation and it came to rest in a loose, remolded condition in which fissures subsequently developed. Rainwater entering surface cracks passes along the relict fissures and erodes their sides.

Dispersive Clays

Occurrence

Erosion tunnels from piping in earth dams constructed with certain clay soils are a relatively common occurrence that can seriously affect the stability of the embankment (Sherard et al., 1972). It was originally believed that the clay soils susceptible to dispersion erosion were limited to dry climates, but in recent years these soils and their related problems have been found to exist in humid climates. Sherard et al. (1972) cite examples in the United States from Oklahoma and Mississippi and from western Venezuela.

The Phenomenon

Dispersive clays erode in the presence of water by dispersion or deflocculation. In certain clay soils in which the electrochemical bond is weak, contact with water causes individual particles to detach or disperse. Flowing fresh water readily transports the dispersed particles in the form of piping erosion, in time creating voids or tunnels in the clay mass as illustrated in Figure 2.37 and Figure 2.38. Any fissure or crack from desiccation or settlement can provide the initial flow channel. In expansive clays, the cracks will close by swelling and dispersion will not occur. The failure of many small dams and dikes has been attributed to dispersive clays.

Soil Susceptibility

The main property governing susceptibility to dispersion appears to be the quantity of dissolved sodium cations in the pore water relative to the quantities of other main cations

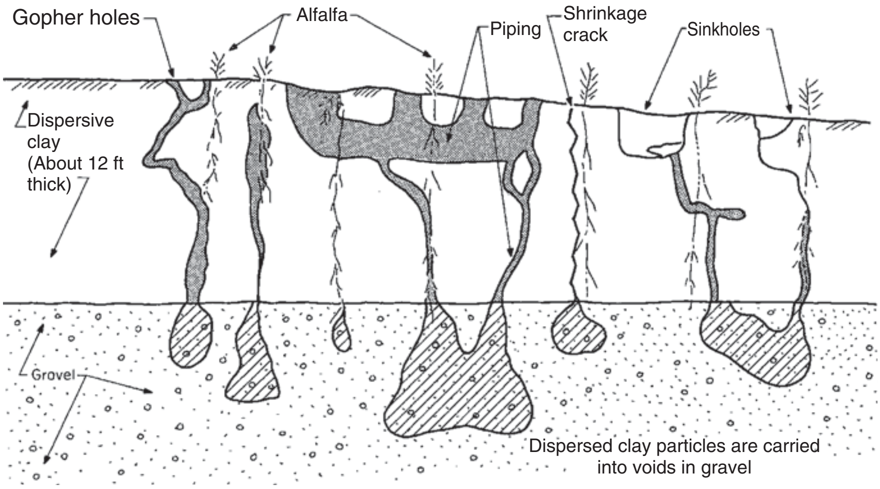


FIGURE 2.37
Damage to agricultural fields in Arizona from piping and sinkhole formation in dispersive clays. (From Sherard, J. L. et al., *Proceedings ASCE*, Purdue University, Vol. I, 1972. With permission.)

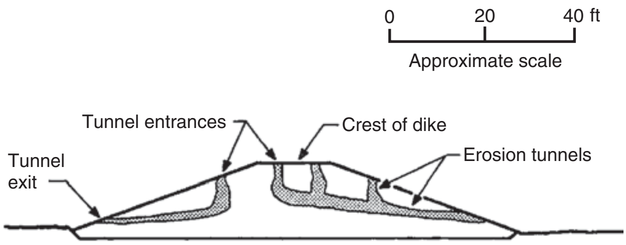


FIGURE 2.38
Schematic of typical rainfall erosion tunnels in clay flood-control dike in badly damaged section. (From Sherard, J. L. et al., *Proceedings ASCE*, Purdue University, Vol. I, 1972. With permission.)

(calcium and magnesium), i.e., the higher the percentage of sodium cation, the higher the susceptibility to dispersion. Soil scientists refer to this relationship as "exchangeable sodium percentages" (ESP).

As of 1976, there appeared to be no good relationship between the ESP and the index tests used by the geotechnical engineer to classify soils, except for the identification of expansive clays that are nonsusceptible. Highly dispersive clays frequently have the same Atterberg limits, gradation, and compaction characteristics as nondispersive clays. These clays plot above the *A* line on the plasticity chart and are generally of low to medium plasticity (LL = 30–50%; CL-CH classifications), although cases of dispersion have been reported in clay with a high liquid limit.

The pinhole test is a simple laboratory test developed to identify dispersive clays (Sherard et al., 1976).

Prevention of Piping and Dispersion

Piping in natural deposits is difficult to prevent. The formation of large tunnels is rare and limited to loose, slightly cohesive silty soils where they are exposed in banks and steep slopes and seepage can exit from the slope. A practical preventive measure is to place a filter at the erosion tunnel outlet to reduce flow exit velocities, but in some cases such as shown in Figure 2.36, the area should be avoided since tunnels are likely to open at other locations.

Piping in dispersive clays used in embankment construction is prevented by the proper design of filters to control internal seepage and by the use of materials that are not susceptible to the phenomenon. Where piping is already occurring it may be necessary to reconstruct the embankment using proper design and materials, if placing a filter at the outlet is not effective.

2.6 Heave in Soil and Rock

2.6.1 General

Origins of Ground Heave

Heave on a regional basis occurs from tectonic activity (see Section 3.3.1 and the Appendix). On a local basis heave occurs from stress release in rock excavations, expansion from freezing, and expansion from swelling in soil or rock.

The Swelling Hazard

Swelling in Geologic Materials

Clay soils and certain minerals readily undergo volume change, shrinking when dried, or swelling when wet. When in the dry state, or when less than fully saturated, some clays have a tremendous affinity for moisture, and in some cases may swell to increase their volume by 30% or more. Pressures in excess of 8 tsf can be generated by a swelling material when it is confined. Once the material is permitted to swell, however, the pressures reduce. There is a time delay in the swelling phenomenon. Noticeable swell may not occur for over a year after the completion of construction, depending on the soil's access to moisture, but may continue for 5 years or longer.

Damage to Structures

Ground heave is a serious problem for structures supported on shallow foundations, or deep foundations if they are not isolated from the swelling soils. Heave results in the uplift and cracking of floors and walls, and in severe cases, in the rupture of columns. It also has a detrimental effect on pavements. Shrinkage of soils also causes damage to structures. Damages to structures in the United States each year from swelling soils alone have been reported to amount to $700 million (ENR, 1976).

Geographic Distribution

Swelling Soils

Swelling soils are generally associated with dry climates such as those that exist in Australia, India, Israel, the United States, and many countries of Africa. In the United States, foundation problems are particularly prevalent in Texas, Colorado, and California, and in many areas of residual soils, as shown on the map given in Figure 2.39.

Swelling in Rocks

Swelling in rocks is associated primarily with clay shales and marine shales. In the United States, it is particularly prevalent in California, Colorado, Montana, North and South Dakota, and Texas.

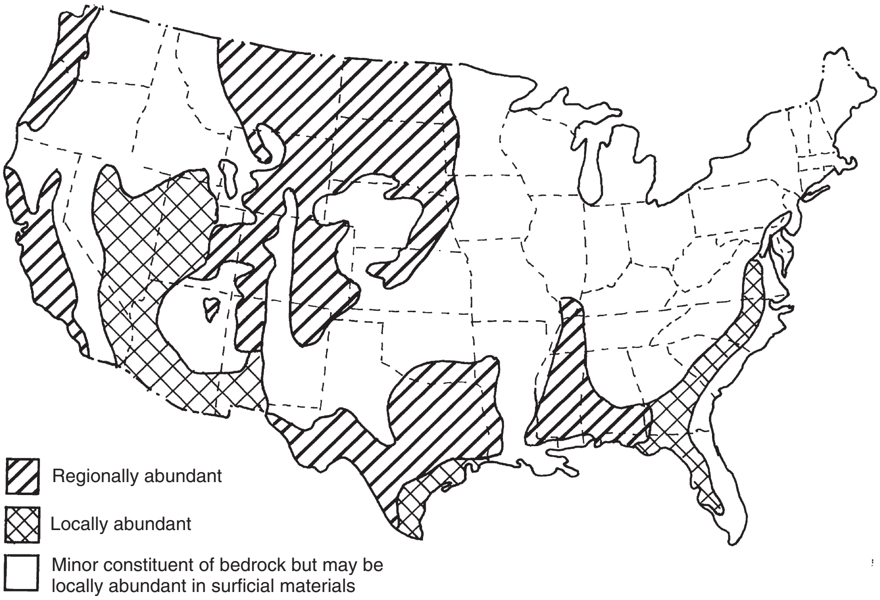


FIGURE 2.39
Distribution of expansive soils in the United States. They are most widespread in areas labeled "regionally abundant," but many locations in these areas will have no expansive soils, while in some unshaded portions of the map, some expansive soils may be found. (From Godfrey, K. A., *Civil Engineering, ASCE,* 1978, pp. 87–91. With permission.)

2.6.2 Swelling in Soils

Determining Swell Potential

Basic Relationships

The phenomenon of adsorption and swell is complex and not well understood, but it appears to be basically physicochemical in origin. Swell potential is related to the percentage of the material in the clay fraction (defined as less than 2 μm, 0.002 mm), the fineness of the clay fraction, the clay structure, and the type of clay mineral. Montmorillonite has the highest potential for swell, followed by illite, with kaolinite being the least active. Thus, mineral identification is one means of investigating swell potential.

Clay Activity

Swell potential has been given by Skempton (1953) in terms of activity defined as the ratio of the plasticity index to the percent finer by weight than 2 μm. On the basis of activity, soils have been classified as inactive, normal, and active. The activity of various types of clay minerals as a function of the plasticity index and the clay fraction is given in Figure 2.40. It is seen that the activity of sodium montmorillonite is many times higher than that of illite or kaolinite.

Prediction from Index Tests

Any surface clay with plasticity index PI $>$ 25 (CH clays) and a relatively low natural moisture content approaching the plastic limit must be considered as having swell potential. The colloid content (percent minus 0.001 mm), the plasticity index, and shrinkage limit are used

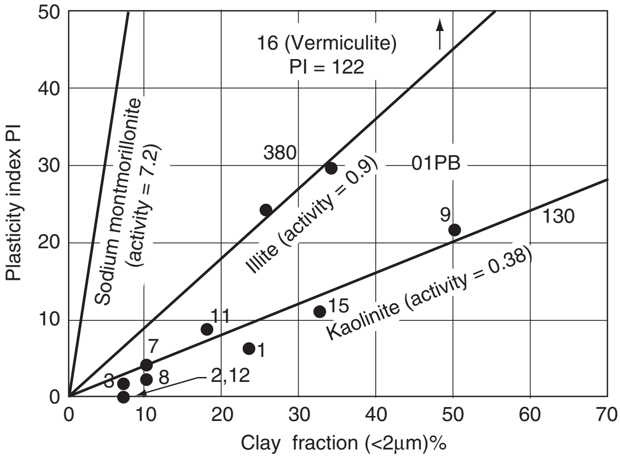


FIGURE 2.40
Clay fraction and plasticity index of natural soils in relation to activity of natural soils in relation to activity chart of Skempton, A. W., *Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering,* Zurich, Vol. I, 1953, pp. 57–61. (After Basu, R. and Arulanandan, K., *Proceedings of the 3rd International Conference on Expensive Soils, Haifa, Israel,* 1973.)

TABLE 2.4
Relation of Soil Index Properties to Probable Volume Changes for Highly Plastic Soils[a]

Data from Index Tests[b]				
Colloid Content (%) <0.001 mm	**Plasticity Index**	**Shrinkage Limit (%)**	**Probable Expansion, Percent Probable Total Volume Change (dry to saturated condition)**	**Degree of Expansion**
>28	>35	<11	>30	Very high
20–31	25–41	7–12	20–30	High
13–23	15–28	10–16	10–20	Medium
<15	<18	>15	<10	Low

[a] From USBR, Earth Manual, U.S. Bureau of Reclamation, Federal Center, Denver, Colorado, 1974. With permission.
[b] All three index tests should be considered in estimating expensive properties.
[c] Based on a vertical loading of 1.0 psi as for concrete canal lining. For higher loadings the amount of expansion is reduced, depending on the load and day characteristics.

by the USBR (1974) as criteria for estimating the probable total volume change from the dry to the saturated condition, as given in Table 2.4.

In the method developed by Seed et al. (1962), expansion was measured as percent swell by placing samples at 100% maximum density and optimum water content in a Standard AASHO compaction mold under a surcharge of 1psi and then soaking them. A family of curves given in Figure 2.41 was developed to describe the percent swell potential for various clay types in terms of activity and clay fraction present. The chart given in Figure 2.42 provides another basis for estimating potential expansiveness.

Index tests have limitations. They do not always identify the swell potential for all natural deposits, nor provide information on the true amount of heave or pressures that may develop. Because the tests are made on remolded specimens, the natural structure of the material is destroyed and other environmental factors are ignored.

Laboratory Tests

Tests in the laboratory on undisturbed samples trimmed into the consolidometer will provide an indication of potential heave and the swelling pressures that may develop.

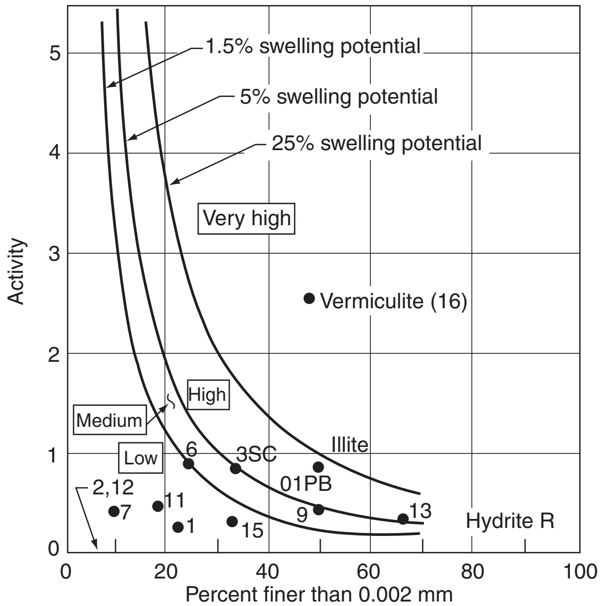
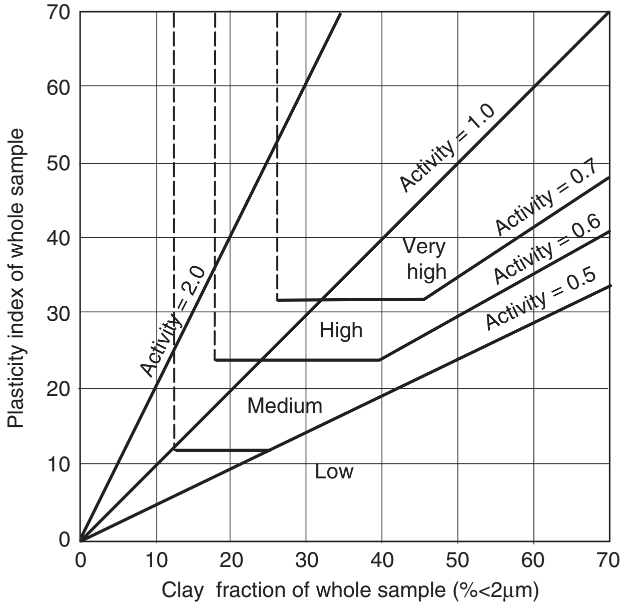


FIGURE 2.41
Classification chart for swelling potential. (After Seed, H. B. et al., *Proc. ASCE J. Soil Mech. Found. Eng. Div.*, 88, 1962. With permission. From Basu, R. and Arulanandan, K., *Proceedings of the 3rd International Conference on Expensive Soils, Haifa, Israel*, 1973.)



Potential expansiveness	Inch per foot of soil
Very high	1.0
High	0.5
Medium	0.25
Low	0

FIGURE 2.42
Proposed modified chart for determining expansiveness of soils. (From Williams, A. A. and Donaldson, G. W., *Proceedings of the 4th International Conference on Expansive Soils*, Denver, Colorado, Vol. II, 1980, pp. 834–844. With permission.) The Table is after Van der Merwe, U. H., *Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, Durban, Vol. 2, 1975, pp. 166–167.

Environmental Factors

Basic Factors

Water-table depth: For a soil to develop substantial heave it must lie above the static water table in a less than saturated state and have moisture available to it. Moisture can originate from capillary action or condensation as well as in the form of free water.

Climate: Highly expansive soils are found in climates from hot to cold. Long periods with little or no rainfall permit the water table to drop and the soils to decrease in moisture. The soils dry and shrink, large cracks open on the surface, and fissures develop throughout the mass, substantially increasing its permeability. In Texas, during the dry season, these cracks can extend to depths of 20 ft. Rainfall or other moisture then has easy access for infiltration to cause swelling.

Topography affects runoff and infiltration. Poorly drained sites have a higher potential for ground heave than slopes.

Environmental Changes Cause Surface Movements

Decrease in moisture, causing shrinkage and fissuring, results from:

- Prolonged dry spells or groundwater pumping producing a drop in the water table.
- Growth of trees and other vegetation producing moisture loss by transpiration.
- Heat from structures such as furnaces, boiler plants, etc., producing drying.

Increase in moisture, causing swelling and heave, results from:

- Rainfall and a rise in the water table.
- Drilling holes, such as for pile foundations, through a perched water table that permits permeation into a lower clay stratum (ENR, 1969).
- Retarding evaporation by covering the ground with a structure or a pavement.
- Thermo-osmosis, or the phenomenon by which moisture migrates from a warm zone outside a building area to the cool zone beneath the building.
- Condensation from water lines, sewers, storm drains, and canals.
- Removal of vegetation that increases susceptibility to fissuring and provides access for water.

Time Factor

Usually, a year or more passes after construction is completed before the effects of heave are apparent, although heave can occur within a few hours when the soil has sudden access to free water as from a broken water main or a clogged drain.

A plot of yearly rainfall and heave as a function of time for a house in the Orange Free State in South Africa is given in Figure 2.43. It shows almost no heave for the first year, then a heave of 11.6 cm occurring over a period of 4 years, after which movement essentially stops. The long-term effect is the result of the slow increase, due to natural events, of moisture content beneath a covered area.

2.6.3 Swelling in Rock Masses

Marine and Clay Shale

Characteristics

Montmorillonite is a common constituent of marine and clay shale; therefore, these shales have a high swell potential. They are commonly found disintegrated and badly

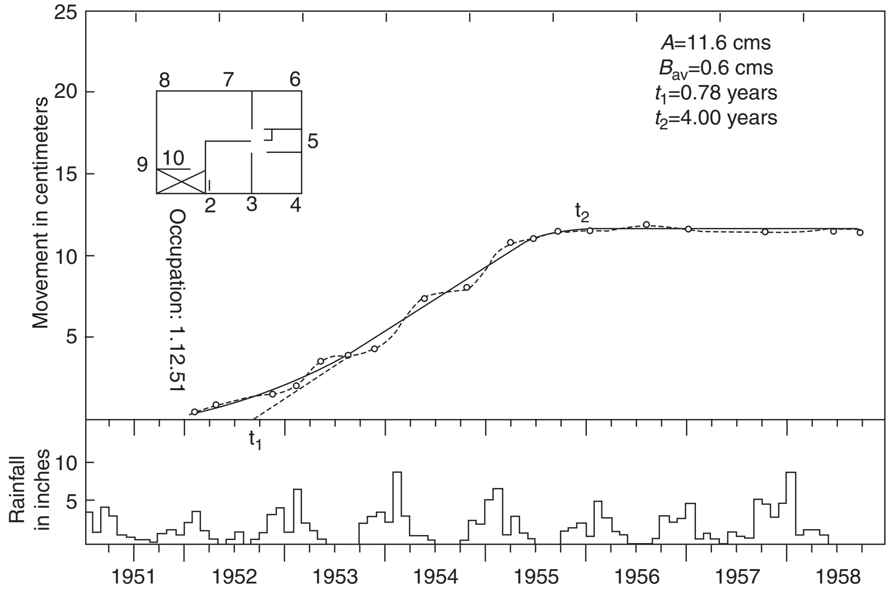


FIGURE 2.43
Typical heave record of single-story brick house in Orange Free State gold fields. (From Jennings, J. E., *Trans. South African Inst. Civil Eng.*, 1969. With permission.)

broken and microfissured from the weathering and expansion of clay minerals. As a result, their shallower portions consist of a mass of hard fragments in a soil matrix. The hard fragments can vary from a medium-hard rock to a very hard indurated clay. Where the weathering is primarily mechanical from the swelling of the clay minerals the disintegrated shale can be found extending downward nearly from the surface, or where chemical weathering has dominated it can be found under a residue of highly expansive clay.

Classification of these materials as either a residual soil or a weathered rock is difficult because of their properties and characteristics. Clay and marine shale are often described as soils in engineering articles and as rock in most geologic publications. They are truly transitional materials, even referred to at times as claystone.

Example: Menlo Park, California

Project: A large urban development suffered substantial and costly damage from the expansion of clay shale (Meehan et al., 1975).

Geologic conditions: Prior to development, the area was virgin hilly terrain underlain by interbedded sandstone and claystone dipping typically at 40 to 60°. In the lower elevations, the claystones are overlain by 3 to 10 ft of black clayey residual or colluvial soils. The weathered portions of the claystone are yellow to olive brown in color, becoming olive gray in fresh rock. The fresh claystone, a soft rock, has two major joint systems, the master system spaced at about 3 ft and the secondary system from 1 to 3 in. Groundwater is generally about 30 ft below the surface. Typical claystone properties are:

- Index properties: LL = 70%, PI = 20, Gs = 2.86, γ_t = 135 pcf
- Clay mineral: Montmorillonite
- Swell pressures: Measured in the consolidometer, 4 to 9 tsf

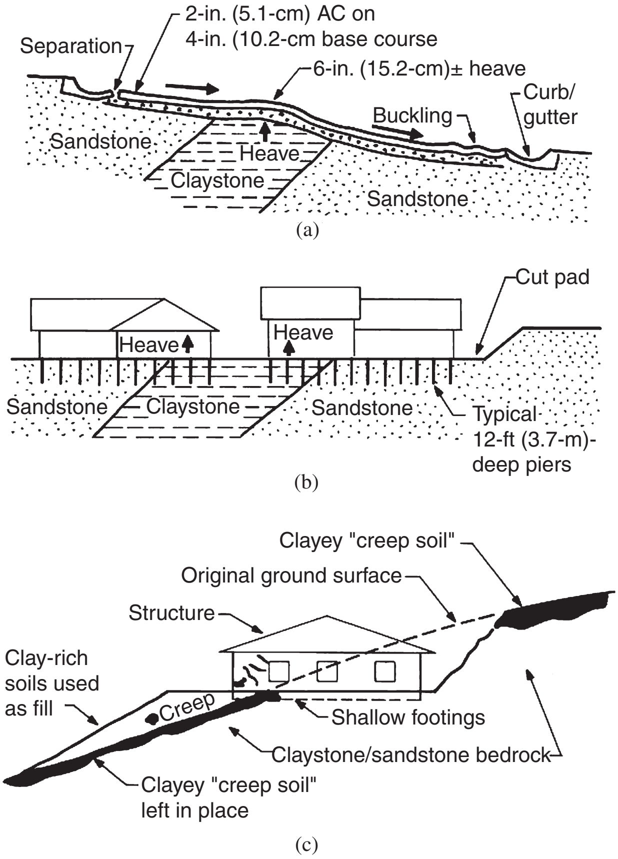


FIGURE 2.44
Problems of heave and creep in the interbedded claystones and sandstones at Menlo Park, California: (a) typical pavement damage; (b) damage to houses on shallow piers; (c) conditions resulting in creep damage. (From Meehan, R. L. et al., *Proc. ASCE J. Geotech. Eng. Div.,* 101, 1975. With permission.)

The problems: Damage from heave to houses and street pavements had been severe, with movements of the order of 4 to 6 in. The major problem was that of differential movements because of the alternating surface exposure of the dipping beds of sandstone and claystone as illustrated in Figure 2.44a and b. Claystone swelling where structures have been placed in cuts typically occurs over several years following construction. It may not be observed for 1 or 2 years, but generally continues to be active for 5 to 7 years after construction. Slopes are unstable, even where gentle. In dry weather the heavy black clays exhibit shrinkage cracks a few feet or so deep. During the winter rains, the cracks close and the clay becomes subject to downhill creep at about 1/2 in. (1.2 cm) per year. Downhill movement of fills has occurred, causing severe distress in structures located partly in cut and partly on fill as shown in Figure 2.44c. Attempted and successful solutions are described in Section 2.6.4.

Other Rocks

Pyritic Shales

In Kansas City, the Missouri limestone has been quarried underground and the space left from the operations has been used since 1955 for warehousing, manufacturing, office, and laboratory facilities. Underlying the limestone and exposed in the floors of many facilities is a black, pyritic shale of Upper Pennsylvanian age. Sulfide alteration of the pyrite results in swelling, and in subsequent years as much as 3 to 4 in. (8 to 11 cm) of floor heave occurred, causing severe floor cracking as well as cracking of the mine pillars left to provide roof support (Coveney and Parizek, 1977). Possible solutions are discussed in Section 2.6.4.

Heaving from the swelling of a black, pyritic carbonaceous shale is reported to have caused damage to structures in Ottawa, Canada (Grattan-Bellew and Eden, 1975). In some instances, the concrete of floors placed in direct contact with the shale has turned to "mush" over a period of years. Apparently, the pyrite oxidizes to produce sulfuric acid, which reacts with calcite in the shale to produce gypsum, the growth of which results in the heave. The acid builds up in the shale to lower the pH to an observed value of 3, leaching the cement from the concrete. The phenomenon does not appear to affect the more deeply embedded footing foundations.

Gneiss and other metamorphic rocks may contain seams of montmorillonite that can be troublesome to deep foundations, tunnels, and slopes.

2.6.4 Treatments to Prevent or Minimize Swelling and Heave

Foundations

Excavations

Sections as small as practical should be opened in shales, and water infiltration prevented. The opening should be covered immediately with foundation concrete, cyclopic concrete, or compacted earth. The objective is to minimize the exposure of the shales to weathering, which occurs very rapidly, and is especially important for dams and other large excavations.

Deep foundations, which generally are drilled piers extending below the permanent zone of saturation, eliminate the heave hazard. The piers, grade beams, and floors must be protected against uplift from swelling forces.

Shallow rigid mat or "rigid" interconnected continuous footings may undergo heave as a unit, but they provide protection against differential movements when adequately stiff.

Other methods, often unsatisfactory, include:

- Preflooding to permit expansion, then designing to contend with settlements of the softened material and attempting to maintain a balance between swell and consolidation.
- Injection with lime has met with some success, but in highly active materials it may aggravate swelling, since the lime is added with water.
- Excavation of the upper portion of the swelling clays, mixing that portion with lime, which substantially reduces their activity, and then replacing the soil–lime mixture as a compacted fill is a costly, and not always satisfactory, procedure.

Floors

Floors should consist of a structural slab not in contact with the expansive materials or supported on a free-draining gravel bed that permits breathing. Infiltration of water must be prevented.

Pavements

Foundations for pavements are prepared by excavating soil to some depth, which depends on clay activity and environmental conditions, and replacing the materials with clean granular soils, with the same soil compacted on the wet side of optimum, or with the same

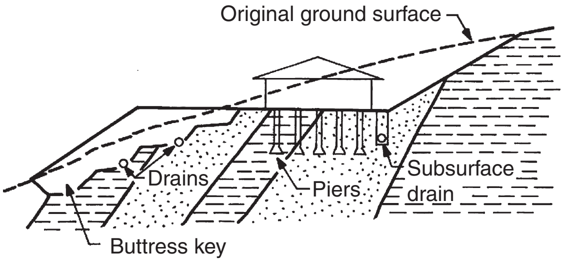


FIGURE 2.45
Site grading and foundation solutions at Menlo Park, California. (From Meehan, R. L. et al., *Proc. ASCE J. Geotech. Eng. Div.*, 101, 1975. With permission.)

soil mixed with lime. Protection against surface water infiltration is provided at the pavement edges, adequate underdrainage is provided, and pavement cracks are sealed.

Solutions at Menlo Park

Site Grading

The procedures cited in Meehan et al. (1975) to correct the problems described in Section 2.6.3 called for the stripping of the shallow, surface soils, and keying fills into slopes in a series of steps. Subsurface drains were provided to prevent water migration into the expansive weathered rock as illustrated in Figure 2.45.

Foundation support was provided by drilled piers taken to depths of 20 to 30 ft, or to the sandstone if shallower — a satisfactory but costly solution for homes.

Lime injected into closely spaced holes was not successful in reducing heave, and at times the situation was aggravated because the lime was added with water.

Pavements were treated by ripping the subgrade to a depth of about 5 ft, injecting a lime slurry into the loosened claystone, then recompacting the surface before paving or repaving. New streets were constructed with “full-depth” asphaltic concrete with typical thicknesses of 5 in. laid directly on the subgrade. When Meehan et al. (1975) published their paper, the success history of the treatments for pavements was about 2 years.

Possible Solutions for Kansas City

For the problems in the pyritic shales in Kansas City (see Section 2.6.3), several possible solutions appear feasible:

- Immediate coating of the shale after excavation with bitumen or a comparable airtight substance.
- Removal of the shales beneath the floor areas and replacement with concrete to some moderately substantial depth, but with consideration given to the phenomenon reported for the shales in Ottawa (see Section 2.6.3).
- Bypass the old mine pillars as roof support with concrete supports founded beyond the active zone of the shale.

References

Allen, D. R., Subsidence, Rebound and Surface Strain Associated with Oil Producing Operations, Long Beach, California, *Geology, Seismicity and Environmental Impact*, Special Publishing, Association of Engineering Geology, University Publishers, Los Angeles, 1973.

Allen, D. R. and Mayuga, M. N., The Mechanics of Compaction and Rebound, Wilmington Oil Field, Long Beach, CA, *Land Subsidence*, Vol. II, Pub. No. 89, IASH-UNESCO-WHO, 1969, pp. 410–423.

Amould, M., Problems Associated with Underground Caverns in the Paris Region, *Proc. Annual Mtg., Assoc. Engrg. Geol*., 1970.

Averitt, P., Coal Resources of the United States, January, 1967, *USGS Bull. 1275*, U.S. Govt. Printing Office, Washington, DC, 1967.

Barden, L., McCown, A., and Collins, K., The collapse mechanism in partially saturated soil, *Eng. Geol.*, 49–60, 1973.

Basu, R. and Arulanandan, K., A New Approach to the Identification of Swell Potential for Soils, *Procedings of the 3rd International Conference on Expansive Soils*, Haifa, Israel, 1973.

Bau, D., Gambolati, G., and Teatini, P., Residual Land Subsidence Over Depleted Gas Fields in the Northern Adaiatic Basin, *Environmental and Engineering Science*, Vol. V, 1999, pp. 389–405, Joint Pub., AEG and GSA.

Bell, J. W., Amelung, F., Ramelli, A. R., and Blewitt, G., Land Subsidence in Las Vegas, Nevada, 1935–2000. New Geodetic Data Show Evolution, Revised Spatial Patterns, and Reduced Rates, *Environmental and Engineering Science*, Vol. VIII, 2002, pp. 389–405, Joint Pub, AEG and GSA.

Bell, J. W., Price, J. G., and Mifflin, M. D., Subsidence-induced fissuring along preexisting faults in Las Vegas Valley, NV, *Proceedings. AEG*, 35th Annual Mtg., Los Angeles, 1992, pp. 66–75.

Brink, A. B. A. and Kantey, B. A., Collapsible Grain Structure in Residual Granite Soils in Southern Africa, *Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering*, Paris, Vol. 1, 1961, pp. 611–614.

Bull, W. B., Alluvial Fans and Near-surface Subsidence in Western Fresno County, CA, Geological Survey Prof. Paper 437-A, Washington, DC, 1964.

Castle, R. O. and Youd, T. L., Discussion: the Houston fault problem, *Bull. Assoc. Engrg. Geol*. 9, 1972.

Civil Engineering, Venice Is Actually Rising? ASCE, November 1975, p. 81.

Civil Engineering, Houston Area Tackles Subsidence, ASCE, March 1977, p. 65.

Civil Engineering, Ground Surface Subsidence Is a World-Wide Problem, ASCE, June 1978, p. 18.

Cohen, K. K., Trevits, M. A., and Shea, T. K., Abandoned Metal Mine Reclamation in New Jersey Applies Geophysics to Target Subsurface Hazards for Safety, Remediation and Land Use AEG National Mtg., Abstracts, 1996.

Corbel, J., Erosion en terrain calcaire, *Ann. Geog., 68*, 97–120, 1959.

Coveney, R. M. and Parizek, E. J., Deformation of mine floors by sulfide alteration, *Bull. Assoc. Engrg. Geol*., 14, 131–156, 1977.

Clemence, S. P. and Finbarr, A. O., Design Considerations and Evaluation Methods for Collapsible Soils, *Proc. ASCE*, Preprint 80-116, April 1980, 22 pp.

Croxton, N. M., Subsidence on I-70 in Russell County, Kansas related to salt dissolution — a history, *The Professional Geologist*, 2002, pp. 2–7.

Curtin, G., Collapsing Soil and Subsidence, *Geology, Seismicity and Environmental Impact*, Spec. Pub., Assoc. Eng. Geol., University Publishers, Los Angeles, 1973.

Davidson, E. S., Geohydrology and Water Resources of the Tucson Basin, Arizona, Open File Report, Geological Survey, U.S. Dept. Of Interior, Tucson, Arizona, May 1970.

Davis, G. H., Small, J. B., and Counts, H. B., Land Subsidence Related to Decline of Artesian Pressure in the Ocala Limestone at Savannah, Georgia, Engineering Geology Case Histories No. 4, The Geological Society of America, Engineering Geology Div., New York, 1964, pp. 185–192.

ENR, Sinkhole Causes Roof Failure, *Engineering News-Record*, Dec. 11, 1969, p. 23.

ENR, Expansive Soils Culprit in Cracking, *Engineering News-Record*, Nov. 4, 1976, p. 21.

ENR, Houston Land Subsidence Sinks 1970 Bridge Specs, *Engineering News-Record*, June 9, 1977a, p. 11.

ENR, USGS Bets on Las Vegas: Sinking Is No Emergency, *Engineering News-Record*, Dec. 1, 1977b, p. 17.

ENR, Limestone Cavity Sinks Garage, *Engineering News-Record*, Feb. 9, 1978, p. 11.

ENR, Groundwater Bottled Up to Check Moving Piles, *Engineering News-Record*, Oct. 18, 1979, p. 26.

ENR, Flood Control Plan Recycles Water, *Engineering News-Record*, April 24, 1980, pp. 28-29.

ENR, Reclaimed Site Pays Off, *Engineering News-Record*, Jan. 3, 1980, p.14.

Farmer, I. W., *Engineering Properties of Rocks*, E. & F. N. Spon, London, 1968.

Flint, R. F., Longwell, C. R., and Sanders, J. E., *Physical Geology*, John Wiley & Sons, New York, 1969.

Foose, R. M., Groundwater behavior in the Hershey Valley, *Bull. Geol. Soc. Amer*., 64, 1953.

Godfrey, K. A., Expansive and Shrinking Soils-Building Design Problems Being Attacked, *Civil Engineering*, ASCE, October 1978, pp. 87–91.

Gowan, S.W. and Trader, S.M., Mine Failure Associated with a Pressurized Brine Horizon: Retsof Salt Mine, Western New York, *Environmental and Engineering Science*, 6, 2000, pp. 57–70.

Grattan-Bellew, P. E. and Eden, W. J., Concrete deterioration and floor heave due to biogeochemical weathering of underlying shale, *Canadian Geotech. J*., 12, 372–378, 1975, National Research Council of Canada, Ottawa.

Gray, R. E. and Meyers, J. F., Mine subsidence and support methods in the Pittsburgh area, *Proc. ASCE J. Soil Mech. Found. Eng. Div*., 96, 1970.

Greewald, H. P., Howarth, H. C., and Hartmann, I., Experiments on Strength of Small Pillars of Coal in the Pittsburgh Bed, U.S. Bureau of Mines, Report of Investigations 3575, June 1941.

Griffith, W. and Conner, E. T., Mining Conditions Under the City of Scranton, Pennsylvania, *Bull. 25,* Bureau of Mines, U.S. Dept. of Interior, Washington, DC, 1912.

Holdahl, S. R., Zilkoski, D. B., and Holzschuh, J. C., Subsidence in Houston, Texas, 1973–1987, *Procedings of the 4th International Symposium on Land Subsidence*, Johnson, A.I., Ed., IAHS Publ. No. 200, E. 1991.

Hunt, R. E., Round rock, Texas New Town: geologic problems and engineering solutions, *Bull. Assoc. Engrg. Geol.,* 10, 1973.

Hunt, R. E. and Santiago, W. B., A fundo critica do engenheiro geologo em estudos de implantaclo de ferrovias, *Proc. 1, Cong. Brasileiro de Geologia de Engenharia*, Rio de Janeiro, August, Vol. 1, 1976, pp. 79–98.

Jansen, R. B., Dukleth, G. W., Gordon, B. B., James, L. B., and Shields, C. E., Earth Movement at Baldwin Hills Reservoir, *Proc. ASCE J. Soil Mechs. and Found. Eng. Div*., 93, 1976; Jansen, R.B., Dukleth, G. W., Gordon, B. B., James, L. B., and Shields, C. E., Subsidence-A Geological Problem with a Political Solution, *Civil Engineering* ASCE, 1980, pp. 60–63.

Jennings, J. E., Building on Dolomites in the Transvaal, *Trans. South African Inst. of Civ. Eng*., January 1966.

Jennings, J. E., The Engineering Problems of Expansive Soils, *Proceedings of the 2nd International Research and Engineering Conference on Expansive Soils*, Texas A&M Univ., College Station, Texas, 1969.

Jennings, J. E. and Knight, K., A Guide to Construction on or with Materials Exhibiting Additional Settlement Due to 'Collapse' of Grain Structure, *6th Regional Conference for Africa on Soil Mechs. Found. Engrg*., September 1975, pp. 99–105.

Ko, H. Y. and Gerstle, K. H., Constitutive Relations of Coal, *New Horizons in Rock Mechanics*, Proceedings ASCE, 14th Symposium Rock Mechanics, Univ. Park, PA. June 1972, 1973, pp. 157–188.

Leake, S. A., Land Subsidence from Groundwater Pumping USGS, 2004, from the Internet

Legget, R. F., Duisburg Harbour Lowered by Coal Mining, *Canadian Geotech. J*., 9, 1972.

Mabry, R. E., An Evaluation of Mine Subsidence Potential, *New Horizons in Rock Mechanics, Proceedings ASCE, 14th Symposium on Rock Mechanics.,* Univ. Park, PA, June 1972, 1973.

Mansur, C. I. and Skouby, M. C., Mine grouting to control building settlement, *Proc. ASCE, J. Soil Mechs. Found. Engrg. Div*., 96, 1970.

Meehan, R. L., Dukes, M. T. and Shires, P. O. A case history of expansive clay stone damage, *Proc. ASCE J. Geotech. Engrg. Div*. 101, 1975.

Millet, R. A. and Moorhouse, D. C., Bedrock Verification Program for Davis-Besse Nuclear Power Station, *Proc. ASCE, Special Conference Structural Design Nuclear Power Plant Facilities*, Chicago, December 1973, pp. 89–113.

MSHA, Report Released on Collapse, Flooding of New York Salt Mine, News Release No. 95-032, Mine Safety and Health Admin., Aug. 8, 1995.

NCB Principles of Subsidence Engineering, Information Bull. 63/240, National Coal Board of the United Kingdom, Production Department, 1963.

NCB *Subsidence Engineers Handbook*, National Coal Board of the United Kingdom, Production Department, 1966.

PaDEP, Final Report, Effects of Longwall Mining on Real Property Value and the Tax Base of Green and Washington Counties, Pennsylvania, Commonwealth of PA, Department of Environmental Protection, Bur. Of Mining and Reclamation, Contract No. BMR-00-02, Dec. 13, 2002.

Penn. Geol. Survey, *Engineering Characteristics of the Rocks of Pennsylvania*, Pennsylvania Geological Survey, 1972.

Pierce, H. W., Geologic Hazards and Land-Use Planning, *Earth Science and Mineral Resources in Arizona*, Vol. 2, No. 3, September, Ariz. Bur. Mines, 1972.

Proctor, R. J., Geology and Urban Tunnels-Including a Case History of Los Angeles, *Geology, Seismicity and Environmental Impact*, Spec. Pub. Assoc. Engrg. Geol., University Publishers, Los Angeles, 1973, pp.187–193.

Prokopovich, N. P., Some geologic factors determining land subsidence, *Bull. Assoc. Engrg. Geol.*, 75–81, 1976.

Rudolph, M., Urban Hazards: Sinking of a Titantic City, *Geotimes*, July 2001, p. 10.

Seed, H. B., Woodward, R. J., and Lundgren, R. Prediction of swelling potential for compacted clays, *Proc. ASCE J. Soil Mechs. Found. Eng. Div.*, 88, 1962.

Sherard, J. L., Decker, R. S., and Ryker, N. L., Piping in Earth Dams of Dispersive Clay, *Performance of Earth and Earth Supported Structures, Proc. ASCE*, Purdue Univ., Vol. I, Part 1, 1972.

Sherard, J. L., Dunnigan, L. P., Decker, R. S., and Steele, E. F. Pinhole test for identifying dispersive soils, *Proc. ASCE J. Geotech.Eng. Div.*, 102, 69–85, 1976.

Skempton, A. W., The Colloidal Activity of Clays, *Proceedings of the 3rd International Conference on Soil Mechanics Foundational Engineering* Zurich, Vol. I, 1953, pp. 57–6l.

Sowers, G. F., Failures in limestones in humid subtropics, *Proc. ASCE J. Geotech. Eng. Div.*, 101, August 1975.

Spencer, G. W., The Fight to Keep Houston from Sinking, *Civil Engineering*, ASCE, September, 1977 pp. 69–71.

Sultan, H. A., Foundation Failures on Collapsing Soils in the Tucson, Arizona Area, *Proceedings of the 2nd International Research and Engineering Conference on Expansive Clay Soils*, Texas A & M Univ., College Station, Texas, 1969.

Terzaghi. K., Landforms and Subsurface Drainage in the Gacka Region in Yugoslavia, reprinted in *Theory to Practice in Soil Mechanics*, Wiley, NY, 1960, 1913.

Testa, S. M., Elevation Changes Associated with Groundwater Withdrawal and Reinjection in the Wilmington Area, Los Angeles Coastal Plain, California, *Proceedings of the 4th International Conference on Land Subsidence*, Houston, TX, May 1991, *IAHS* Pub. No. 200

Trevits, M. A. and Cohen, K. K., Remediation of Problematic Shafts Associated with the Abandoned Schuyler Copper, Mine, North Arlington, NJ, Abstracts, AEG National Mtg., 1996.

Tschebotarioff, G. P., *Foundations, Retaining and Earth Structures*, 2d ed., McGraw-Hill, New York, 1973.

USBR, *Earth Manual*, U.S. Bureau of Reclamation, Federal Center, Denver, CO, 1974.

Van der Merwe, U. H., Contribution to Specialty Session B, Current Theory and Practice for Building on Expansive Clays, *Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, Durban, Vol. 2, 1975, pp. 166–167.

Van Siclen, D. C., The Houston Fault Problem, Amer. Inst. Prof. Geol., Proc. 3rd Annual Meeting Texas Section, Dallas, 1967.

Vargas, M., Fundañao de Barragems de Terra Sobre Solos Porosos, *VIII Seminario Nacional de Grandes Barragems*, São Paulo, Nov. 27, 1972.

Voight, B. and Pariseau, W., State of Predictive Art in Subsidence Engineering, *Proc. ASCE J. Soil Mechs. Found. Eng*. Div. 96, 1970.

White, W. B., Speleology, *Encyclopedia of Geomorphology*, Dowden, Hutchinson & Ross Publ., Stroudsburg, PA, 1968, pp. 1036–1039.

William, K., Geology, Mining History, and Recent Developments at the Akzo-Nobel Salt Mine, Retsof, NY, AIPG Abstracts, Oct. 24, 1995.

Williams, A. A. B. and Donaldson, G. W., Building on Expansive Soils in South Africa: 1973–1980, *Proceedings of 4th International Conference on Expansive Soils*, Denver, CO, Vol. II, 1980, pp. 834–844.

Zeevaert, L., Foundation Design and Behavior of Tower LatinoAmericano in Mexico City, *Geotechnique*, 7, 1957.

Zeevaert, L., *Foundation Engineering for Difficult Subsoil Conditions*, Van Nostrand Reinhold Co., New York, 1972, pp. 245–281.

Zur, A. and Wiseman, G., A Study of Collapse Potential of an Undisturbed Loess, *Proceedings of the International Conference on Soil Mechanics and Foundation Engineering* Moscow, Vol. 2.2, 1973, p. 265.

Further Reading

Braun, W., Aquifer Recharge to Lift Venice, Underground Services, 1973.

Christie, T. L., Is Venice sinking? in *Focus on Environmentol Geology*, R. Tank, Ed., Oxford University Press, New York, 1973, pp. 121–123.

Dahl, H. D. and Choi, D. S., Some Case Studies of Mine Subsidence and its Mathematical Modeling, Applications of Rock Mechanics, *Proceedings ASCE, 15th Symposium on Rock Mechanics*., Custer State Park, ND, 1975, pp. 1–22.

Davies, W. E., Caverns of West Virginia, West Vir. Geol. Econ. Surv. Pub. 19A, 1958.

Dudley, I. H., Review of Collapsing Soils, *Proc. ASCE, J. Soil Mech. Found. Eng. Div.*, 96, 1970.

ENR, Texas Puts Land Subsidence Cost at $110 million, *Engineering News-Record*, Oct. 31, 1974, p. 18.

Fuqua, W. D. and Richter, R. C., Photographic Interpretation as an Aid in Delineating Areas of Shallow Land Subsidence in California, *Manual of Photographic Interpretation*, Amer. Soc. of Photogrammetry, Washington, DC, 1960.

Goodman, R., Korbay, S., and Buchignani, A., Evaluation of Collapse Potential over Abandoned Room and Pillar Mines, *Bull. Assoc. Engrg. Geol.*, 17, 27–38, 1980.

Green, J. P., An Approach to analyzing multiple causes of subsidence, in *Geology, Seismicity and Environmental Impact*, Spec Pub. Assoc. Engrg. Geol., University Publishers, Los Angeles, 1973.

Greenfield, R. J., Review of the Geophysical Approaches to the Detection of Karst, *Bull. Assoc. Eng. Geol.*, 16, 393–408, 1979.

Herak, M. and Stringfield, V. T., *Karst*, Elsevier, New York, 1972.

Holtz, W. G. and Gibbs, H., Engineering properties of expansive clays, *Trans. ASCE*, 120, 1956.

Lamoreaux, P. E., Remote Sensing Techniques and the Detection of Karst, *Bull. Assoc. Eng. Geol.*, 16, 383–392, 1979.

Legget, R. F., *Cities and Geology*, McGraw-Hill Book Co., New York, 1973.

Liszkowski, J., The influence of karst on geological environment in regional and urban planning, *Bull. Assoc. Eng. Geol.*, December 1975.

Livneh, M. and Greenstein, J., The Use of Index Properties in the Design of Pavements on Loess and Silty Clays, Transportation Research Inst. Pub. No. 77–6, Technion-Israel University, Haifa, 1977.

Mathewson, C. C., Dobson, B. M., Dyke, L. D., and Lytton, R. L., System interaction of expansive soils with light foundations, *Bull. Assoc. Eng. Geol.*, 17, 55–94, 1980.

Mayuga, M. N. and Allen, D. R., Long Beach Subsidence, in *Focus on Environmental Geology*, Tank, R., Ed., Oxford University Press, New York, 1973, p. 347.

Meyer, K. T. and Lytton, A. M., Foundation Design in Swelling Clays, paper presented to Texas Section, ASCE, October 1966.

Mitchell, J. K., Influences of Mineralogy and Pore Solution Chemistry on the Swelling and Stability of Clays, *Proceedings of the International Conference on Expansive Soils*, Haifa, Israel, 1973.

Poland, J. F., Land Subsidence in the Western United States, in *Focus on Environmental Geology*, Tank, R., Ed., Oxford University Press, New York, 1973, p. 335.

Price, D. G., Malkin, A. B., and Knill, J. L., Foundations of multi-story blocks on the coal measures with special reference to old mine workings, *Q. J. Eng. Geol. London*, 1969.

Prokopovich, N. P., Past and future subsidence along San Luis Drain, San Joaquin Valley, California, *Bull. Assoc. Eng. Geol.*, 12, 1975.

Soderberg, A. D., Expect the unexpected: foundations for dams in karst, *Bull. Assoc. Eng. Geol.*, 16, 409–426, 1979.

Sowers, G. F., Shallow foundations, in *Foundation Engineering*, Leonards, G. A., Ed., McGraw-Hill Book Co., New York, 1962, Chap. 6.

Spanovich, M., Construction over Shallow Mines: Two Case Histories, ASCE Annual and National Meeting on Structural Engineering, Pittsburgh, September 1968, Preprint 703.

Turnbull, W. J., Expansive Soils — Are We Meeting the Challenge? *Proceedings of the 2nd International Conference on Expansive Clay Soils*, Texas A&M Univ., College Station, TX, 1969.

Van Siclen, D. C. and DeWitt, C., Reply: The Houston Fault Problem, *Bull. Assoc. Eng. Geol.*, 9, 1972.

Zeevaert, L., Pore Pressure Measurements to Investigate the Main Source of Subsidence in Mexico City, *Proceedings of the 3rd International Congress: Soil Mechanics Foundation Engineering*, Zurich, Vol. II, 1953, p. 299.

Zeitlen, J. G., Some Approaches to Foundation Design for Structures in Expansive Soil Areas, *Proceedings of the 2nd International Research and Engineering Conference on Expansive Clay Soils*, Texas A&M Univ., College Station, TX, 1969.