Part II
Case histories of land subsidence due to ground-water withdrawal

8 Types of land subsidence, by Alice S. Allen, Bureau of Mines, U.S. Department of the Interior, Washington, D. C.

8.1 INTRODUCTION

Land subsidence is merely the surface symptom, and the last step, of a variety of subsurface displacement mechanisms. Not all of these mechanisms are well understood. Subsidence processes are concealed below ground; their development to the point of surface deformation may involve long periods of time; and for at least some mechanisms, significant evidence may lie outside the area directly beneath the surface subsidence. Furthermore, at some sites more than one condition favourable to subsidence occurrence may be present and require consideration in analyzing causal mechanisms and devising remedial procedures.

Subsidence is a familiar accompaniment of a variety of natural events that comprise the geologic history of many areas. For practical reasons geologic processes that are accompanied by subsidence have been examined for evidence that the range in their rates of progress extends into a time frame that may produce damaging effects in terms of man’s time scale. The processes investigated are those that remove or rearrange subsurface materials to produce void space or significant volume reduction--solution, underground erosion, lateral flow, and compaction--or, in the case of tectonic activity, deep-seated downward displacement. For all of these naturally occurring geologic processes, examples of related surface subsidence have been found, though some are rare (Allen, 1969). The incidence of subsidence is greater where some of these geologic processes are set in motion or accelerated by man’s engineering activities that involve excavation, loading, or changes in the ground-water regime.

The term "subsidence" is used in this discussion in a broad sense to include both gentle downwarping and the collapse of discrete segments of the ground surface. Displacement is principally downward, although the associated small horizontal components have significant damaging effects. The term is not restricted on the basis of size of area affected, rate of displacement, or causal mechanism.

An overview of favorable geologic settings and engineering operations that may contribute to land subsidence is presented as background for the specialized treatment of subsidence caused by ground-water withdrawal, which is the subject of this guidebook. Topics on which information is widely available are mentioned briefly. More space is given to topics for which published information is less readily available for most readers. Mining subsidence is not reviewed, but several examples of interaction between mining and natural geologic processes are cited. Subsidence in regions underlain by permafrost and in areas of active volcanism is not discussed.

8.2 THE ROLE OF SUBSURFACE SOLUTION IN SUBSIDENCE

Common soluble components of earth materials that may be associated with subsidence include salt, gypsum, and the carbonate rocks--limestone and dolomite. The roles that these soluble materials play in the development of surface subsidence depends in part on the degree of their solubility, and in part on other physical characteristics.

8.2.1 Salt

Although rock salt (sodium chloride) is one of the most soluble of the common earth materials, the presence of underlying salt deposits has only rarely been associated with surface subsidence under natural conditions in recent times. This is in part because the original occurrence of salt deposits is limited geographically, and in part because salt deposits have already been removed to considerable depths except in arid climates by the leaching action of ground water. Collapse breccias found in strata overlying salt-bearing horizons constitute geologic evidence that subsidence has taken place under natural conditions in the geologic past. Collapse breccias due to solution along the margins of underlying salt deposits have been reported from the Michigan Basin (Landes, 1963), the Delaware basin of West Texas and southeastern New Mexico (Maley and Huffington, 1953), and in the western Canadian area underlain by evaporites of the Prairie Formation (DeMille and others, 1964).

In south-central Kansas where salt deposits still exist at depths between 90 and 120 m, a dramatic example of natural subsidence in historic time was documented in photographs in 1879 (Johnson, 1901, Pls. 136-138). A deep crater about 60 m in diameter was discovered disrupting a cattle trail. The interrupted tracks of a wagon that had passed 3 weeks earlier were clearly seen on both sides of the sinkhole. Another sinkhole about 130 km to the northeast carried away a railroad station overnight (Johnson, 1901, p. 713, footnote).

Indirect evidence of natural subsidence is the presence of surface depressions occupied by lakes or swamps in areas where underlying salt deposits have been undergoing dissolution. The eastern boundary of the Kansas salt deposits is fairly abrupt where salt deposits 60 m thick are missing in wells a few miles to the east. Above, the blunt edge of the salt is a narrow belt of marshes, swamps, and lakes, many of which contain salty water (Bass, 1926). Lakes also occur overlying salt domes in Louisiana (Barton, 1936, O'Donnell, 1935). At an oil-producing operation at Sour Lake dome in Texas, Sellards (1930) described one lake that had formed under natural conditions, and a large sinkhole that appeared in 1929 which was attributed to removal of salt in the saturated water that had been produced along with the oil over a long period of time.

The incidence of subsidence in some areas underlain by salt deposits has been stimulated by salt mining operations. In Cheshire, England, where salt has been mined since pre-Roman times, the effects of solution subsidence on the topography and on structures have been spectacular (Calvert, 1915; Howell and Jenkins, 1977; Wallwork, 1973). Early mining was by the room-and-pillar method in which pillars were left to support the surface. As unsaturated ground water gained access to old mine workings, the dissolution of pillar salt led to surface subsidence, though it was limited as the ground water in contact with the salt became saturated. When methods of salt production changed to pumping the so-called "wild" brines, surface subsidence was greatly accelerated. Water levels were lowered by continued pumping, and additional under-saturated ground water circulated randomly through the cavernous saltbeds, continually removing any protective envelope of saturated brine that may have developed. The topography, previously modified by natural solution subsidence, was further changed by the development of craterlike depressions 10 to 200 m in diameter, and linear hollows over 200 m wide and 8 km long. Streets and railroad tracks were distorted. In Northwich, very few pre-1900 buildings survived the subsidence damage. In the 1970's, natural brine pumping is being phased out, and most salt production is by controlled solution mining. Fresh water or undersaturated brine is injected through boreholes into deeper deposits of massive salt, creating regularly spaced solution cavities about 90 m in diameter. Mature cavities are maintained in stable condition by flooding with saturated brine.

A recent investigation of subsidence related to salt dissolution in Kansas found only five subsidence events due to salt mining over an 88-year period, and eight subsidences related to oil and gas operations (Walters, 1977). The rare subsidence occurrences were attributed to aquifers above the salt not being adequately isolated by surface casing or, in the case of salt-water disposal wells, casing failures which permitted flow of unsaturated brine across the salt.

8.2.2 Gypsum

Gypsum ($CaSO_4 \cdot 2H_2O$) is a soluble rock-forming mineral which, with its anhydrous counterpart, anhydrite, occurs abundantly in marine evaporite basin deposits. Evidence that surface subsidence was caused by dissolution of gypsum in past geologic times includes collapse features in rocks overlying gypsum deposits in the Roswell basin, New Mexico (Bean, 1949), and in the southern Black Hills of South Dakota and Wyoming (Bowles and Braddock, 1960), and solution-subsidence troughs in the gypsum plain of west Texas and southeastern New Mexico (Olive, 1957).

Sinkholes on the present-day land surface have been reported in areas underlain by gypsum-bearing rocks in New Mexico (Bean, 1949; Morgan, 1942) and Oklahoma (Fay, 1959) in the United States and at various localities in Europe (International Association of Engineering Geology, 1973).

In addition to collapse and sinkholes that overlie deposits of relatively pure gypsum, subsidence may also be associated with rocks and soils that contain minor amounts of gypsum. Klein (1966) described several types of gypsum occurrence in a very arid part of the San Joaquin Valley, California, which were investigated by the Bureau of Reclamation in connection with locating and designing a large water-transfer, pump-storage, and irrigation project. Along margins of periodic shallow lakes, efflorescent accumulations of gypsum contained solution cavities that were believed responsible for damages to canals and embankments. Weathered-shale bedrock contained secondary gypsum in veinlets and seams, which made up from 2 to 5 per cent of the rock mass. Similar veinlets and seams of gypsum characterized the weak, clayey gravel in one of the abutments of the St. Francis dam near Los Angeles, which failed disastrously in 1928 soon after the reservoir had been filled. Solution of gypsum was cited as a likely contributor to disintegration of the weak foundation material (Ransome, 1928).

The presence of small quantities of gypsum (1 to 3 per cent of dry weight) appears to be a general indicator of soils in the San Joaquin Valley that are susceptible to subsidence on wetting, but the role played by the gypsum is conjectural. Bull (1964) concluded that the gypsum content cannot be used exclusively as an indicator of potential subsidence, and he did not consider solution of gypsum to be a major cause of the subsidence. Differences in gypsum content of subsiding and nonsubsiding soils reflects compositional differences in their source areas, and possibly the removal of gypsum from nonsubsiding soils by water percolating from streams. Klein (1966) believed that the presence of gypsum contributed to the flocculation of clay particles influencing the size and amount of pore space, and shared responsibility for the low density of these deposits with the presence of trapped air inherited from their mudflow origin. Noting that much of the gypsum occurred as minute efflorescent crystals coating the small voids characteristic of the subsiding soils before hydrocompaction, he suggested that gypsum supplemented the clay minerals as a weak and easily soluble cementing agent.

8.2.3 Carbonate rocks

The carbonate rocks, limestone and dolomite, are responsible for the most widespread incidence of subsidence related to solution, not because of a high degree of solubility, but because of wide geographic distribution. A great deal of information is available on solution features in carbonate rocks (Internat. Assoc. of Engineering Geology, 1973; Tolson and Doyle; 1977, Transportation Research Board, 1976). LaMoreaux, LeGrand, and Stringfield (1975, p. 45-47) list more than 50 symposia and conferences on hydrology of carbonate rocks held throughout the world in the past 30 years.

The incidence of sinkhole development may be greatly increased when equilibrium conditions are disturbed by man’s construction projects or mining operations, particularly those that alter ground-water levels or increase surface infiltration. Newton (1976) reported that more than 4,000 induced sinkholes, areas of subsidence, or other-related features have occurred in Alabama since 1900, most of them since 1950. In Missouri, 97 catastrophic surface failures have been recorded since the 1930’s, of which 46 were attributed to man’s activity (Williams and Vineyard, 1976). Subsidence accelerated by dewatering of underground mines in carbonate terrain has been described by Foose (1968).

Collapse at the ground surface may appear suddenly, but is the culmination of a sequence of processes starting with the development of solution openings in bedrock. Interconnecting systems of solution passageways develop over geologic time and persist, owing to a combination of the slow rate of dissolution of carbonate rocks and their high compressive strength, which maintains the integrity of the cavity systems. Subsequently, unconsolidated overburden materials may be slowly washed down into bedrock cavity systems. The resulting voids in the overburden may become enlarged until the remaining cover is too thin to support the surface and collapse takes place.

8.3 THE ROLE OF SUBSURFACE MECHANICAL EROSION IN SUBSIDENCE

Subsurface mechanical erosion is the term used for an infrequently recognized phenomenon in which temporary subsurface flow channels are developed in unconsolidated or friable materials that may lead to surface collapse. The term "piping" has also been used for this process. Water percolating through pervious surficial materials becomes diverted to a more or less horizontal path on reaching the water table or a less pervious stratum. The water, which transports grains of silt and sand, finds an outlet along a nearby valley wall or cliff face or internally in caves, mine openings, or boreholes. Erosion tends to work headward from the outlet, creating and enlarging a tunnel that intersects the vertical flow channel of concentrated percolation water. As tunnel enlargement and upward propagation of the roof reduce the support capacity of the surface materials, the ground surface collapses to produce sinkholes.

In order to produce surface subsidence, the subsurface erosion mechanism is believed to require three conditions (Allen, 1969): (1) A pervious, easily erodible material must be overlain by material sufficiently competent, at least temporarily, to form a roof above the developing tunnel; (2) water must have access to the erodible material with sufficient head to transport grains of silt or sand; and (3) some sort of outlet must be available for disposal of the flowing water and the sediment grains that it transports. Examples of subsidence attributed to subsurface erosion in a variety of geologic materials are summarized in Table 8.1.

Table 8.1. Occurrences of subsidence due to subsurface erosion.

Location (Reference)	Surface expression	Erodible material	Roof	Channel development	Outlet
Kanus, China (Fuller, 1922)	Circular holes 1.5-6 m diam.; vertical walls	Loess	Dry loess	Tunnels up to 1 m wide, 3 m high	Ravine walls; plateau rims
Kamloops, B.C., Canada (Buckham and Cockfield, 1950)	Circlar sinkholes 15-30 m diam; funnel shaped	Pleistocene "White Silts" on Thompson River Terraces	Dry silt	Nearly horizontal passageways up to 1 m high; at temporary water table	Gulleys; terrace front
Eastern Oregon (Parker and other, 1964)	"Pseudokarst" (resembles karst topography in limestone areas)	Altered tuff and volcanic ash	Dry tuff and ash	Eroded cavern complex 200 m long with 4 levels	Hanging valley walls
Chuska Mountains NW New Mexico (Wright, 1964)	Pleistocene depressions containing intermittent lakes	Uncemented eolian sandstone	Cemented sandstone strata	Inferred but not observed; process inactive at present	Steep mountain escarpments
Zuni Dam, New Mexico (Eckel, 1939)	Cracking and subsidence of abutments, 1909, 1936	Sand bed 1-2.5 m thick; underlain by clay	Basalt flow, jointed	Through joints in basalt, water under head flushed out sand, creating large voids	Cliff below dam
Memphis, Tennessee (Terzaghi, 1931)	Subsidence of building and strip of land 200 m long; bluff subsided 18 m over 2 1/2 month period	Bed of uniform, rounded, fine quartz sand, 14 m thick	Fairly stiff clay	Inferred channel eroded along under side of sand, leaving cavity which collapsed	Mississippi River bluff
Minneapolis-St. Paul, Minnesota (Schwartz, 1936; Soper, 1915)	A few sinkholes; not all tunnels broken through to surface	Poorly cemented St. Peter sandstone (Ordovician)	Platteville limestone	Caves developed in friable sandstone, probably more than a kilometre long in places	River gorges
Attala County, Mississippi (Parks, 1963)	Caves and sinkholes near hilltops and heads of gullies	Thin deposits of leached silt	Quartzitic sandstone, fractured	Through cracks in quartzitic bed, water washes out silt forming tunnels 6+ m long and caves 1+ m high	Heads of gullies

The material that forms the roof of tunnels at some localities is a different, and more competent, material than that in the eroded horizon. In other localities, the material forming the roof and the eroded horizon are the same (i.e. loess, altered volcanics), but the competency of the roof is dependent on cohesion in a dry condition provided by montmorillonitic clay bonding. Such cohesion is lost upon saturation as the component particles become disaggregated when wet.

Cases of underground mechanical erosion are difficult to identify. At least part of the process must be inferred in the absence of direct observation. Tunnel development is concealed below ground and may only be disclosed by the apparently sudden collapse at the surface. The collapse is the last step in a long continued process in which sediments are eroded grain by grain and transported to the outlet. Accumulations of transported sediments are rarely observed because the silt and sand grains either become incorporated in the colluvium below the outlet on a valley wall, or are washed down into cavities or excavations in the bedrock.

8.4 LATERAL FLOW AS A SUBSIDENCE MECHANISM

Lateral flow of subsurface materials as a cause of subsidence is uncommon but not unknown. Examples have been reported both under natural geologic conditions and under loading by man’s activities (Allen, 1969). Common earth materials susceptible to plastic flow are salt, gypsum, clay, and clay shale.

Geologic examples of subsidence by salt flowage are rim synclines surrounding salt domes in coastal Texas and Louisiana (Nettleton, 1934; Ritz, 1936) and broad synclines associated with salt tectonics in the Paradox Basin in Utah and Colorado (Cater, 1954). Where the Green and Colorado Rivers have cut deep canyons well down into the formation overlying salt and gypsum in Utah, the removal of load has permitted salt and gypsum to flow laterally, resulting in very local folds and graben (Baker, 1933).

Flowage of shale has produced a geologic subsidence feature termed "cambering" in the Jurassic iron ore locality in east-central England (Hollingworth and Taylor, 1951). Cambering occurs in deeply dissected areas in which a competent rock such as ironstone or limestone overlies Lower Jurassic clay shale. Lateral flowage of clay shale toward the valley axes has lowered the overlying competent rock as much as 30 m; in places the lowering has been intensified by subsurface erosion along the shale-ironstone contact and by sliding of the ironstone on the shale surface.

On thick glacial clay deposits in the Great Lakes region of North America, lateral flowage has been induced beneath stockpiles of ore, resulting in slight-lowering of the ground surface and increasing the distance between ore-retaining walls over a few decades by nearly 2 m (Terzaghi, 1953).

8.5 COMPACTION AS A CAUSE OF SUBSIDENCE

A common cause of ground-surface subsidence is reduction in the volume of low-density sedimentary deposits that accompanies the process of compaction, in which particles become more closely packed and the amount of pore space is reduced. Compaction may be induced by loading, by drainage, by vibration, by extraction of pore fluids, and under certain conditions by the application of water. Compaction occurs both naturally and by man’s manipulation.

The amount of subsidence effected by compaction is a function of the relative amount of pore space in the material as originally deposited, the effectiveness of the compacting mechanism, and the thickness of the deposit undergoing compaction. Natural deposits of unusually high initial porosity include modern delta deposits, terrigenous mudflows, undisturbed loess, and peat.

8.5.1 Loading

The effects of natural loading are most apparent where great thicknesses of fine-grained sediments accumulate rapidly. The process of compaction is accompanied by contemporary subsidence. On the modern Mississippi River delta, 300 to 500 million tons of sediment is deposited each year. Fisk and others (1954) found that levee deposits on the lower delta had subsided 6 m and interdistributary marsh deposits, 8.5m.

At New York’s La Guardia Airport, the natural compaction of an 18-m thickness of saturated organic silt and clay deposits was accelerated by artificial loading (Engineering News Record, 1949; Kyle, 1951). Half the airport was reclaimed from Flushing Bay by placing 7.5 m of fill over the saturated sediments. After 25 years of operation, parts of the filled area had subsided

2.5 m, with further subsidence anticipated (Halmos, 1962). Protection from tide waters is furnished by a dike around three sides of the airport, which was built on soil stabilized by sand drains that extend 20 m below sea level.

8.5.2 Drainage

In low-lying areas, lowering of the water table by artificial drainage stimulates compaction of sediments with accompanying subsidence of the surface. Compaction rates have much more than academic significance in areas such as the polders of the Netherlands where vast regions have been reclaimed from the sea by building dikes and installing pumps. Bennema and others (1954) found that clay deposits containing 30 to 35 per cent of a minus 2-micrometre fraction compressed to about half their original thickness after reclamation over a 100-year period. Sediments with about 20 per cent fine fraction compressed about 25 per cent; compaction of sand layers was negligible.

Drainage of peat areas can be expected to result in subsidence for two reasons. Peat is commonly underlain by, and frequently interbedded with, fine sediments that are susceptible to compaction when drained. In addition, peat has certain physical and chemical characteristics that lead to extreme volume changes upon drying (Highway Research Board, 1954; MacFarlane, 1959; Stephens and Speir, 1969). Peat has a water-holding capacity ranging from 300 to 3,000 per cent.

Its bulk density is extremely low--about 960 kg/m^3 when wet and 64 kg/m^3 when dry. Particle specific gravity is also low--between 1.0 and 2.0. Furthermore, peat undergoes irreversible biochemical changes on drying that reduce volume. The largest peat areas in the United States that have been subsiding following reclamation for agricultural development are the Florida Everglades (Stephens and Speir, 1969) and the delta area at the confluence of the Sacramento and San Joaquin Rivers in California (Weir, 1950). In the Chikuho coalfield in Japan, subsidence in areas underlain by thick peat and organic clay deposits was attributed to their compaction in response to the lowering of ground-water levels during mining operations (Noguchi, Takahashi, and Tokumitsu, 1969).

8.5.3 Vibration

Sedimentary materials may be compacted by vibration under natural conditions during earthquakes. Buildings on saturated alluvium or uncompacted fill may subside or settle differentially in response to earthquake vibrations or, if the foundations are tied to a lower stable stratum, the buildings may appear to rise as the surrounding sediments subside by compaction.

A variety of manmade sources of vibration have been cited by Terzaghi and Peck (1967) as having produced subsidence by compaction of underlying earth materials. These sources of vibration include heavy rock-crushing equipment, turbogenerators, truck traffic, an elevated railway, pile driving and blasting. At sites of structures to be built on saturated sand, future subsidence may be forestalled by the vibroflotation process of foundation treatment. Giant vibrators fitted with jets are lowered to the desired depth and withdrawn slowly, resulting in cylinders of compacted sand (Sowers and Sowers, 1961). Loose foundation materials may also be densified by buried charges of explosives (Lyman, 1942). Underground nuclear explosions in unconsolidated materials are characterized by craters on the ground surface (Drell, 1978).

8.5.4 Extraction of pore fluids

Of all causes of land subsidence, both natural and those induced by man’s activities, subsidence associated with extracting fluids from subsurface formations is best understood. Many areas of subsidence caused by pumping of artesian water, oil, and gas have been identified, surface and subsurface changes have been monitored, and corrective measures have been devised. A decade ago the topic of "Land Subsidence Due to Fluid Withdrawal" was reviewed by Poland and Davis (1969). Current progress in identifying and coping with subsidence caused by withdrawing ground water in many parts of the world is reported in the case histories comprising Chapter 9 of this volume.

8.5.5 Hydrocompaction

Certain materials of unusually low density deposited in areas of low rainfall undergo significant compaction when they become thoroughly wetted. The process, termed "hydrocompaction," produces rapid and irregular subsidence of the ground surface, ranging from 1 to nearly 5 m. Reclamation projects that import and distribute irrigation water in dry areas underlain by loess and by mudflow deposits have encountered subsidence problems. It is thought that clay bonding of the particles is responsible for maintaining open textures while the deposits are in a dry condition, and for rapid disaggregation and volume loss when immersed in water (Bull, 1964). Surface subsidence resulted from wetting without the addition of surcharge load at many sites; at others, a combination of water infiltration and surface loading was required. A review of the phenomenon of hydrocompaction by Lofgren (1969) describes the process and associated subsidence occurrences in the United States, Europe, and Asia.

8.6 TECTONIC SUBSIDENCE

Large areas of measurable downward displacement have been associated with a few earthquakes of large magnitude. The 1959 Hegben Lake earthquake in Montana produced an asymmetrical subsided area 69 by 22 km, in which the maximum subsidence was 6.6 m (Myers and Hamilton, 1964). During the 1960 series of earthquakes in Chile, subsidence of 1 to 1.5 m was reported to have affected a coastal area 600 by 30 km (Weishcet, 1963). The 1964 Alaska earthquake produced an asymmetrical downwarped area 800 by 160 km. Tectonic subsidence, which ranged up to 2.3 m, was augmented in many places by compaction of unconsolidated materials (Plafker, 1965).

8.6.1 Discussion

The state-of-the-art in land-subsidence analysis progresses unevenly because the degree of understanding of various subsidence mechanisms varies. Most study has been directed to subsidence related to man’s engineering activities. This is facilitated by availability of data on quantities of subsurface material removed (or injected), on rates and duration of extraction operations, and on changes in ground-water levels. Natural processes are not as easily quantified.

A case of land subsidence is necessarily the integrated surface expression of whatever processes may be active at that site, whether natural or manmade, or both. A working hypothesis as to the mechanism or combination of mechanisms operative at the specific site is requisite for designing control measures. The complexity of subsidence mechanisms and their interaction requires cooperative effort among different disciplines, both in collecting physical evidence and in developing the rationale for the processes involved. The hydrologic sciences have been, and will continue to be, significant contributors to land subsidence investigations.

8.7 REFERENCES

ALLEN, ALICE S. 1969. Geologic settings of subsidence, in Reviews in Engineering Geology, Volume II. Geol. Soc. America, P. 305-342.

BAKER, A. A. 1933. Geology and oil possibilities of the Moab district, Grand and San Juan Counties, Utah. U.S. Geol. Survey Bull. 841, 95 p.

BARTON, D. C. 1936. Late recent history of the Côte Blanche salt dome, St. Mary Parish, Louisiana. Am. Assoc. Petroleum Geologists Bull., v. 20, no. 2, p. 179-185.

BASS, N. W. 1926. Structure and limits of the Kansas salt beds. Kansas Geol. Survey Bull. 11, P. 90-95.

BEAN, R. T. 1949. Geology of the Roswell Artesian Basin, New Mexico, and its relation to the Hondo Reservoir. New Mex. State Engineer Office Tech. Rept., no. 9, p. 1-31.

BENNEMA, J., GEUZE, E. C. W. A., SKITS, H., and WIGGERS, A. J. 1954. Soil compaction in relation to Quaternary movements of sea-level and subsidence of the land especially in the Netherlands. Geologie en Mijnbouw, new ser., v. 16, no. 6, p. 173-178.

BOWLES, C. G., and BRADDOCK, W. A. 1960. Solution breccias in the upper part of the Minnelusa sandstone, South Dakota and Wyoming (Abstract). Geol. Soc. America Bull., v. 71, p. 2032.

BULL, W. B. 1964. Alluvial fans and near-surface subsidence in western Fresno County, California. U.S. Geological Survey Professional Paper 437-A, 71 p.

BUCKHAM, A F., and COCKFIELD, W. E. 1950. Gullies formed by sinking of the ground (British Columbia). Am. Jour. Sci., v. 248, no. 2, p. 137-141.

CALVERT, A. F. 1915. Salt in Cheshire. London, E. and F. N. Spon, Ltd.; New York, Spon and Chamberlain, 1,206 p.

CATER, F. W., Jr. 1954. Geology of the Bull Canyon Quadrangle, Colorado. U.S. Geol. Survey Geol. Quad. Map (GQ-33).

DeMILLE, G., SHOULDICE, J. R., and NELSON, H. W. 1964. Collapse structures related to evaporites of the Prairie Formation, Saskatchewan. Geol. Soc. America Bull., v. 75, p. 307-316.

DRELL, S. D. 1978. The case for the test ban. Washington Post, July 4, p. A17.

ECKEL, E. B. 1939. Abutment problems at Zuni Dam, New Mexico. Civil Eng., v. 9, no. 8, p. 490-492.

ENGINEERING NEWS-RECORD. 1949. Saving LaGuardia Airport. Eng. News-Rec., v. 143, no. 24, p. 35-37.

FAY, R. 0. 1959. Guide to Roman Nose State Park, Blaine County, Oklahoma. Oklahoma Geol. Survey Guidebook 9, 31 p.

FISK, H. N., McFARLAN, EDWARD, Jr., KOLB, C. R., and WILBERT, L. J. 1954. Sedimentary framework of the modern Mississippi delta. Jour. Sed. Petrology, v. 24, no. 2, p. 76-99.

FOOSE, R. M. 1968. Surface subsidence and collapse caused by ground-water withdrawal in carbonate rock areas. 23d Internat. Geological Congress, Proceedings, v. 12, p. 155-166.

FULLER, M. L. 1922. Some unusual erosion features in the loess of China. Geog. Rev., v. 12, p. 570-584.

HALMOS, E. E. 1962. Face lift for a busy airport. Excavating Engineer, v. 56, no. 6, p. 18-22.

HIGHWAY RESEARCH BOARD. 1954. Survey and treatment of marsh deposits. Natl. Research Council, Highway Research Board Bibliography 15, Pub. 314, 95 p.

HOLLINGWORTH, S. E., and TAYLOR, J. H. 1951. The Northampton Sand Ironstone; stratigraphy, structure and reserves. Great Britain, Geol. Survey Memoir, 211 p.

HOWELL, F. T., and JENKINS, P. L. 1970. Some aspects of the subsidences in the rocksalt districts of Cheshire, England, in Proceedings, 2d International Symposium on Land Subsidence. Anaheim, Calif., 1976; Internat. Assoc. Sci. Hydrologists Pub. No. 121, pp. 507-520.

INTERNATIONAL ASSOCIATION OF ENGINEERING GEOLOGY. 1973. Symposium--sinkholes and subsidence-engineering-geological problems related to soluble rocks. Proceedings published by Deutsche Gesellschaft für Erd und Grundbau, Essen.

JOHNSON, W. D. 1901. The High Plains and their utilization. U.S. Geol. Survey 21st Ann. Rept., pt. 4, 601-741.

KLEIN, IRA E. 1966. Foundation and ground-water problems related to the occurrence of gypsum in hydraulic engineering works of the United States Bureau of Reclamation in the San Luis Unit of the Central Valley Project in California, in International Symposium on Public Works Construction in Gypsiferous Terrains, Madrid, 1962. Servicio Geologico de Obras Publicas, Madrid, Paper C.2-7, 38 p.

KYLE, J. M. 1951. Settlement correction at La Guardia Field. Am. Soc. Civil Engineers Trans., v. 116, p. 1343-1348.

LaMOREAUX, P. E., LeGRAND, H. E., and STRINGFIELD, V. T. 1975. Progress of knowledge about hydrology of carbonate terrains, in Burger, A., and Debertret, L., Hydrogeology of karstic terrains. Internat. Assoc. of Hydrogeologists, Internat. Union of Geological Sciences, Ser. B., no. 3, Paris, p. 41-52.

LANDES, K. K. 1963. Effects of solution of bedrock salt in the earth's crust, p. 64-73 in Bersticker, A. C., and others, eds., Symposium on salt. Cleveland, Northern Ohio Geol. Soc., 661 p.

LOFGREN, B. E. 1969. Land subsidence due to the application of water, in Reviews in Engineering Geology, Volume II. Geol. Soc. of America, p. 271-303.

LYMAN, A. K. B. 1942. Compaction of cohesionless foundation soils by explosives. Am. Soc. Civil Engineers Trans., v. 107, Paper no. 2160, p. 1330-1341.

MacFARLANE, I. C. 1959. A review of the engineering characteristics of peat. Am. Soc. Civil Engineers Proc., Jour. Soil Mechanics and Found. Div., V. 85, no. SMI, pt. 1, p. 21-35.

MALEY, V. C., and HUFFINGTON, R. M. 1953. Cenozoic fill and evaporite solution in the Delaware Basin, Texas and New Mexico. Geol. Soc. America Bull., v. 64, p. 539-546.

MORGAN, A. M. 1942. Solution phenomena in the Pecos Basin in New Mexico. Am. Geophys. Union Trans., v. 23, pt. 1, p. 27-35.

MYERS, W. B., and HAMILTON, WARREN. 1964. Deformation accompanying the Hebgen Lake earthquake of August 17, 1959. U.S. Geol. Survey Prof. Paper 435-1, p. 55-98.

NETTLETON, L. L. 1934. Fluid Mechanics of salt domes. Am. Assoc. Petroleum Geologists Bull., v. 18, no. 9. p. 1175-1204.

NEWTON, J. G. 1976. Induced and natural sinkholes in Alabama--a continuing problem along highway corridors. Natl. Acad. Sci. Transportation Research Record 612, Part 1, pi 9-16.

NOGUCHI, T., TAKAHASHI, R., and TOKIMITSU, Y. 1969. On the compression subsidence of peat and humic layers in the Kami-Shinbashi area, Kurate-Machi, Kurate-Gun, Fukuoka Prefecture, in Tison, L. J., ed., Land Subsidence, v. II. Internat. Assoc. Sci. Hydrology, Pub. 89, p. 458-466.

O'DONNELL, LAWRENCE. 1935. Jefferson Island salt dome, Iberia Parish, Louisiana. Am. Assoc. Petroleum Geologists Bull,. v. 68, p, 351-358.

OLIVE, W. W. 1957. Solution-subsidence troughs, Castile formation of Gypsum Plain, Texas and New Mexico. Geol. Soc. America Bull., v. 68, p. 351-358.

PARKER, G. G., SHOWN, L. M., and RATZLAFF, K. W. 1964. Officer's Cave, a pseudokarst feature in altered tuff and volcanic ash of the John Day Formation in eastern Oregon. Geol. Soc. America Bull., v. 75, p. 393-402.

PARKS, W. S. 1963. Attala County mineral resources. Mississippi Geol. Econ. and Topog. Survey Bull. 99, 191 p.

PLAFKER, GEORGE. 1965. Tectonic deformation associated with the 1964 Alaska earthquake of March 27, 1964. Science, v. 148, no. 3678, p. 1675-1687.

POLAND, J. F., and DAVIS, G. H. 1969. Land subsidence due to withdrawal, of fluids, in Reviews in Engineering Geology, Volume II. Geol. Soc. America, p. 187-269.

RANSOME, F. L. 1928. Geology of the St. Francis damsite. Economic Geology, v. 23, p. 553-563.

RITZ, C. H. 1936. Geomorphology of Gulf Coast salt structures and its economic application. Am. Assoc. Petroleum Geologists Bull., v. 20, no. 11, p. 1413-1438.

SCHWARTZ, G. M. 1936. The geology of the Minneapolis-St. Paul metropolitan area. Minnesota Geol. Survey Bull. 27, 267 p.

SELLARDS, E. H. 1930. Subsidence in Gulf coastal plain salt domes. Texas Univ. Bull. 3001, p. 9-36.

SOPER, E. K. 1915. The buried rock surface and pre-glacial river valleys of Minneapolis and vicinity. Jour. Geology, v. 23, p. 444-460.

SOWERS, G. B., and SOWERS, G. F. 1961. Introductory soil mechanics and foundations (2d ed.). New York Macmillan, 386 p.

STEPHENS, J. C., and SPEIR, W. H. 1969. Subsidence of organic soils in the U. S. A., in Tison, L. J., ed., Land subsidence, v. 1. Internat. Assoc. Sci. Hydrology, Pub. No. 89, p. 523-534.

TERZAGHI, KARL. 1931. Earth slips and subsidences from underground erosion. Eng. News-Rec., v. 107, no. 3, p. 90-92.

TERZAGHI, KARL. 1953. Foundation of buildings and dams, bearing capacity, settlement observations, regional subsidences. Zurich, 3d Internat. Conf. on Soil Mechanics and Foundation Eng., 3rd, Proc., v. 3, p. 158-159.

TERZAGHI, KARL, and PECK, R. B. 1967. Soil mechanics in engineering practice (2d ed.). New York, John, Wiley, 729 p.

TOLSON, J. S., and DOYLE, F. L., eds. 1977. Karst hydrology. Internat. Assoc. of Hydrogeologists, 12th Internat. Congress, Huntsville, Alabama, 1975, Proceedings, 578 p.

TRANSPORTATION RESEARCH BOARD. 1976. Subsidence over mines and caverns, moisture and frost actions, and classification. Nat. Acad. Sci., Transportation Research Record 612, 83 p.

WALLWORK, K. L. 1973. Salt and environmental planning: an historical perspective to a contemporary land-use problem, in Fourth Symp. on Salt. Northern Ohio Geol. Soc., v. 1, pp. 435-441.

WALTERS, R. F. 1977. Land subsidence in central Kansas related to salt dissolution. Kansas Geol. Survey Bull. 214, Univ. of Kansas Pub., Lawrence, Kans., 82 p.

WEIR, W. W. 1950. Subsidence of peat lands of the Sacramento-San Joaquin delta, California. Hilgardia (California Agr. Expt. Sta. Jour.), v. 20, no. 3, p. 37-56.

WEISCHET, WOLFGANG. 1963. Further observations of geologic and geomorphic changes resulting from the catastrophic earthquakes of May 1960, in Chile. Seismol. Soc. America Bull., v. 53, no. 6, p. 1237-1257 (trans. by R. Von Huene).

WILLIAMS, J. H., and VINEYARD, J. D. 1976. Geologic indicators of catastrophic collapse in karst terrain in Missouri. Natl. Acad Sci. Transportation Research Record 612, pt. 1, p. 31-37.

WRIGHT, H. E., Jr. 1964. Origin of the lakes in the Chuska Mountains, northwestern New Mexico. Geol. Soc. America Bull., v. 75, p. 589-598.

9 Case Histories

Case History No. 9.1. Latrobe Valley, Victoria, Australia, by C. S. Gloe, State Electricity Commission of Victoria, Victoria, Australia

9.1.1 INTRODUCTION

The Gippsland Basin covers an area of some 40,000 km^2. Four-fifths of this area is located off-shore and the remaining fifth in the Gippsland Region of south-eastern Victoria. The offshore area contains a number of oil and gas fields, while the on-shore portion includes an area of some 800 km^2 known as the Latrobe Valley Depression where major deposits of brown coal occur beneath a thin cover of overburden.

The excavation of brown coal for the generation of electricity and production of briquettes commenced in the Latrobe Valley some 50 years ago. The coal is won from open cuts, the development of which has been accompanied by significant vertical and horizontal movements, both within the excavation as well as in surrounding areas. To enable the coal from the second major open cut to be excavated under safe operating conditions, it has been necessary to reduce the artesian pressures of underlying aquifers. The resulting increased effective stresses have induced consolidation of strata and cause subsidence which is now regional in extent.

9.1.2 GEOLOGY

The Gippsland Basin developed in the off-shore area in Upper Cretaceous times with the deposition of lacustrine and fluviatile sands and clays and a number of brown coal seams. The basin gradually developed westwards and lithologically similar sediments were deposited in the onshore area in Lower Tertiary times (Figure 9.1.1).

Within the Latrobe Valley Depression some 700 m of Tertiary sediments named the Latrobe Valley Coal Measures, and including some volcanics towards the base, were deposited mainly on Lower Cretaceous arkoses and shales. The coal measures include three groups of major coal seams, separated and underlain by clays and sands (Gloe, 1975).

In late Tertiary times the coal measures were tilted, folded and faulted. Extensive erosion followed, virtually to the stage of peneplanation. Subsequently, a thin cover of clays, silts and sands was deposited on the eroded surface. As a result of the peneplanation, considerable thicknesses of sediments were removed from uplifted blocks. In the areas of the Yallourn and Morwell open cuts, the two major open cuts in the Latrobe Valley, up to 150 m and 300 m respectively of clays, sands and brown coals were removed (Figure 9.1.2).

In the Yallourn open cut the Yallourn seam averages 60 m in thickness and underlies 10 to 15 m of younger overburden. Beneath the Yallourn seam are some 120 to 150 m of sands and clays overlying a further thick coal seam.

The Morwell 1 (Ml) seam being excavated in the Morwell open cut ranges from about 90 to 135 m in thickness beneath 12 to 15 m of overburden. Underlying the coal seam are 15 to 23 m of sands and clays followed by the Morwell 2 (M2) coal seam which is up to 50 m thick in this area. A further sequence of clays and sands, including an almost completely weathered layer of basalt and a thick basal silty gravel, totalling some 140 m in thickness, underlies the M2 seam.

9.1.3 HYDROLOGY

Unconfined ground waters are present over most of the area, but quantities are mainly small. The water occurs in the overburden sands as well as in the joint system of the uppermost coal seam.

Confined waters are found in the sands underlying coal seams, and in some fresh basalt flows. Prior to the development of the Morwell open cut, water struck in bores in low-lying areas flowed at the surface, sometimes under considerable head (Gloe, 1967).

In the area of the Morwell open cut some 8 to 9 m of medium to coarse grained, poorly sorted and highly permeable sands, known as the M1 aquifer, occur beneath the Ml seam. The sands are irregular and typical sheet deposits (Barton, 1971). Along the northern edge of the open cut the original piezometric level stood at +60 AHD, a height some 150 m above the level of the aquifer in that area.

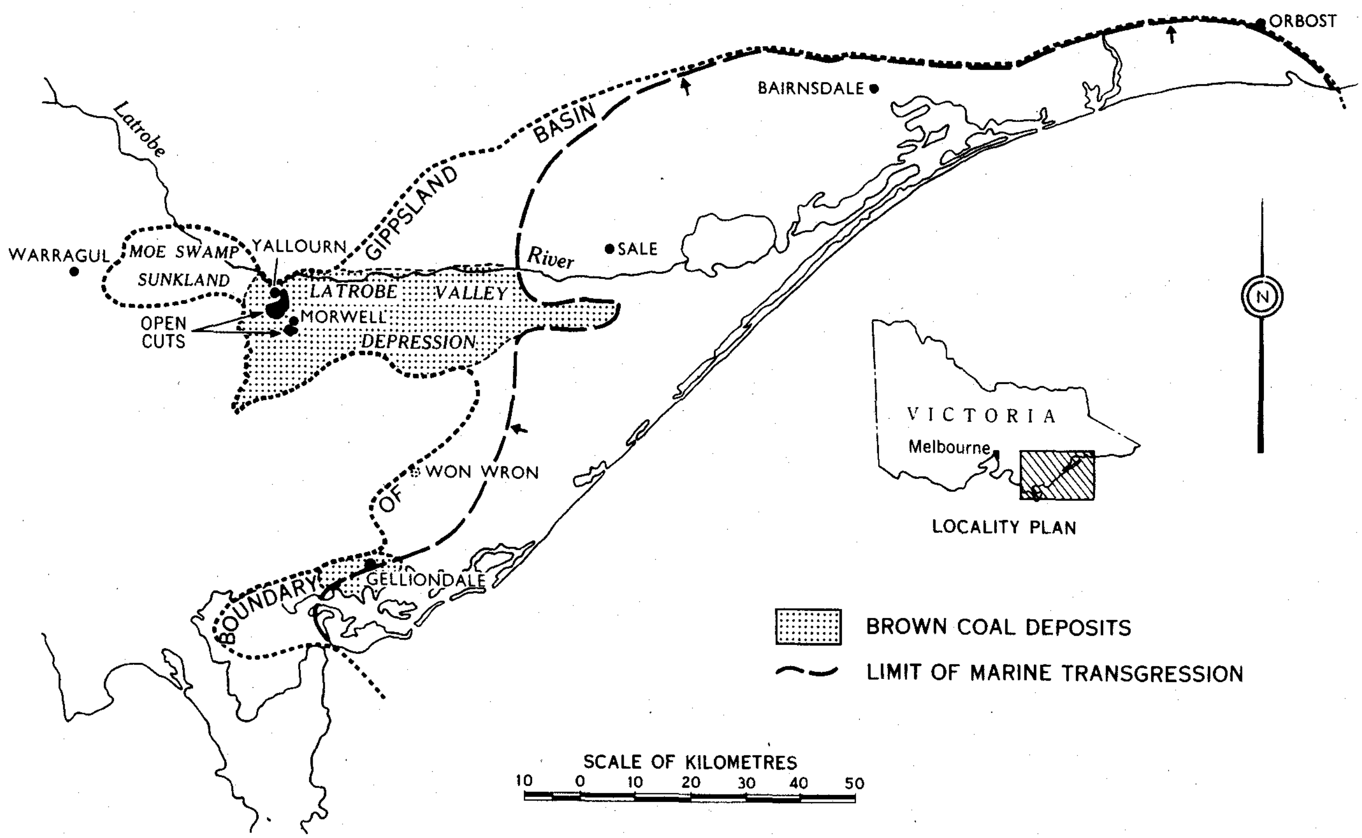


Figure 9.1.1 Gippsland Basin.

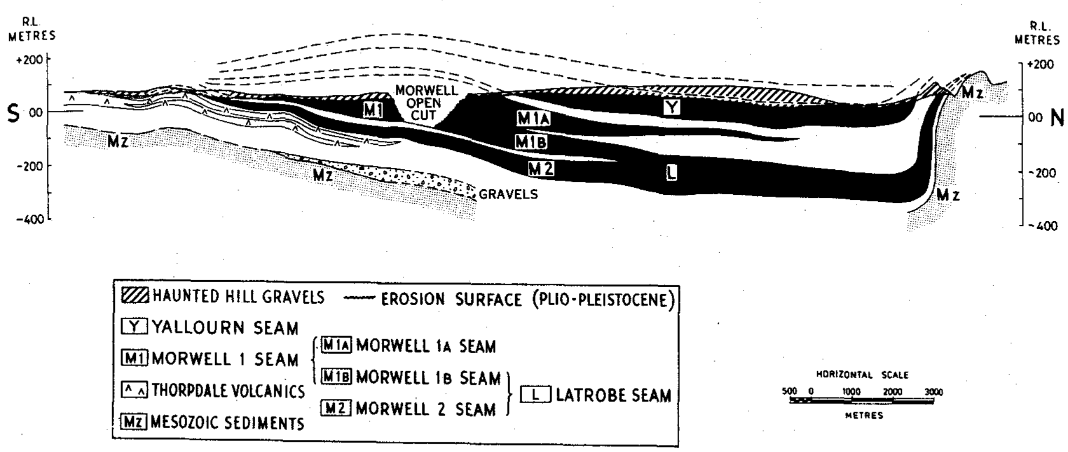


Figure 9.1.2 Reconstruction of pre-erosional stratigraphy

The M2 aquifer consists of several irregular sand beds occurring between 3 and 50 m below the M2 seam. There is evidence of vertical leakage between the M1 and M2 aquifers--partly due to numerous boreholes, partly to the fracturing which accompanied heaving of the floor of the open cut as new levels were established, but also naturally as part of a leaky aquifer system.

In the Yallourn open cut the weight of the clays underlying the Yallourn seam is sufficient to withstand the hydrostatic pressure of the artesian waters present in that area. However, at Morwell it had been calculated that the weight of coal and clay would be unable to withstand the artesian pressures once a working level had been established to an area 300 m across at a depth of some 65 m below the original surface. As this still left some 50 m of coal above the base of the seam, it was clear that the M1 aquifer pressures would need to be progressively reduced as the open cut was developed in depth (Gloe, 1967).

The main program of pressure reduction, frequently called dewatering, commenced in 1960 as the first coal cut was excavated. Initially, free-flow bores were used, new bores being established as new levels were opened up. By 1967 it was found necessary to construct pumping bores in the M1 aquifer. Subsequent investigations established that the pressures from the M2 aquifer, although already substantially reduced through leakage, would require further lowering to ensure safe operating conditions. This reduction was achieved initially through free-flow bores and subsequently using pumping bores with yields of up to 160 l/s.

The maximum rate of pumping from the M2 aquifers was 1160 l/s at which time the total pumping rate was 1320 1/s. Piezometric levels were lowered to safe operational levels and have been maintained for two years with yields of 925 1/s from M2 aquifer and 130 1/s from M1 aquifer. Total artesian water pumped from Morwell open cut to June 1977 was 250,000 x 10^6 1.

Contours of the M1 aquifer piezometric surface as at July 1977 are shown in Figure 9.1.3. The M2 aquifer levels have a generally similar pattern. The original levels were of a gently sloping surface with values of +60 AHD at Morwell open cut and rising to +65 AHD in the west.

Investigations of recharge and intake areas have included carbon dating of water samples. The youngest water from near the western edge of the basin was 2200 years old, while the water pumped from the Morwell open cut gave values of 23 500 years for the M1 and 13 800 years for the M2 aquifer waters. These ages conform with the concept of a multi-aquifer and aquitard, or leaky aquifer system, with a large volume of water in storage, but in which the upper aquifers at least are not replenished by rapid infiltration of rainwater in intake areas. It is considered that much of the water pumped from the M1 aquifer has been derived through leakage from lower aquifers and from compaction of aquitards.

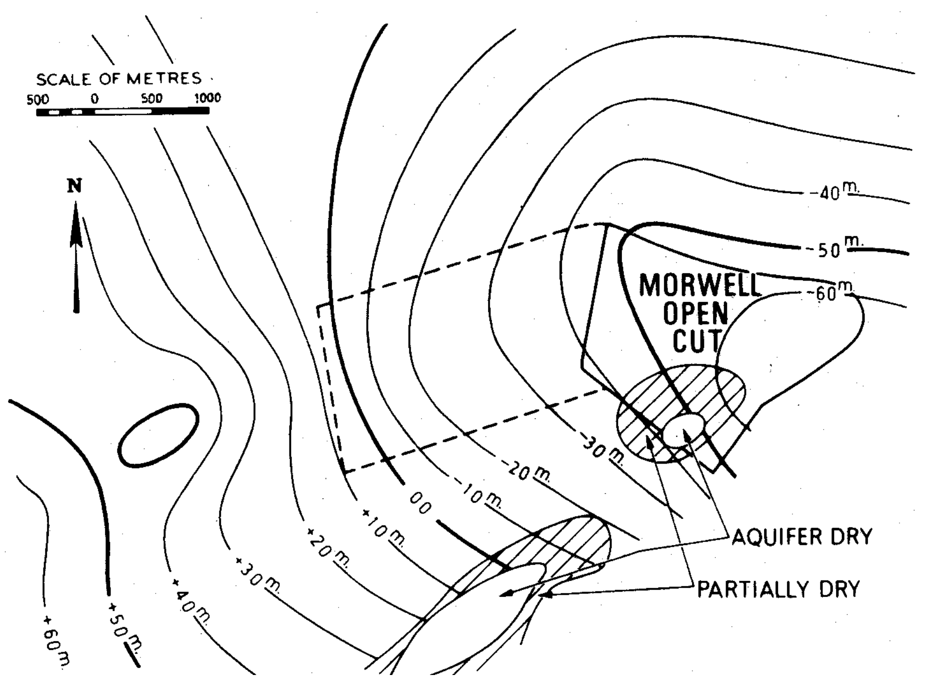


Figure 9.1.3 Piezometric surface of Morwell 1 aquifer.

9.1.4 MECHANICAL PROPERTIES OF BROWN COAL AND ASSOCIATED STRATA

9.1.4.1 Brown coal

Properties of brown coal have been described by Gloe, James and McKenzie (1973). Brown coal was shown to be a highly preconsolidated organic material with a low bulk density (1.13 g/cm^3) and very high moisture content (up to 200 per cent as expressed on an engineering basis).

Average values of preconsolidation pressures assigned to M1 and M2 coals in the vicinity of the Morwell open cut based on estimates made using Casagrande's method were 2300 kPa and 2900 kPa respectively.

The coefficient of volume decrease (mv) depends both on the consolidation pressure and initial moisture content, but for purposes of calculating consolidation settlements where the consolidation pressure is less than 1300 kPa the following values of (mv) were assigned:

M1 coal--0.2 cm^2/kN at top of seam to 0.1 cm^2/kN at base of seam;

M2 coal--0.1 cm^2/kN.

9.1.4.2 M1 aquiclude

Beneath the M1 seam there is a 3 to 13 m layer of stiff grey preconsolidated silty clay. The clay is composed of kaolinite and a-quartz with a plasticity index around 20 to 25 per cent and a liquid limit of about 60 per cent.

Average properties for the clay are

Clay fraction, 44 per cent
Bulk density, 1.8 8/cm^3
Compression index (C_c), 0.5
Coefficient of volume decrease (mv), 0.2 cm^2/kN
Consolidation pressures, 1300 kPa

9.1.4.3 M1 and M2 aquifer sands

The gradation of the M1 sand is highly variable ranging from coarse sand with fine gravel and fine sand to silty fine sand. The sand is dense to very dense and relatively incompressible. In the area of the open cut the M2 sands are generally thicker and hence have a higher transmissibility than those of the M1 aquifer. In other respects the sands are similar.

9.1.4.4 M2 aquicludes and aquitards

The M2 aquicludes and aquitards range from clays to silts with properties generally similar to those of the M1 aquiclude.

9.1.5 EXTENT OF MOVEMENTS

Surface movements, both inside and outside the Yallourn and Morwell open cuts have occurred ever since excavation commenced. Regular surveys are carried out to determine the amounts of these movements.

The movements at Morwell open cut exceed those at Yallourn open cut, mainly because of the dewatering operations and greater depth of the open cut. The surveys at Morwell which were initiated prior to the commencement of open cut operations are based on a datum line remote from the open cut with survey beacons around the open cut being located by triangulation. The beacons form the control for precise traverses of pin lines established in and around the open cut.

By 1977 when the open cut had reached its full depth and was being developed to the west, horizontal movements had reached as much as 2.25 m and vertical movement 1.68 m at the top of the northern and eastern batters. These movements decrease outwards from the edge of the open cut.

Contours of horizontal movement in the Morwell area and of subsidence in the Yallourn-Morwell area are shown on Figures 9.1.4 and 9.1.5 respectively. The pattern of horizontal movement is roughly concentric about the floor of the open cut with the major movement occurring within 400 m of the edge of the open cut. The 20 cm contour is at present stationary and approximately 1000 m north and east of the open cut. On the other hand, subsidence is far more regional in extent--now affecting the whole of the Yallourn-Morwe11 area and extending eastwards into Loy Yang, some 20 km east of Morwell. By 1977 the 20 cm and 50 cm subsidence contours were located some 7.2 km and 4.5 km respectively north of the open cut with the 50 cm contour embracing an area of 47 km^2 and including much of Morwell township.

The C line of survey marks passes through the southern portion of Morwell township at right angles to the northern edge of the open cut. Horizontal movement and subsidence profiles along the C line are shown in Figure 9.1.6. By 1977 the total southerly displacement of a point adjacent to the open cut was 2.01 m, but at a distance of 400 m from the open cut was only 0.39 m. Similar patterns of movement are found on other pin lines extending outwards from the open cut. In the vicinity of the open cut subsidence commenced at a slower rate, and at one stage was little more than half that of horizontal movement. However, a steady rate of subsidence has been maintained while horizontal movements have decreased after deepening of the open cut ceased and development extended westwards. At 200 m from the northern edge of the open cut, vertical and horizontal movements are now roughly equal, while at 800 m vertical movements exceed horizontal by 0.95 m (Figure 9.1.6).

Evidence of subsidence such as protrusion of casing above ground surface is visible at Morwell. Figure 9.1.7 shows clamps on an observation bore casing set in the M1 aquifer now standing 1.0 m above the shallow surface casing on which they originally rested.

9.1.6 CAUSES OF MOVEMENT

Factors contributing to movement were discussed by Gloe, James and Barton (1971), Gloe (1976) and Hutchings, Fajdiga and Raisbeck (1977). Apart from the geometry of the cut and the

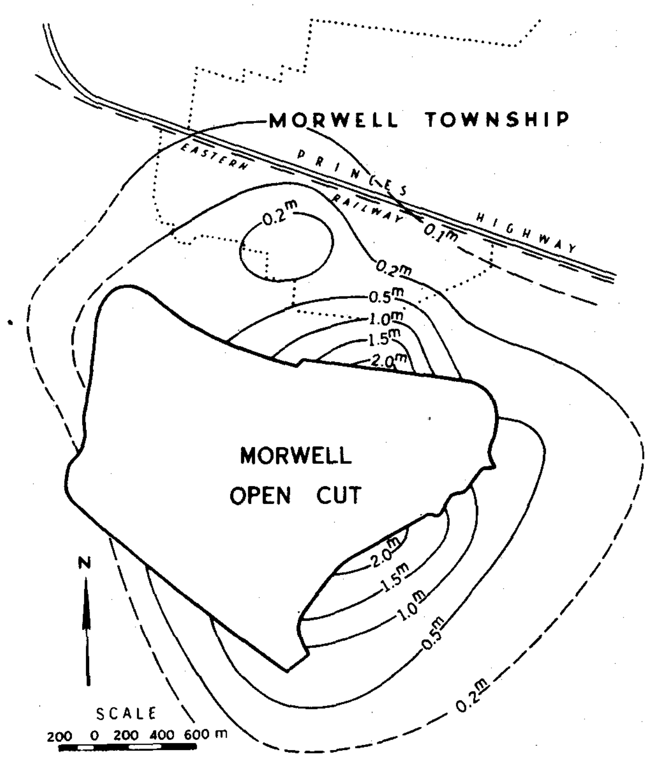


Figure 9.1.4 Horizontal movement adjacent to Morwell open cut

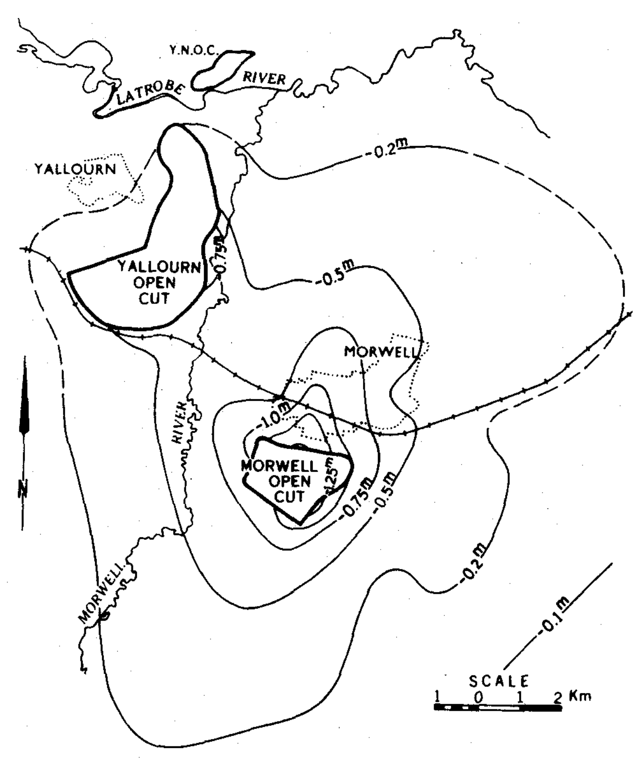


Figure 9.1.5 Regional subsidence in the Yallourn-Morwell area.

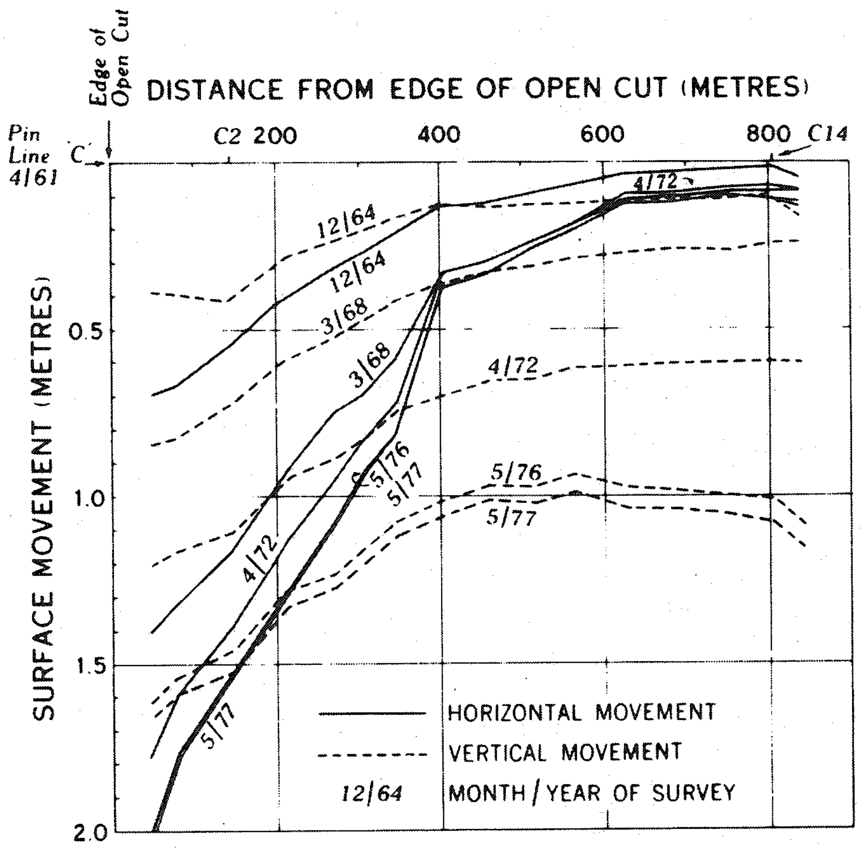


Figure 9.1.6 Horizontal movement and subsidence profiles on C line.

.

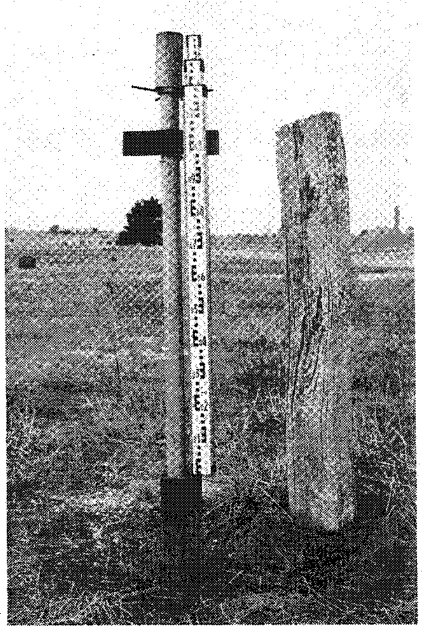


Figure 9.1.7 Protruding bore casing north of Morwell open cut.

geological structure, the significant factors influencing movements in the area around Morwell open cut are pressure relief and reduction in artesian and ground-water pressures.

As stated above, the major horizontal movements occur within a distance of some 400 m from the edge of the open cut and are considered to be due to pressure relief as well as to response to differential subsidence.

Subsidence near the open cut is influenced by pressure relief (inward and downward movement of batters), but regional subsidence is attributed mainly to consolidation of strata through increase of effective stresses resulting from the lowering of artesian water pressures. The lowering of the ground-water table by natural drainage through joints or horizontal bores drilled at toes of batters for lengths of up to 250 m also results in consolidation. However, such effects are not considered to extend beyond distances of 600 m from the open cut.

The relationships between subsidence, piezometric levels and flow rates are shown on Figure 9.1.8. Since 1960 the M1 and M2 aquifer pressure levels have been lowered by some 125 m and 120 m respectively in the area of the floor of the open cut. Dewatering of M2 aquifer was not commenced until 1970 and hence the 50 m reduction in piezometric level of this aquifer must have been achieved through upward leakage.

Although no significant horizontal movements have occurred at C14 (800 m north of open cut) this survey mark is subsiding at a rate similar to that at C2 (150 m north) where substantial horizontal movements have taken place. The regional character of subsidence is further illustrated by the steady lowering of Pin M158, located 5 km north of open cut, but where no horizontal movements have been recorded.

9.1.7 PREDICTED FUTURE MOVEMENTS

Future vertical movements in areas beyond the perimeter of the open cut have been estimated from consolidation theory on the assumption that drainage of batters and reduction of artesian pressures are the major factors contributing to subsidence. Fortuitously the effective stresses resulting from the dewatering will be lower than preconsolidation pressures. Hence, future subsidence will occur as a result of consolidation on the recompression portion of the field consolidation curves.

With the full development of Morwell open cut, ultimate settlement values are predicted to reach approximately double those which have occurred to 1977 in the main Morwell township area. Values of up to 3 m are expected at the southern edge of the town and 1 m at the northern boundary. Present subsidence contours will increase towards the west and the 3 m contour will be located west of the Morwell River (about 3.5 km west of the present floor of the open cut).

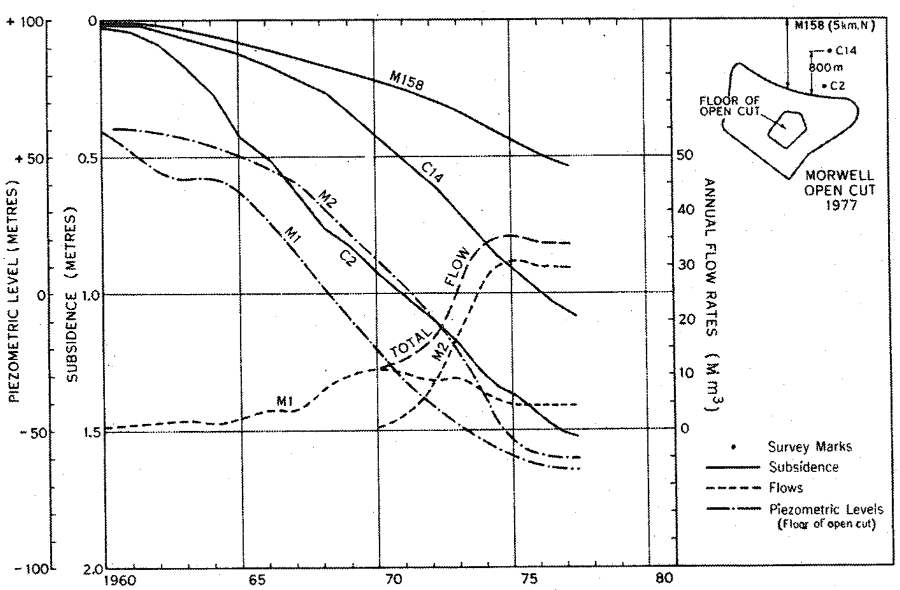


Figure 9.1.8 Relationship of piezometric levels and subsidence

Regional subsidence within the Latrobe Valley depression is likely to extend, both into the Moe Basin, a probable intake area, as well as to the east. The new major open cut at Loy Yang has a designed depth of 200 m and the required lowering of piezometric levels in this area is likely to result in even greater settlements than will occur at Morwell. Ultimately, the settlement "basins" at Morwell and Loy Yang are expected to coalesce.

9.1.8 SIGNIFICANCE OF MOVEMENTS

Structural damage due to earth movements is related more to the degree of horizontal strain and differential subsidence than to absolute values of movements. The strain values are determined from precise surveys of survey pins generally 60 m apart and, therefore, abrupt discontinuities which may have developed could be masked. One such feature has been detected on the ground and can be traced from the open cut for some 200 m into the southern limits of the township.

9.1.8.1 Effects on Morwell Township

Morwell township of 16 000 inhabitants extends to within 300 m of the northern edge of the open cut (Figure 9.1.5). Houses are mainly single storey and of timber or brick veneer construction with fibrous plaster lining. The commercial centre is located 1 km from the open cut and contains brick buildings of one or two stories. The recorded maximum value of north-south horizontal strain is less than 0.8 per cent. Strains are negligible over the northern portion of the township. Future strains in the southern township fringe are predicted to be about 0.5 per cent. Larger strains could occur in some localized areas and could affect houses and services.

Differential subsidence has rarely exceeded 0.3 per cent in Morwell township, and values are commonly less than 0.1 per cent over most of the area. With the predicted doubling of total subsidence in the township area, and the general differential subsidence values unlikely to exceed 0.3 per cent, it is concluded that differential subsidence will not significantly affect buildings and services within the township.

All relevant authorities in the township and district are fully aware of the history and amounts of movement occurring. Each receives copies of the annually revised earth movement survey data and the information is available to the public. A technical panel, consisting of two experienced geotechnical personnel from the State Electricity Commission, and a representative from the Commonwealth Scientific and Industrial Research Organization, Division of Applied Geomechanics, has been established to review claims for damage from owners of private property, and to assess whether the damage was due to open cut operations or to some other cause. A six-year period of surveillance has resulted in the detection of some minor cracking in concrete pavements and brickwork; however, to date no claims have been proved. It has also been shown that grades on drainage and sewage lines have not been significantly affected by earth movements.

9.1.8.2 Effects on engineering structures

Important engineering structures associated with the mining operations and power generation are located within the open cut and beyond the perimeter. These structures include dredgers, conveyor systems, pipelines, power stations, storage bunkers, cooling towers and water storages. Risks of damage within the open cut are reduced by the designed geometry of the excavation, and by the design of equipment to minimize the effect of movements. Beyond the perimeter of the open cut, power stations and associated structures are generally located 500 to 700 m from the edge of the open cut where future ground strains are estimated to be within acceptable limits.

9.1.9 CONCLUSIONS

Large vertical and horizontal movements have resulted from the development of deep and extensive open cuts in the brown coal deposits of the Latrobe valley.

Regional subsidence has been due to the reduction of artesian water pressures in aquifers underlying the coal seams, while horizontal movements are due mainly to pressure relief within the open cuts and are localized around each excavation.

Total, movements are expected to double as development of the Morwell open cut continues. Although the southern fringe of Morwell township could be affected by horizontal strain, serious problems are not anticipated. The regional subsidence is likely to be relatively uniform and the resulting low differential subsidence values should not affect buildings and services in Morwell township, nor major engineering structures outside the perimeter of the open cut.

Regular survey and surveillance programs will be continued in collaboration with the various authorities involved in the area, but no special measures to control or ameliorate subsidence are planned, other than to limit the reduction of artesian pressures to the minimum value consistent with safety of operations.

9.1.10 ACKNOWLEDGEMENTS

The material in this case history is published with the permission of the State Electricity Commission of Victoria. The investigations described were carried out in collaboration with Golder Brawner and Associates Ltd of Vancouver.

9.1.11 REFERENCES

BARTON, C. M. 1971. The Morwell interseam sands. J. Geol. Soc. Aust., 17, pp. 191-204.

GLOE, C. S. 1967. The lowering of the artesian water pressure surface in the vicinity of the Morwell Open Cut. Inter. Assoc. Hydrogeol. Cong. Hannover, 1965, vii, pp. 193-196.

GLOE, C. S., JAMES, J. P., and BARTON, C. M. 1971. Geotechnical investigations for slope stability studies in brown coal open cuts. Proc. Ist Aust-N.Z. Conf, Geomech. 1, pp. 329-336.

GLOE, C. S., JAMES, J. P., and McKENZIE, R. J. 1973. Earth movements resulting from brown coal open cut mining--Latrobe Valley, Victoria. Subsidence in Mines, 4th Ann Symp. Illawarra Branch, Australias. Inst. Min. Metall., 8 pp. 1-9.

GLOE, C. S. 1975. Latrobe Valley Coalfield, in Economic geology of Australia and Papua-New Guinea. 2. Coal. Australias. Inst. Min. Metall., pp. 345-359.

GLOE, C. S. 1977. Land subsidence related to brown coal open cut operations, Latrobe Valley, Victoria, Australia. Second International Symposium on Land Subsidence. Proc. Anaheim Symp. 1976, IASH-Unesco, pp. 399-407

HUTCHINGS, R., FAJDIGA, M., and RAISBECK, D. 1977. The effects of large ground movements resulting from brown coal open cut excavations in the Latrobe Valley, Victoria. Large ground movements and structures. Conf. Cardiff, 1977 (in print).

Case History No. 9.2. Shanghai, China, by Shi Luxiang and Bao Manfang, Shanghai Geological Department, Shanghai, China

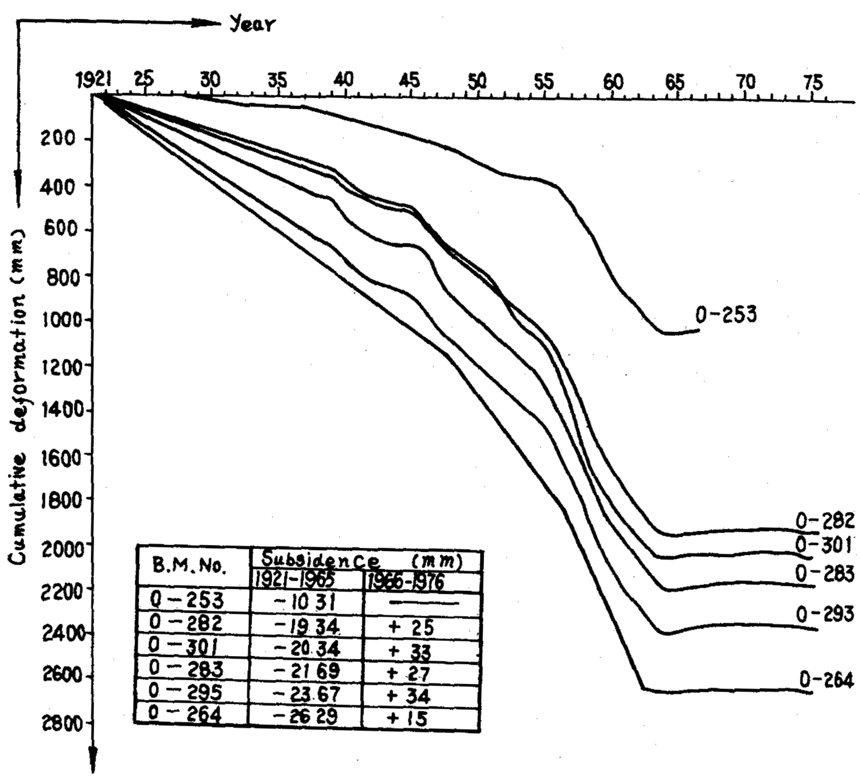
9.2.1 INTRODUCTION

Shanghai is the largest industrial city in China, standing on the coast of the East China Sea and situated at the front edge of the Yangtze Delta. The elevation[1] of the flat-lying city area is 3-4 metres above sea level. The summer-winter temperature variation is very large. The Whangpoo River and Soochow Creek, both being the outlets of Taihu Lake, are the chief tide waterways of the city.

Land subsidence was first reported in 1921. After liberation, with the rapid development of industrial production, the exploitation of ground water increased, and subsidence continued. The greatest subsidence occurred from 1956 to 1959, at an annual rate of 98 mm. Up to 1965, the maximum cumulative subsidence, as indicated by one of the bench marks in the city area, was as high as 2.63 m (Figure 9.2.1). The area of cumulative subsidence exceeding 500 mm was 121 km^2, forming two plate-shaped depressions in the urban district and affecting the suburban districts, too.

____

[1]"Elevation" in this paper is based upon the Wusong datum.



B.M. No.	Subsidence (mm)	
	1921-1965	1966-1976
0 - 253	-10 31	———
0 - 282	-19 34	+ 25
0 - 301	-20.34	+ 33
0 - 283	-21 69	+ 27
0 - 295	-23.67	+ 34
0 - 264	-26 29	+ 15

Figure 9.2.1 Cumulative deformation shown by some typical bench marks in the urban area of Shanghai. +, rebound; -, subsidence

After 1963, antimeasures were taken against land subsidence. In 1965, the annual rate of subsidence in the urban area was reduced to 23 mm. From the research results of the preceding period, calculations for the relations among pumpage, water level, and subsidence for the year of 1966 were made in 1965, and the scheme for planning exploitation for the year of 1966 was drawn up. The exploitation of factories followed the plan, and therefore, in 1966, annual rebounding of 6.3 mm occurred in the urban area.

9.2.2 GENERAL GEOLOGICAL CONDITION OF THE OVERBURDEN

In the Shanghai area, unconsolidated materials, about 300 metres thick, of alternating marine and continental facies were deposited on the bedrock during the Quaternary Period. The upper portion of 150 m is composed of clayey soil and sand of littoral and fluvial delta facies; the lower portion of 150 m consists of alternating sand layers of fluvial and variegated clays of lacustrine facies.

Based on the hydrogeological characteristics of the overburden, one water-bearing layer and five aquifers may be identified (hereinafter called aquifers, Figure 9.2.2). The general features of these aquifers are: flat-lying, thick, fine-grained, with small hydraulic gradient and low velocity of ground-water flow. These aquifers are marked by a distinct regularity of lithological changes, finer grained with decreasing thickness from northeast to southwest. Aquitards are widely distributed, only absent in the eastern part, or in local areas along the Whangpoo River, thus bringing about direct hydraulic interconnections between the first, second and third aquifers.

According to its engineering-geological character, the overburden may be divided into 13 layers. Among them are three stiff clay layers below the second aquifer, with fairly high compressive strength; their void ratio is less than 0.70, and their coefficient of compressibility less than 0.025 cm^2/kg. The amount of compression of the layers is comparatively small, as shown in the surveys made over the years. Above the second aquifer are three compressible layers (soft clay layers) with low compressive strength and one dark-green stiff clay layer with fairly high compressive strength. The void ratio of these layers, their water content, coefficient of permeability, coefficient of compressibility, and other principal physical mechanical indices decrease as the depth of the layers increases (Figure 9.2.3).

9.2.3 RELATIONSHIP BETWEEN GEOLOGICAL STRUCTURE AND LAND SUBSIDENCE

According to geological surveys and analytical studies of the observation data, land subsidence principally occurred in the upper layers of the overburden. To present in full the factors causing subsidence, the urban area of Shanghai may be divided into four geological structure areas of land subsidence, based on the different combinations, from the depth of 70 m upward, of three compressible layers and one dark-green stiff clay layer, and based on their relationship with the lst and 2nd aquifers (Figure 9.2.4).

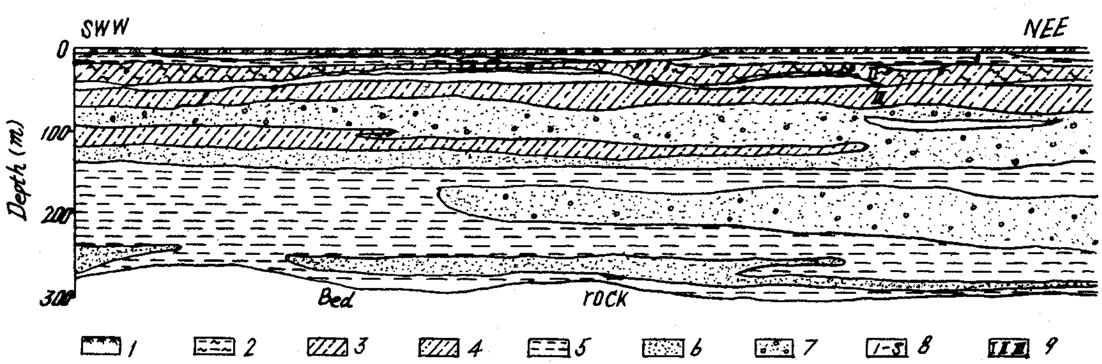


Figure 9.2.2. Geological profile of the urban area of Shanghai. 1, surface soil; 2, muddy clay; 3, muddy clayey loam; 4, clayey loam with sand; 5, stiff clay; 6, sand; 7, sand with gravel; 8, confined aquifer; 9, compressible layer.

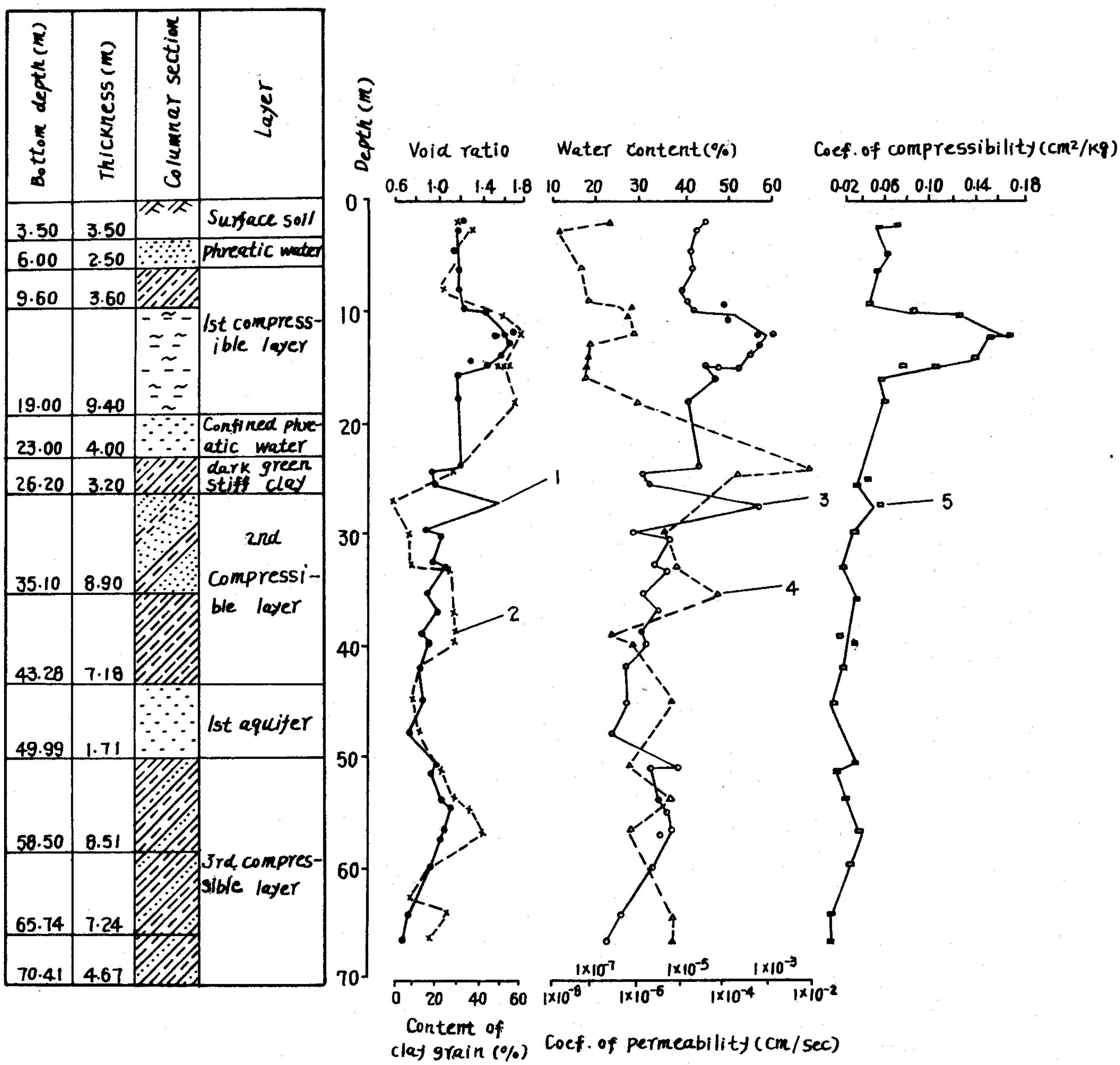


Figure 9.2.3 Variations of main physical-mechanical properties of soil layers with depth. 1, void ratio; 2, content of clay particle; 3, water content; 4, coefficient of permeability; 5, coefficient of compressibility. Legend of columnar section as shown in Figure 9.2.2.

<ins>Area No. l(I)</ins>. Consisting of the lst compressible layer, the dark-green stiff clay layer and the lst and 2nd aquifers. Due to the thin compressible layers and the thick sand layers in addition to the dark-green stiff clay layer, the rebound of land surface was comparatively great after measures were taken.

<ins>Area No. 2(II)</ins>. Consisting of the lst and 3rd compressible layers, the dark-green stiff clay layer and the lst and 2nd aquifers. Owing to the fact that the 3rd compressible layer is comparatively thick, the cumulative subsidence was relatively greater than that in Area No. 1 before measures were taken. However, this is also an area in which the rebound is comparatively great after taking the measures.

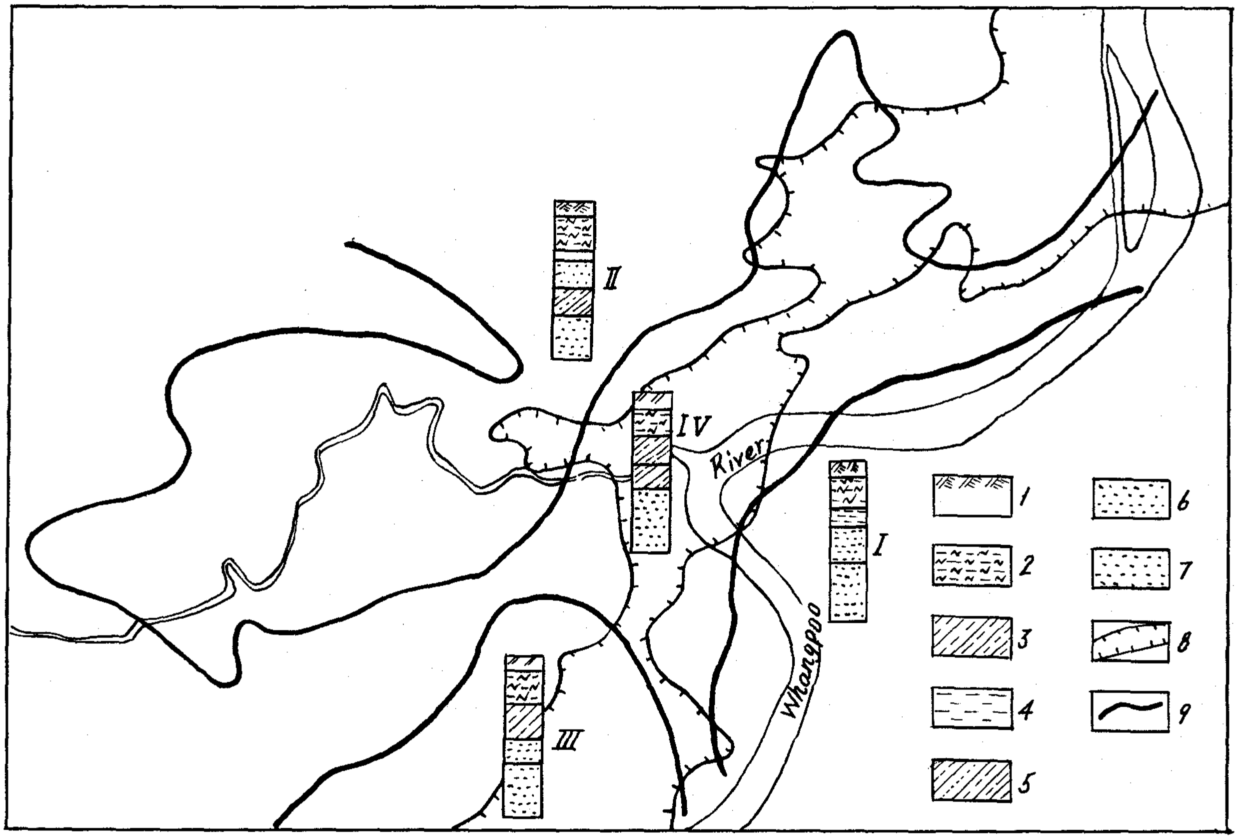


Figure 9.2.4 Relationship between the geological structure and land subsidence in urban area of Shanghai. 1, surface soil; 2, lst compressible layer; 3, 2nd compressible layer; 4, dark-green stiff clay layer; 5, 3rd compressible layer; 6, lst aquifer; 7, 2nd aquifer; 8, slight subsidence within the boundary line, rebound area outside the boundary line; 9, geological structure area and boundary line.

Area No. 3 (III). Consisting of the lst and 2nd compressible layers and the lst and 2nd aquifers. Due to the absence of the dark-green stiff clay layer and the existence of the direct hydraulic interconnection between the lst and 2nd aquifers, the cumulative subsidence was comparatively great before we adopted the methods for improvement. Since the measures were taken, the rate of subsidence has slowed down, though there is still slight subsidence in the lst and 2nd compressible layers.

Area No. 4 (IV). Lying in the central part of the city in a NE-SW direction. This is largely an unexploited and unrecharged area. It comprises the lst, 2nd and 3rd compressible layers and the 2nd aquifer. Since the three compressible layers are thick and there is no dark-green stiff clay layer, the cumulative subsidence had been comparatively great before remedial measures were applied. After we had taken these measures, with the recovery of the ground-water level, the third compressible layer was no longer under compression and began to swell. However, continuation of slight compression of the first two compressible layers has been observed. The geological structure of this area is generally weak.

From the exploration data, we recognized as follows:

1. In areas with the same geological conditions of ground-water exploitation, the amplitude of subsidence varies with the thickness and the compressibility of the compressible layers. Generally, the greater the thickness and the compressibility, the greater the subsidence.

2. The amount of compression of the lst compressible layer depends upon whether the dark-green stiff clay layer is present in the lower part. The subsidence in Areas Nos. 3 and 4, where there is no dark-green stiff clay layer, is greater than that in Areas Nos. 1 and 2 where the dark-green stiff clay layer is present.

3. The rate of compression of the lst and 2nd compressible layers depends upon whether hydraulic interconnection exists between the lst and 2nd aquifers. Therefore, the rate is greater in Area No. 3 where the hydraulic interconnection exists than in Area No. 2 where it does not.

It may be understood that in the course of history, the soil layers have been under the action of fairly low water head. From the relation of preconsolidation pressure of the various soil layers, we know it is equivalent to the action of preloading. Therefore, each layer has its own preconsolidation pressure p_C and preconsolidation ratio $c_r = P_C/P_O$, in which P_O is the overburden pressure). From the depth of the soil layers above 70 m, p_C values are worked out by laboratory tests. We know that the average preconsolidation pressure p_C of the lst and 2nd compressible layers approaches the overburden pressure p_0 of the overlying soil layers. It may be the normal consolidation layer. The 3rd compressible layer is 40-70 m below the surface, and its p_C and c_r increase with depth. The compression of the 3rd compressible layer (over-consolidated) will not occur if the average $(p_C - p_O) \approx 1$ kg/cm^2, i.e., if the drawdown of the ground-water level in the 2nd aquifer does not exceed the average $(p_C - p_0)$ value while the measures controlling land subsidence are being taken.

9.2.4 CALCULATION

According to the mechanism of deformation of the soil layers under the pumping drawdown, it is suitable to apply a one-dimensional equation to calculate the compressive deformation of the soil layers. The drain path of the lst, 2nd and 3rd compressible layers is mainly upward and downward toward the sand layers. Although the ground-water cone exists, yet it has a wider range and small hydraulic gradient. The difference of horizontal water pressure is not great, so the transverse drainage may not be considered.

This principle has been applied to the annual calculation since 1965, and the results have been checked the next year in order to make a comparison with the practice to correct the calculated indices year by year. The calculated value and practice would gradually correlate with each other.

In 1972, according to the elastic-plastic characteristics of deformation of soil layers and the principles of soil mechanics, a physical model was developed. Then through the statistical analysis of a large amount of observed data of deformation, which were influenced by the variation in elevation of the ground-water level under a certain pumping and recharging condition, a simplified mathematical model was developed, in order to get the optimizing numerical solution with computer. Consequently, the forecast and prediction of land subsidence can be made.

Accuracy of calculation has been checked through practice. The period of prediction for scheme calculation is one year. The calculation has been made since 1972. The practice has demonstrated that the maximum error of predicting elevation of water head was ± 0.5 m, when the annual amplitude of water head was less than 6-8 m; the average error of predicting absolute deformation was below 1.7 mm, when the annual average amplitude of deformation was less than 14.3 mm.

9.2.5 MEASURES

The purpose of land subsidence research is to solve the problem of land subsidence; therefore, it is the key of the question to take measures.

The main cause of land subsidence in Shanghai is intensive exploitation of ground water and corresponding drawdown of water level. In order to control land subsidence and to rationalize the exploitation of ground water, the measures for recovering and raising up the ground-water level have been taken on the basis of analysing and researching the laws of land subsidence, according to Shanghai's specific conditions. Preliminary control of subsidence of the urban area has been achieved. The measures taken are chiefly as follows:

1. Restricting and rational usage of ground water.

   Owing to the great disaster made by the land subsidence, in 1963 it was resolved that measures for restricting ground-water exploitation were to be taken by Shanghai Municipal Government. Ground water in Shanghai is mainly utilized to reguate the air and lower the temperature, so the paper, iron and steel industries which do not need cooling energy from the ground water, have changed to using surface water. In addition, some factories have installed refrigeration equipment instead of using ground water. Cooling energy from this equipment may be equivalent to the cooling energy provided by 200 deep wells. Since 1965, the calculation of land subsidence has been made annually, thus giving the planning exploitation scheme for the next year. And the factories, according to the plan for rational usage of ground water, have taken measures for multiple and comprehensive utilization of ground water.

2. Artificial recharge of ground water.

   The "measures of recharging in winter for summer use and recharging in summer for winter use" have been employed since 1965. The textile mills which need cooling energy can use lower temperatures in summer. From the practice over years, it was proved that after recharging, the temperature of ground water of the 2nd and 3rd aquifers gradually decreased by 6-10° C in comparison with the original, and that of the 4th and 5th aquifers also decreased by 9-10° C. Besides, recharging in winter for summer use and recharging in summer for winter use had the function of raising the ground-water level in order to diminish land subsidence.

3. Adjustments of exploited aquifers.

   Before the measures had been taken, ground water was mainly pumped from the 2nd and 3rd aquifers in the urban area. Although the 4th aquifer contains abundant ground water of fairly good quality, the original temperature is high, so it is not suitable for cooling. Hence the pumpage from the 4th aquifer is small. Due to the irregular distribution of the 5th aquifer and the high temperature of its ground water, pumpage from this aquifer also is small. However, after recharging cool water in winter to these aquifers, the temperature of ground water in them decreases and the water can be used for cooling purposes. Therefore, pumpage from the 2nd and 3rd aquifers has been decreased, and the use of ground water from the 4th and 5th aquifers has been increased. On the other hand, the strength of the soil layer underneath the 3rd aquifer is fairly high, and deformation is not obvious after the exploitation of the 4th and 5th aquifers. This will decrease the rate of land subsidence.

9.2.6 REFERENCES

SHANGHAI HYDROGEOLOGICAL TEAM. 1973. On the control of surface subsidence in Shanghai. Acta Geologica Sinica 1973, No. 2.

SHANGHAI HYDROGEOLOGICAL TEAM. 1976. Preliminary studies on land subsidence of the urban area of Shanghai, China. Not published.

Case History No. 9.3. Venice, Italy, by Laura Carbognin, Paolo Gatto, and Giuseppe Mozzi, National Research Council, S. Polo 1364, Venice, Italy; Giuseppe Gambolati, IBM Scientific Center, S. Polo 1364, Venice, Italy; Giuseppe Ricceri, Department of Soil Mechanics, University of Padua, Italy

Reprinted from IAHS-AISH Publication No. 121, 1977, p. 65-81, by permission.

9.3.1 INTRODUCTION

Many areas of Italy are affected by land subsidence. Among these, the area of Venice (Figure 9.3.1) has caused the greatest concern. Its sinking in fact, in spite of the relatively small rate, could be fatal, due to the low level of the city in relation to the sea. The well-known floods (or "acque alte," a local idiom meaning high waters), essentially caused by weather and astronomical factors, are indirectly enhanced by subsidence both in amplitude and in frequency. When the studies were started, it became quite clear that, out of the various factors responsible for the sinking, the withdrawal of underground water was the main one. Thus, after a preliminary analysis, the research effort was mainly directed to hydrogeology.

In 1969, the Italian Consiglio Nazionale delle Ricerche (CNR, National Research Council) constituted a working group for the Venice problem. Starting that year an accurate inventory was made of the data already available but widely scattered. They were filed according to

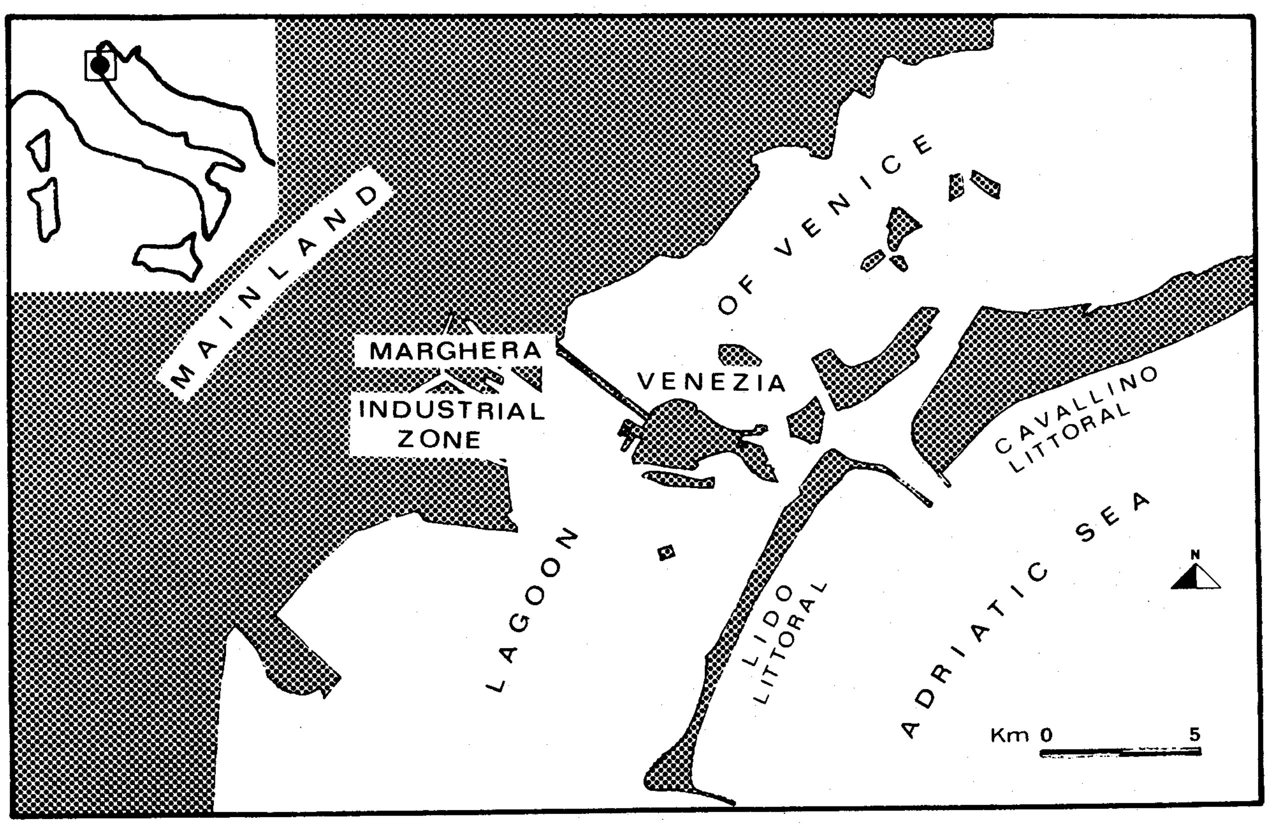


Figure 9.3.1 Map of the Venetian area under investigation.

lithostratigraphy, hydrology, geotechnique, and geodesy. During their processing, specific experimental tests were performed, to validate and supplement the preliminary reconstructions.

The analysis was given two major aims: first, to describe the physical environment where the phenomena under study occur; next, to describe their evolution, to investigate their mechanism, and to make predictions with numerical models. The final results of the research confirm the dependence of the subsidence on the artesian withdrawals, the possibility of stopping the settlement of the city, and even of obtaining a slight rebound by naturally recharging the depleted aquifer system.

9.3.2 THE PHYSICAL ENVIRONMENT

The Venice confined system, down to 1000 m depth (Quarternary basement), is constituted by sand layers (aquifers) bounded by silt and clay layers (aquitards). Moving northwest, towards the foothills of the Alps, the sedimental structure tends to change. Materials are more and more coarse, while the aquitards become thinner, and, at a certain point, they disappear. In the foothill belt the unconsolidated mantle is a whole homogeneous system of sand and gravel. For the hydrologist, it represents the reservoir supplying the aquifer-aquitard system extending beneath Venice and even further.

The Venetian aquifer system has been investigated in detail by taking information from both the existing artesian wells (Alberotanza, et al., 1972) and a new deep test borehole, VE 1 CNR, where continuous samples of the Quarternary series were taken (Consiglio Nazionale Ricerche, 1971). Thousands of analyses were performed on the borehole samples, and a complete physiography of the local subsurface formations was obtained. From this drilling more complete interpretation of the scattered information was made possible. Moreover, the starting point was available for the definition of the hydrogeological stereogram of the region. Figure 9.3.2 is a map of the upper 350 m, where the aquifers are pumped (after Gambolati, et al., 1974, slightly modified). Six aquifers appear, four of which are extensively exploited (2nd, 4th, 5th and 6th).

Permeability, grain-size and clay chemistry of aquifers and aquitards are reported in tables 1, 2 and 3, whose values were obtained by analyzing the cores of the VE 1 CNR and two other test boreholes LIDO 1 and MARGHERA 1.

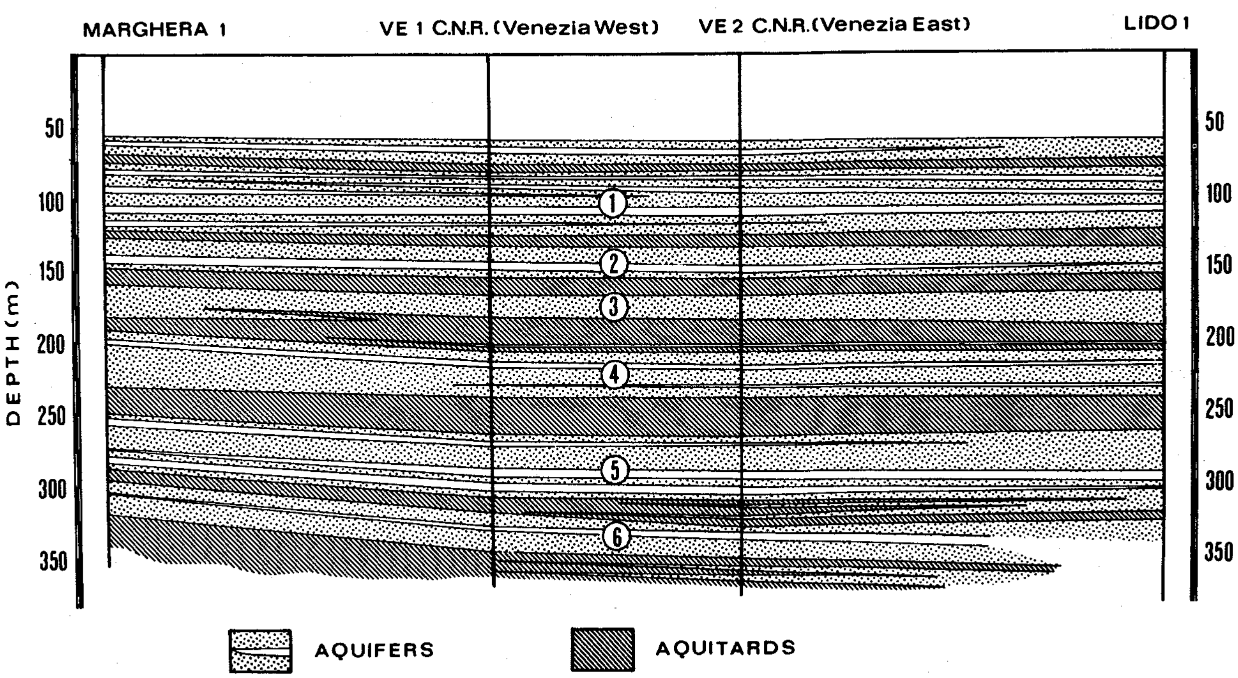


Figure 9.3.2 Hydrogeological map of the confined aquifer system updated using the electric logs recorded in deep test boreholes.

Table 9.3.1 Average permeability of samples taken from VE 1 CNR Borehole (placed in Venice), from laboratory tests.

Depth (metres)	Aquifers (average horizontal permeability)	Aquitards (average vertical permeability)
74- 81		3×10^{-5} cm/sec
81-124	1×10^{-3} cm/sec	
124-132		7×10^{-8} cm/sec
132-153	1×10^{-3} cm/sec	
153-163		3×10^{-6} cm/sec
163-181	4×10^{-5} cm/sec	
181-203		5×10^{-7} cm/sec
203-235	6×10^{-4} cm/sec	
235-260		6×10^{-7} cm/sec
260-302	2×10^{-4} cm/sec	
302-318		6×10^{-6} cm/sec
318-340	10^{-6} cm/sec[1]	

[1] The 6th aquifer is exploited only at Marghera, since at Venice its permeability is too low.

Table 9.3.2 Summary of grain size analysis as measured in the laboratory on samples taken from the VE 1 CNR borehole (after Gambolati, et al., 1974).

Depth	Lithotypes			
(metres)	Coarse fraction	Sands	Silts	Clays
0- 50	0.3	38.0	41.7	20.0
51-100	0.7	50.0	35.0	14.3
101-150	---	46.2	42.2	11.6
151-200	0.4	33.6	48.2	17.8
201-250	---	26.0	54.0	20.0
251-300	5.6	38.4	34.8	21.2
301-350	---	13.5	61.6	24.9
Average	1.0	35.1	45.4	18.5

Permeability (Table 9.3.1) was defined by laboratory tests performed only on clean sand for aquifers and silt clay for aquitards.

The prevailing fraction (Table 9.3.2) is silt, followed by sand and clay.

The illite is dominant (Table 9.3.3); instead the most plastic one, montmorillonite, is in general rather scarce, and its relative abundancy grows towards the historical center and Lido.

Some details of the mechanical properties of these soils are given here (see a recent and more complete paper by Ricceri and Butterfield, 1974). The values of the compressibility coefficient ($m_V=(\Delta e/\Delta P)(l/l+e_o)$) versus depth (Figure 9.3.3) have been computed by oedometric tests at the actual "in situ" pressure (p_o) in the loading (m_{V1}) and unloading (m_{V2}) curves. The maximum load attained in these tests was $5 \div 20 p_o$ for the samples coming from the upper 100 metres and twice the values of p_o for the others, In Figure 9.3.3, solid lines connect the values m_{V1}, and m_{V2} for each sample; dashed lines refer to oedometric tests where loads were increased slightly above p_o and then gradually reduced to zero. The two coefficients decrease with increasing depth. In particular, m_{V2} seems rather insensitive to the maximum load applied in the test, and its average value is about 20 per cent of M_{V1}.

The reader is referred to the bibliography for further information about the physical aspects of the Venetian formations.

Table 9.3.3 Percentages of various clay-types in the core samples taken from MARGHERA 1, VE 1 CNR, and LIDO 1 test boreholes (after Mozzi et al., 1975).

Clay Minerals	Test boreholes			Average
	MARGHERA	VENICE	LIDO	
Illite	48.75	48.45	48.00	48.40
Chlorite	33.75	28.00	30.00	30.58
Kaolinite	11.25	12.80	9.00	11.02
Montmorillonite	6.25	10.75	13.00	10.00

9.3.3 HISTORY OF THE PHENOMENA

By comparing the development of the artesian exploitation and of subsidence, three periods appear distinguishable: the first before 1952, the second from 1952 to 1969 and the last afterwards.

In Figure 9.3.4 the average piezometric level in different places of the Venetian area is plotted versus time.

9.3.3.1 Period before 1952

When the artesian exploitation was not very intensive, subsidence was due only to natural causes; its rate was about one millimetre per year (Leonardi, 1960; Fontes and Bortolami, 1972).

The extraction of the artesian water began about in 1930 when the first factories were established in Marghera. The piezometric level remained above the ground level, except in Venice, where it became lower since the time of World War II. The average decrease was slow all over the area, up to the fifties, when an intensive exploitation started, due to the strong industrial development (Figure 9.3.4).

9.3.3.2 Period between 1952 and 1969

After 1950 the changes became more evident. Artesian water was very actively withdrawn (Serandrei Barbero, 1972) and in the fifties all the hydraulic heads declined below the surface. In the industrial area the average rate reached 0.70 m/y, which is definitely higher than in any other part of the area (Figure 9.3.4). The observed minima were attained in 1969; in Marghera the fourth and fifth aquifers went down to 16 m below surface and in Venice the third and fifth went to 7 m below. From 1952 to 1969, as an average in the industrial zone, a hydraulic head loss of more than 12 m was recorded. In Marghera the withdrawal occurred in about 50 wells, and it was about 460 l/s in 1969; in Venice there were about 10 active wells, but in fact only one (10 l/s) represented the whole extraction of the city. A significant ratio of 1 to 50 existed between the exploitation in the historical center and in Marghera.

In the period 1952-1968 geodetic surveys showed an average subsidence of 6.5 mm/y in the industrial area and 5 mm/y in the city. The most alarming figures appeared between 1968 and 1969, where maxima were observed of more than 17 mm in Marghera and 14 mm in Venice (Caputo et al., 1971) (Figure 5). Overall between 1952 and 1969 the local average subsidence was over 11 cm in the industrial zone and about 9 cm in the city, with local maxima of 14 and 10 cm respectively (Figure 9.3.5)

9.3.3 Years between 1969 and 1975

These years are characterized by a great number of experimental data and theoretical studies worth describing.

After drilling the deep test hole, VE 1 CNR, previously mentioned, two important steps were carried out: the annual repetition of the geodetic survey for controlling the ground movement in the area and the installation of a network of 112 piezometers (24 of which were continuously recording) for controlling the six exploited aquifers (Figure 9.3.6). Therefore it was possible

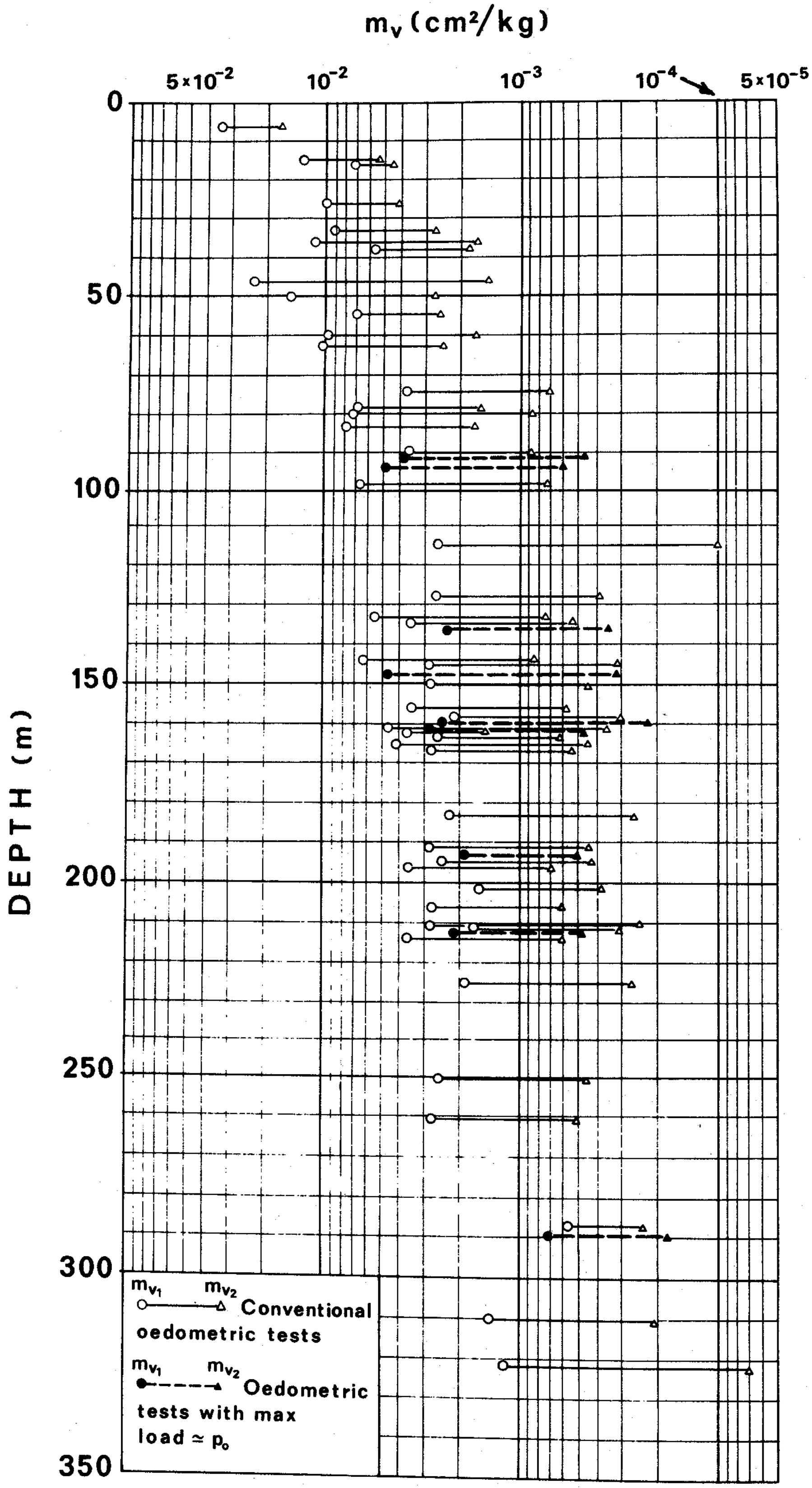


Figure 9.3.3 Coefficients of compressibility m_{v1} and m_{v2} versus depth.

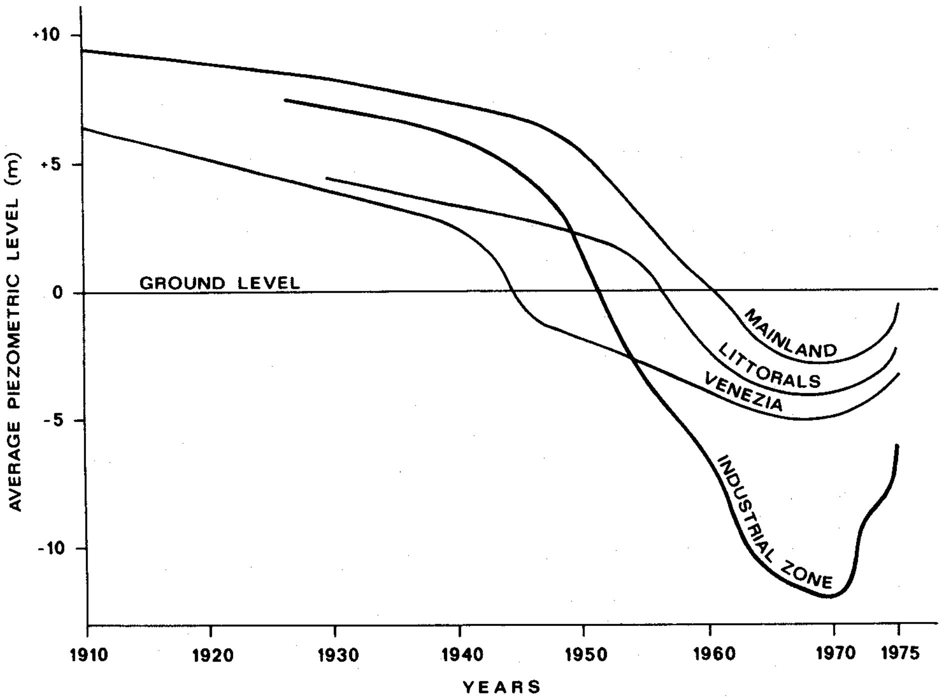


Figure 9.3.4 Average piezometric levels from 1910 to 1975

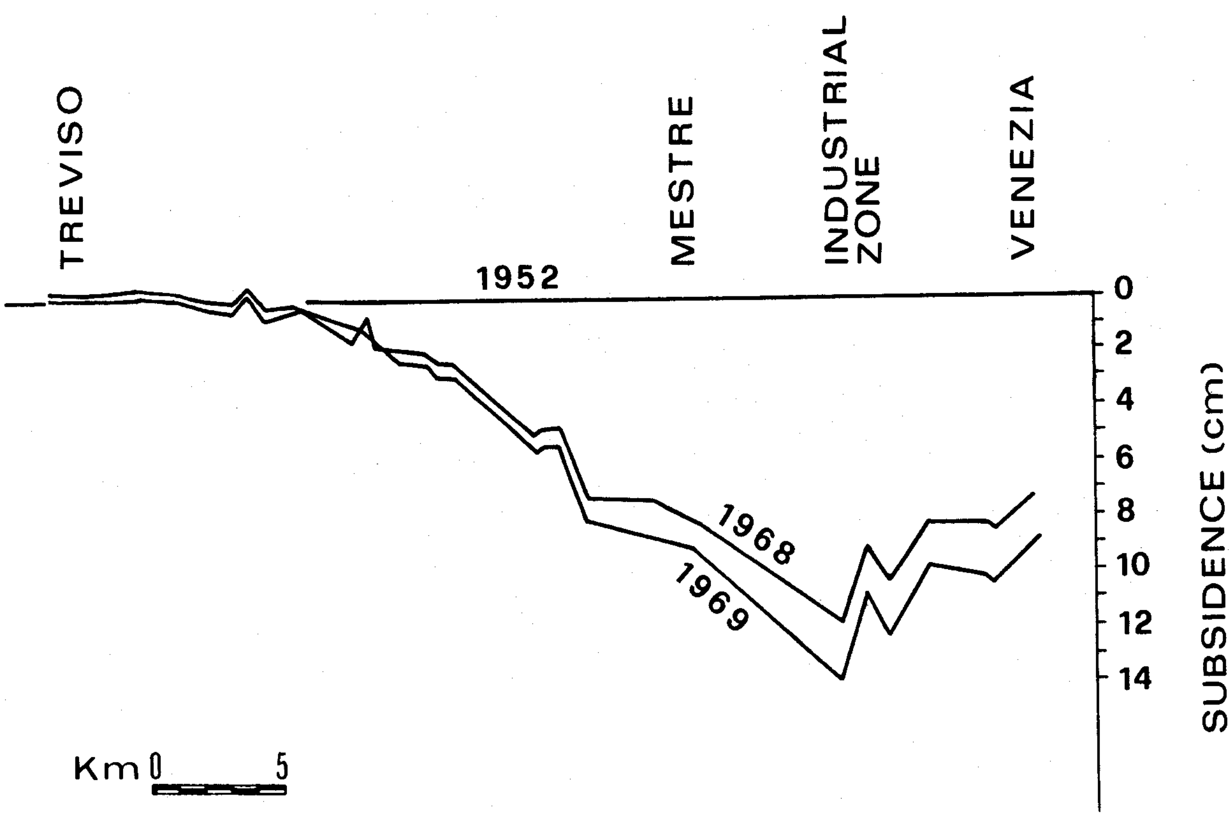


Figure 9.3.5 Comparative plot of the levellings in 1968 and 1969 as referred to 1952

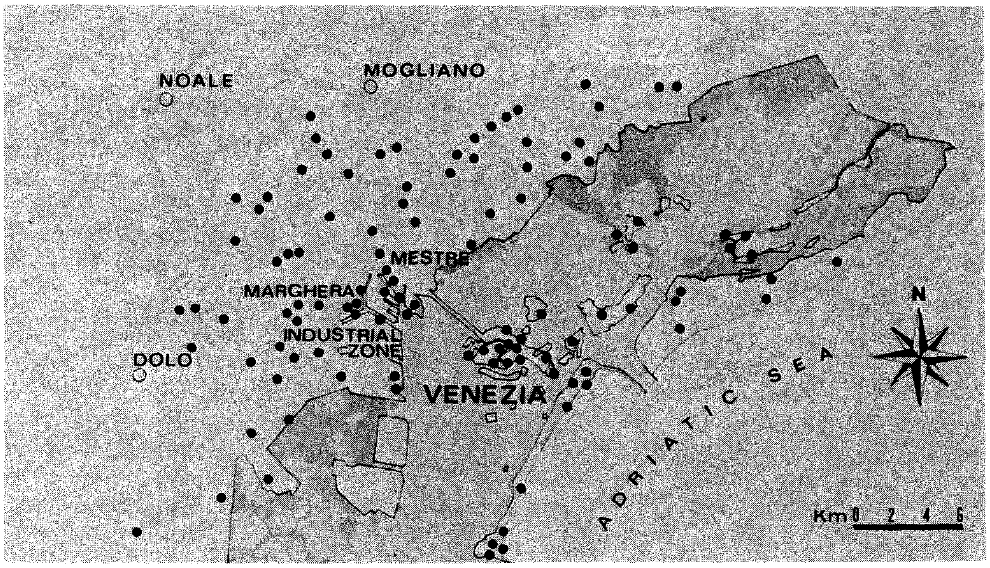


Figure 9.3.6   Map of the 112 piezometers

to annually reconstruct the altimetric profiles and the maps of the equipotential lines (e.g., see Figure 9.3.7 which refers to the 5th aquifer for 1975. Similar behaviour holds true for the previous years).

In general, after the minima recorded in 1969, one can observe a gradual and remarkable, improvement in the piezometric surfaces. In 1975, the average recovery in the industrial area reached a maximum of over 8 m, and in Venice more than 3 m. This new behaviour can be seen in Figure 9.3.4 and it is also well shown in Figure 9.3.8, where the progressive reduction of the depressurized area in recent years is evident.

Similarly to what happened when piezometric levels were declining, a ground-surface rebound is now accompanying the piezometric recovery. After a stability period, which is evident from the 1973 survey (Folloni et al., 1974), the 1975 levelling shows a rebound of the land which, in the historical center, is more than 2 cm with respect to 1969 (Figure 9.3.9). Even taking into account the range of the errors affecting the altimetric curve (Gubellini and DeSanctis Recciardone, 1972), the variation of the ground level in this area remains positive. This is consistent with what appears in the tidal records in Rovinj and Bakar (on the Yugoslavian coast, which is taken to be stable) and those in Venice. Until 1969, the average annual sea level recorded at Venice was apparently increasing with respect to that of the other two stations. In recent years this did not occur any more (Tomasin A., private communication based on official data).

9.3.4 DISCUSSION

9.3.4.1 Analysis of experimental data

We will now analyze the most recent data, i.e., those from the period when the phenomena show a reverse trend.

Looking at the isopiezometric maps, we noted that

1. the piezometric surfaces of the aquifers in the Venetian area show strong depression, assuming the shape of an inverted asymmetrical cone typical of localized pumpage (Mozzi et al., 1975);
2. the maximum drawdown in all the aquifers occurs in the Marghera area, which appears as the main withdrawal center. Minor discrepancies are seen in the islands of Murano, Burano, Le Vignole and Lido:
3. the greatest depressurization is found in the 4th and 5th aquifers;

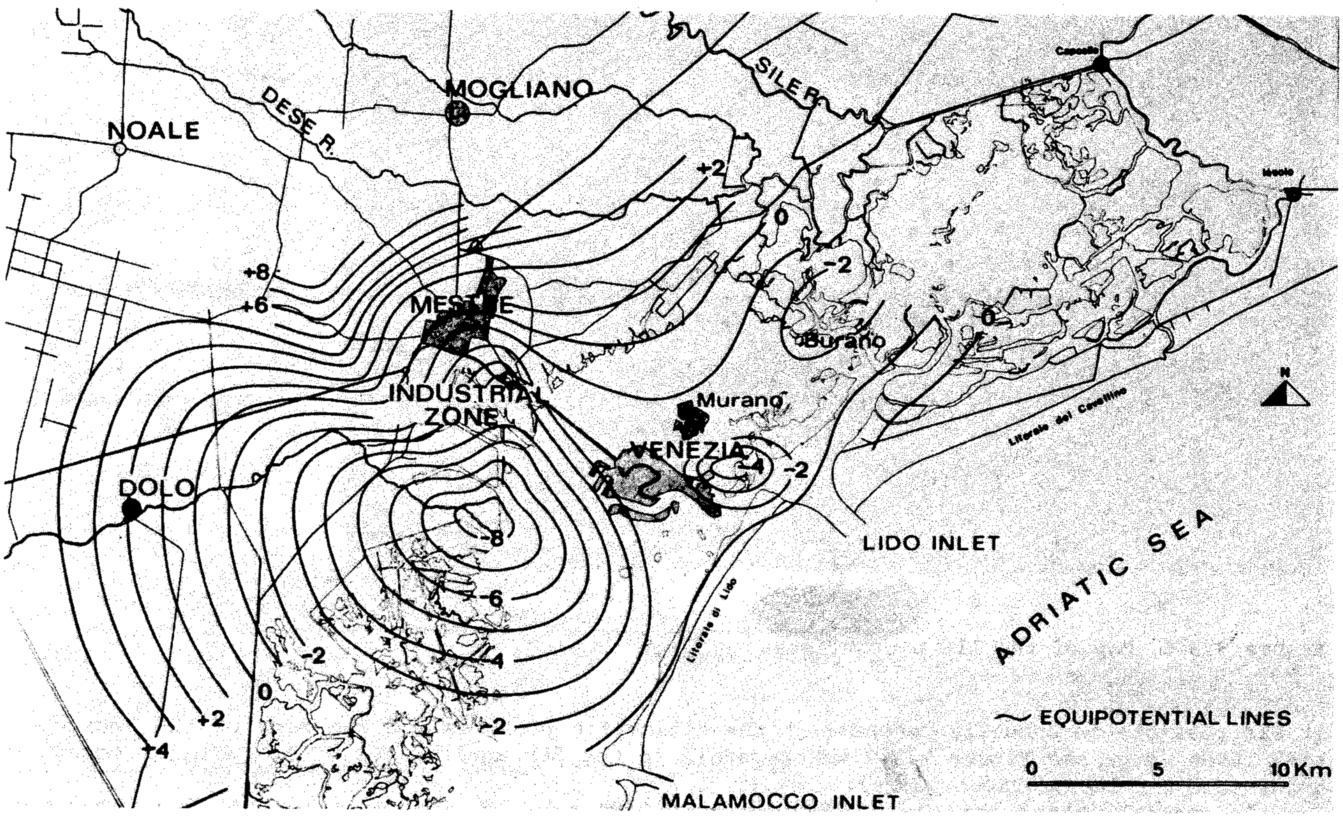


Figure 9.3.7 Piezometric surface of the 5th aquifer in 1975. Equipotential lines are given in metres a.s.l.

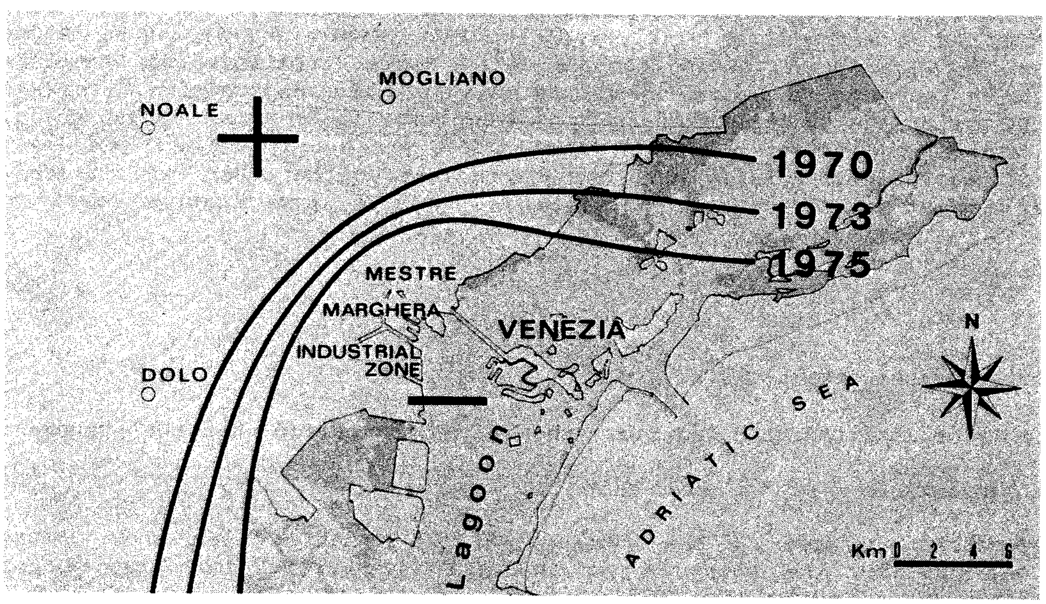


Figure 9.3.8 Boundary of the areas where average piezometric level is above (+) or below (-) the ground level in 1970, 1973 and 1975.

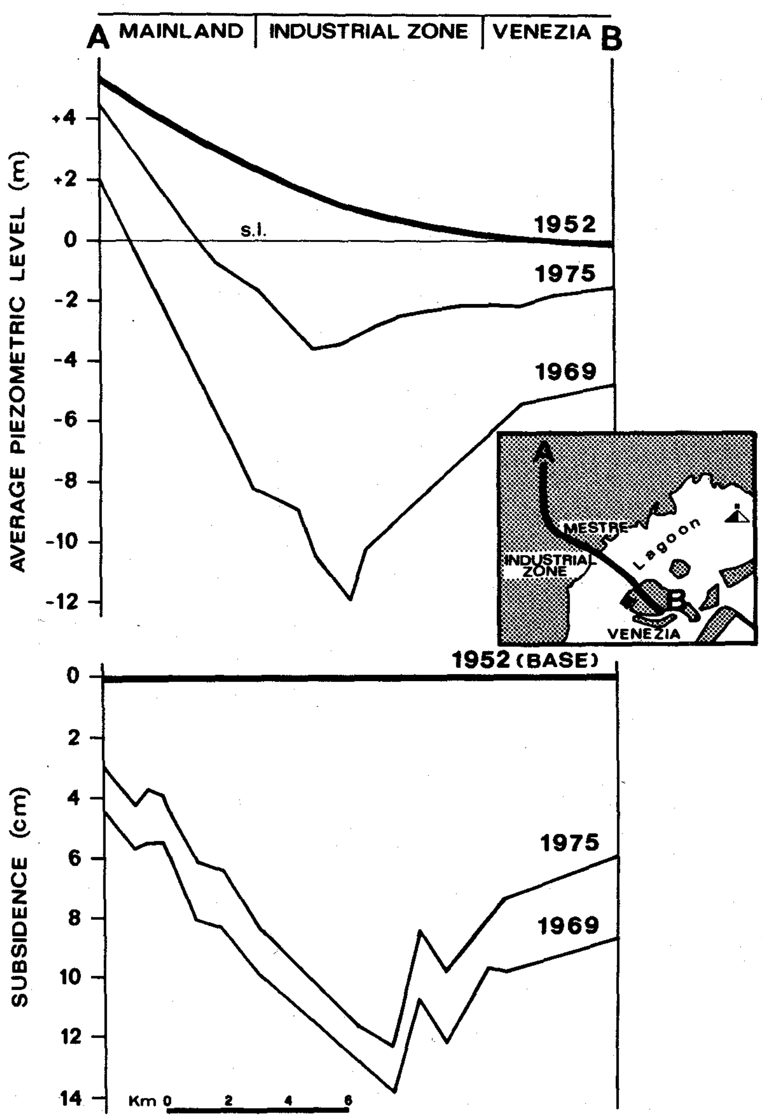


Figure 9.3.9 Comparison of the average piezometric levels and ground levels over mainland (A) and Venice (B).

4. the development of the equipotential lines shows that pumpage at Marghera affects the natural hydraulic balance of the aquifers also in the historical center, where the local withdrawals do not account for the observed drawdown;
5. the distance between equipotential lines gets smaller landward. Figure 9.3.7 also suggests that a no-flux boundary condition exists seaward. This is in keeping with the reconstructed geology.

The aquifer recovery is due to a decrease in the water exploitation. Since 1970, some areas in the district have been supplied by the public aqueduct. The industrial activity of Marghera has been reduced. Above all, well drilling was prohibited in the Venetian plain. In January 1975, the new industrial aqueduct, supplied by the Sile River, was put into operation (a 60 per cent reduction in the number of active wells was observed in Marghera from 1969 to 1975, when the withdrawal was estimated to be about 200 l/s).

The raising of the hydraulic levels is certainly not due to the increased recharge of the aquifers, since in the last decade the natural water supply in the recharge area is diminishing (Carbognin et al., in press).

The levellings, as already stated, show the cessation of the subsidence and a certain rebound. The close connection between withdrawal and subsidence is evident in Figure 9.3.9, where the altimetrical variations and the average piezometric level variations are given for the periods 1952-69 and 1969-75, along the same section from the mainland to Venice. Graphical comparison visualizes the presence of minima in the industrial zone, and similar behaviour of the processes during exploitation and recovery. In the rebound phase, however, we notice that while at Marghera a strong recovery determines a slight altimetrical rebound, at Venice a minor piezometric recovery causes a greater rebound. This can be ascribed to the diverse nature of the cohesive soils at Marghera and Venice.

The assumption of the interdependence between the piezometric and the altimetric variations was statistically verified. In fact, the linear correlation coefficient, with a 95 per cent probability, is between 0.70 and 0.92. The connection between the two variables is therefore expected to be extremely high. Consequently, the coefficient of determination indicates that the piezometric variations account for 70 per cent of the altimetric ones, in terms of variance and in the limiting hypothesis of linear behaviour. The residual variance must be explained by other factors, such as natural subsidence and loading by buildings, but also errors in measurements and deviation from the linear hypothesis.

The interpretation of the subsidence to piezometric variations ratio ($R = \eta/\Delta h$) is also interesting. Its trend in the years 1952-69 (Figure 9.3.10-A) is progressively rising from the industrial zone (1/109) towards the historical center (1/54). This variation can be attributed to the already noted gradual increase towards Venice of the more compressible soils. It explains why in Venice, where less water was pumped than in Marghera, a subsidence of the same magnitude was observed. A similar behaviour is found in the rebound phase (Figure 9.3.10-B) between 1969 and 1975. However in this period, the curve lies definitely below the other one, thus confirming that the elasticity of the system is very limited.

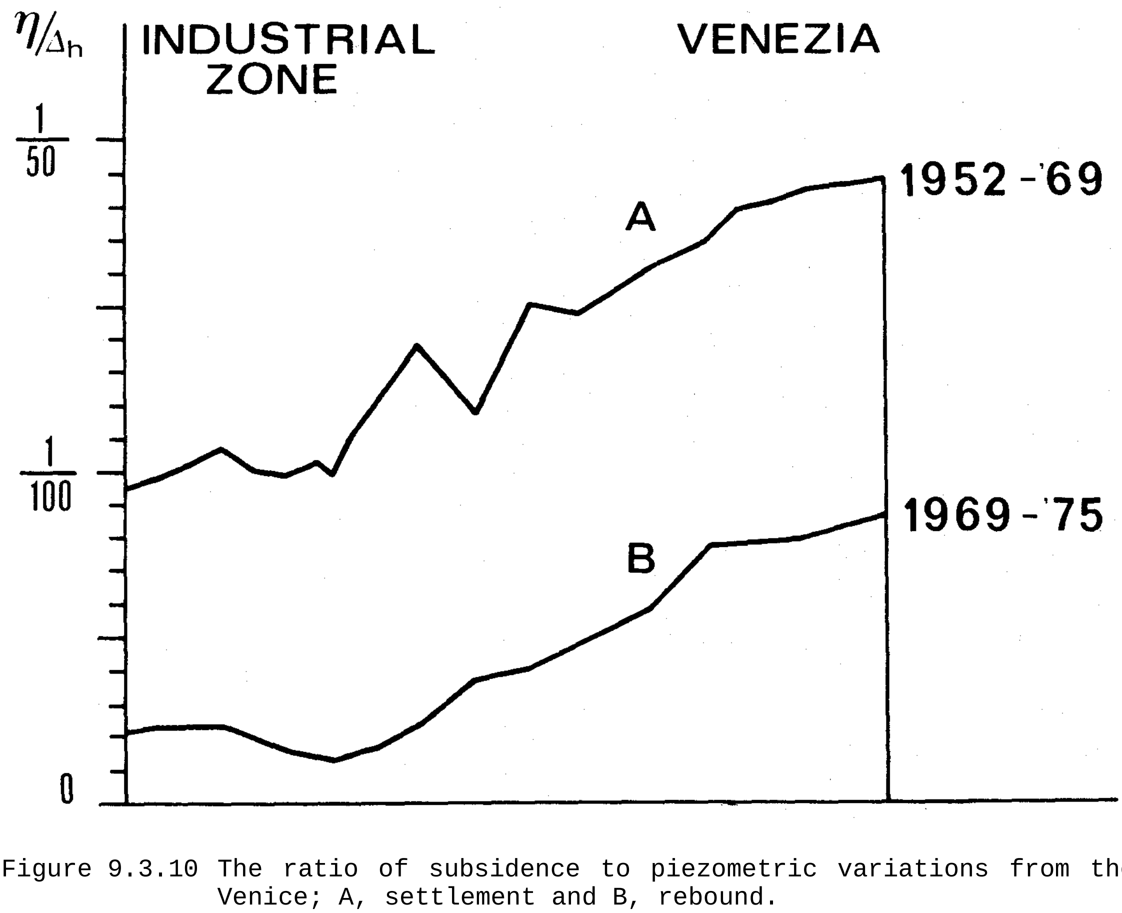


Figure 9.3.10 The ratio of subsidence to piezometric variations from the industrial zone to Venice; A, settlement and B, rebound.

9.3.4.2 Predictive simulations with the new records

Recently, a numerical model based on the classical diffusion equation and one-dimensional vertical consolidation has been used to simulate the past behaviour of the Venice subsidence and to predict the future settlement of the city (Gambolati and Freeze, 1973; Gambolati et al, 1974; Gambolati et al., 1975). A complete description of the approach together with an extensive discussion of the underlying assumptions may be found in the works cited.

The model has been applied again by using the new records to check its ability to reproduce the complex event at hand and to verify "a posteriori" its predictive capacity.

To date the pumpage at Marghera has been reduced to 40 per cent of its maximum value (460 l/s in 1969) and this change in the withdrawal rate has been assumed to have occurred in 1970, for it is apparent from Figure 9.3.4 that the flow field recovery in Marghera started in 1970. Permeability distribution is the same as that used in the previous simulation (Gambolati et al., 1974) while the soil compressibility in rebound has been increased to 20 per cent of the corresponding values in compression, as is evidenced by the most recent laboratory tests summarized in Figure 9.3.3. Therefore, the new results are slightly different from the early predictions given in figures 21 and 22 of the paper by Gambolati et al., 1974.

Figure 9.3.11 and Figure 9.3.12 show the piezometric decline in the first aquifer (where the largest amount of data is available) and the Venice subsidence respectively versus time as provided by the mathematical model using the updated records. For the benefit of the reader the behaviour during the calibration period has been reported as well. The comparison with the experimental observations indicate a fairly good agreement, and especially so, if one considers the degree of uncertainty which is inevitably related to physical events of such a great complexity. This is further evidence of the adequacy of the above model to reliably predict the settlement of Venice. At the same time the results allow the conclusion that the numerical models can be useful tools to investigate and keep under control land subsidence caused by subsurface fluid removal.

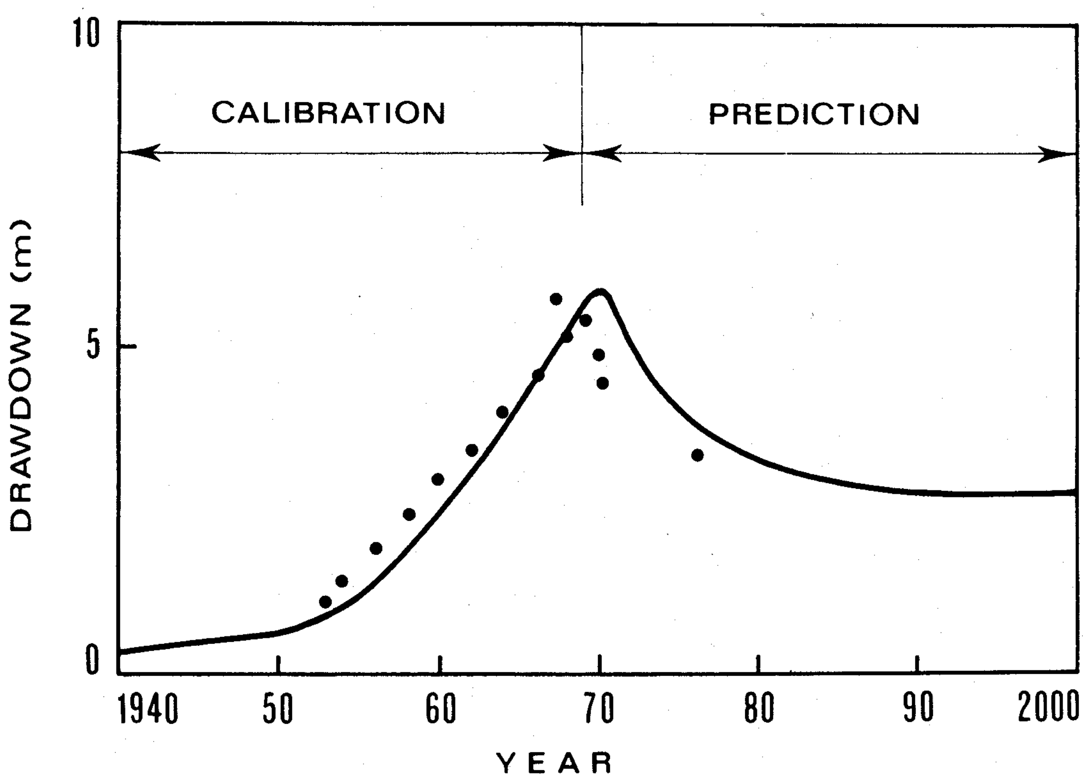


Figure 9.3.11 Piezometric decline versus time in the first aquifer. The closed circles represent experimental records and the solid line gives the response of the model using the new data in our possession.

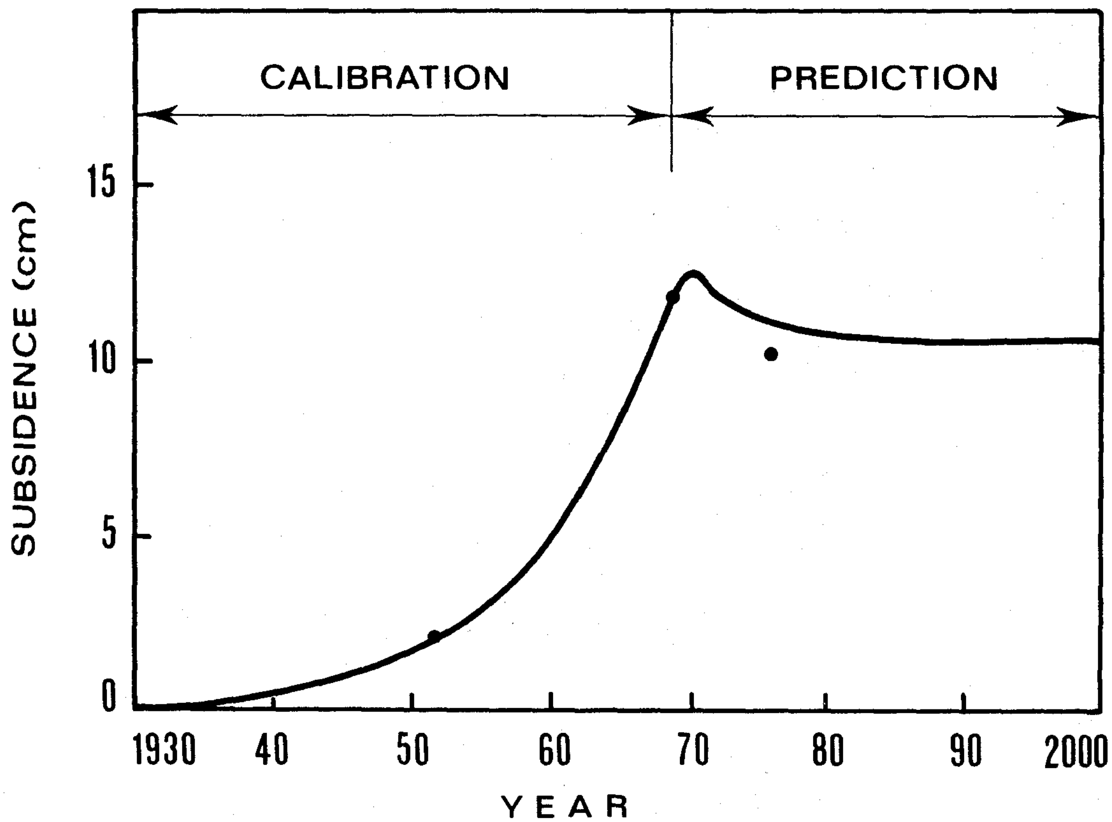


Figure 9.3.12 Subsidence in Venice versus time as provided by the model using the new data in our possession. The experimental records (o) are indicated.

9.3.5 CONCLUSIONS

The results of the experimental research have confirmed that also in the case of Venice, the sinking was caused by the artesian withdrawals. Also statistical analysis attributes 70 per cent of land subsidence occurring between 1952-1969 to the withdrawal of the underground water.

The experimental data showed that the pumpage performed at Marghera has greatly altered the natural flow field under the historical center and that the effects of the resulting subsidence are not uniformly distributed. In fact, for every metre of piezometric decline, the subsidence in the industrial area and in Venice was respectively 1 and 2 centimetres. This is connected to the relative increment towards Venice of the clay-type soils, which are more compressible, making the level of the city more dependent on the piezometric situation.

Soil deformations related to hydraulic head variations occur in a relatively short time due to the fact that aquitards are mainly silty and each of them is interrupted by thin sandy layers which facilitate the drainage.

But the most significant fact that arises from our investigation remains in any case the sudden rise of the piezometric levels recorded in the whole area since 1970, and related to a significant reduction of the artesian withdrawals in the last years. It is also important that there is a parallel surface rebound (2 cm in Venice), that ensures that land subsidence has been arrested. This result is in agreement with the predictions from the mathematical model.

Since a more careful use of the underground waters gives a very quick recovery, one can trust that a complete re-establishment of the natural hydraulic balance can be obtained, maybe with further intervention against wasting water (which can be estimated to be about 4.5 m^3/s due to the spontaneous spilling in the adjacent areas which influence the Venetian aquifer system).

Recovery will not, however, bring back the land to the original position, as it has been demonstrated that the reversibility of the compaction of the aquitards is possible for only 20 per cent (which would correspond to a rebound of about 3 cm).

Although the drawdown of the piezometric levels due to the intensive extractions of 1952-69 was the principal cause of the subsidence, we do not see a need for stopping the residual extractions.

Because of the unstable situation of Venice, it is necessary to continue the control of the piezometric levels of the aquifers and the ground altimetry. This is the only system by which we can evidence possible future variations from the present trend.

9.3.6 REFERENCES

ALBEROTANZA, L., FAVERO, V., GATTO, P., MASUTTI, M., MOZZI, G., PIANETTI, F., and R. SERANDREI. 1972. Catasto pozzi del Comune di Venezia. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 23, 5 vols., Venezia, 1,168 p.

BENINI, G. 1971. Relazione fra emungimenti ed abbassamenti del suolo. Ministero Lavori Pubblici, Comitato Difesa Venezia, III Gruppo, Padova, 97 p.

CAPUTO, M., FOLLONI, G., GUBELLINI, A., PIERI, L., and M, UNGUENDOLI. 1972. Survey and geometric analysis of the phenomena of subsidence in the region of Venice and its hinterland. Riv. Ital. di Geofisica, Milano, v. XXI, n.1/2, pp. 19-26.

CARBOGNIN, L., GATTO, P., and G. MOZZI. 1974. Situazione idrogeologica nel sottosuolo di Venezia. Cons. Naz. Ric., Lab. Din. Gr- Mas., Tech. Rep. 32, Venezia, 29 p.

CARBOGNIN, L., and P. GATTO. 1976. A methodology for hydrogeological data collection in the Venetian Plain. IBM Seminar on "Regional Groundwater Hydrology and Modeling," Venezia, 24 p.

CARBOGNIN, L., GATTO, P., and G. MOZZI. 1976. Movimenti del suolo a Venezia e variazioni piezometriche negli acquiferi artesiani. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 67, Venezia, in press.

CONSIGLIO NAZIONALE DELLE RICERCHE. 1971. Relazione sul Pozzo VE 1 CNR. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 14-21, Venezia.

CONSIGLIO NAZIONALE DELLE RICERCHER. 1975. Livellazione geometrica di precisione rua di Feletto-Venezia. Unpublished report of the Lab. Din. Gr. Mas., Venezia.

FAVERO, V., ALBEROTANZA, L., and R. SERANDREI BARBERO. 1973. Aspetti paleoecologici, sedimentologici e geochimici dei sedimenti attraversati dal pozzo VE, 1 bis CNR. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 63, Venezia, 51 p.

FOLLONI, G., DE SANCTIS RICCIARDONE, and A. GUBELLINI. 1974. Evoluzione recente del fenomeno di subsidenza nella zone di Venezia e del suo entroterra. ist. di Topografia e Geodesia, Universita di Bologna, 16 p.

FONTES, J. CH., and G. BORTOLAMI., 1972. Subsidence of the area of Venice during the past 40,000 years. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 54, Venezia, 11 p.

GAMBOLATI, G., and FREEZE, R. A., 1973. Mathematicam simulation of the subsidence of Venice, lst Theory, Water Resource Research, v. 9, no. 3, pp, 721-733,

GAMBOLATI, G., GATTO, P., and FREEZE, R. A., 1974. Mathematical simulation of subsidence of Venice, 2nd results. Water Resource Research, v. 10, no. 3, pp. 563-567.

GAMBOLATI, G., GATTO, P., and FREEZE, R. A., 1974. Predictive simulation of the subsidence of Venice. Scienze, v. 183, pp. 849-851.

GUBELLINI, A., and G. DESANCTIS RICCIARDONE., 1972. Analisi dei risultati di due livellazioni esequite nella zona di Venezia nel 1970 e nel 1971. Cons. Naz. Ric., Lab. Din. Gr., Mas., Tech. Rep. 41, Venezia, 18 p.

LEONARDI, P., 1960. Cause geologiche del graduale sprofondamento di Venezia e della sua Laguna. Ist. Ven. Sc. Lett. e Arti, Venezia, 21 p.

MINISTERO DEI LAVORI PUBBLICI. 1973. Pozzi LIDO 1 e MARGHERA 1. Comitato Difesa Venezia, III Gruppo, Relazione AGIP, S.Donato Milanese.

MINISTERO DEI LAVORI PUBBLICI. 1974. Catasto dei pozzi artesiani. Comitato Difesa Venezia, IV Gruppo, no. 1, Fasc. no. 18, 35 p.

MOZZI, G., BENINI, G., CARBOGNIN, L., GATTO, P., and M. MASUTTI. 1975. Evoluzione delle pressioni di strato negli acquiferi artesiani. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 66, Venezia, 38 p.

NEGLIA, S. 1972. Pozzi VE 1 e VE 1 bis CNR, Analisi geochimiche sulle carote argillose. Cons. Naz. Ric., Lab. Din.,Gr. Mas., Tech. Rep. 56, Venezia, 16 p.

POLAND, J. F., and MOSTERTMAN, L. J. 1969. Reconnaissance investigation of the subsidence of Venice and suggested steps toward its control. UNESCO report, Paris, 24 p.

RICCERI, G., and BUTTERFIELD, R. 1974. An analysis of compressibiility data from a deep borehole in Venice. Geotechnique, v. 24, no. 2, London, pp. 175-192.

ROWE, P. W. 1973. Soil mechanics aspects of the cores of the deep borehole VE 1 CNR in Venice. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 57, Venezia, 53 p.

SERANDREI BARBERO R. 1972. Indagine sullo sfruttamento artesiano nel Comune di Venezia, 1846-1970. Cons. Naz. Ric., Lab. Din. Gr. Mas., Tech. Rep. 31, Venezia, 97 p.

Note by the authors:

After 1975, both piezometric and geodetic surveys were continued on the studied area. The 1978 situation shows that the natural repressuring of the aquifers has continued and today artesian heads are coming back to the value recorded before the over-pumpage in 1952. At the same time precise leveling shows that the land has stabilized after the 1975 rebound.

Case History No. 9.4. Tokyo, Japan, by Soki Yamamoto, Rissho University, Tokyo, Japan

9.4.1 TOPOGRAPHY AND GEOLOGY OF TOKYO

Tokyo is situated at the bottom of the Kwanto structural basin, the biggest plain in Japan. Kwanto Plain is surrounded by mountains and hills on the north, west, and south where basement rocks of Tertiary and Pre-Tertiary age are exposed. The overlying new strata dip to the center of the basin.

All the rivers, such as the R. Tone, the R. Ara, and the R. Tama, start from this divide, transporting their sediments into Old Tokyo Bay. They developed fans at the foot of the mountain and a deltaic plan at their mouths in Tokyo Bay.

After the intermittent uplift, broad terraces were formed on this basin (Figure 9.4.1); on the surface of the terraces, thin volcanic ashes (Tephra) of different origins and deposited at different times are found. The maximum thickness of sedimentary rocks above the basement complex is about 300 m at the center of the basin.

The metropolis of Tokyo is located on the upland and lowland. The stratigraphic succession and schematic cross section in the Tokyo area are shown in Table 9.4.1 and Figure 9.4.2. There are many buried valleys in the lowland where an alternation of fairly thick sand and gravel layers are deposited, underlain by the Tokyo Group (Figure 9.4.3).

9.4.2 HYDROLOGY

The areas in Tokyo where land subsidence has taken place are tile Musashino upland and the alluvial lowland. There are two groups of aquifers, shallow and deep ones. The main shallow aquifer on the lowland (Koto) is Holocene sand and gravels and that on the upland area is Musashino gravel which is extensively distributed. In addition, deep artesian water is obtained from the Tokyo Group in the lowland and the Tokyo and Kazusa Groups in the upland. The Kazusa Group in the lowland contains natural methane gas which was produced for municipal supply in the Koto district. The large scale ground water development started in 1914 in Tokyo. After that time, the number of deep wells with large diameters increased rapidly. In an area extending from the northern part of the alluvial lowland to the southern part of Saitama Prefecture, there was artesian flow of ground water until the latter half of the 1920’s. At that time the ground-water level in the Koto district had fallen to about 10 m below the ground surface. The ground-water level continued to fall year after year, but toward the end of World War II it rose again temporarily. After the War, as the quantity of ground-water withdrawals increased, the ground-water level again went down until August, 1971, when it reached a low of minus 63-94 m from the mean sea level of Tokyo Bay (Tokyo Peil) in the northern part of the alluvial lowland.

Figure 9.4.4 shows the annual change of the ground-water level in selected observation wells.

The annual amounts of ground-water withdrawal in Tokyo from 1964 to 1975 are shown in Table 9.4.2. In the 23 wards, 1,160,000 m^3/day of ground water was withdrawn in 1964, but the quantity began to decrease by 1966, and it fell to 128,000 m^3/day, or about a tenth, in 1975. This decline is attributed to the withdrawal restrictions imposed to control land subsidence. Figure 9.4.5A shows the distribution of withdrawals by ward in 1967, and Figure 9.4.5B shows the annual total, 1950-1967.

In the northern part of Tokyo, the drilling of wells to a depth of up to 160 m had been banned by December, 1971; by May, 1974 drilling of wells with a depth exceeding 160 m was also banned. As a result, the quantity of ground-water withdrawals, which amounted to 80,000-90,000 M^3/day in the period from May, 1972 to 1973, decreased to 7,000-8,000 m^3/day after May 1974. Furthermore, the ground-water level, which was lowest (T.P.* minus 48.9 m) in July, 1971, rose again gradually, as the quantity of ground-water withdrawals decreased.

---

* T. P. = Tokyo Bay datum

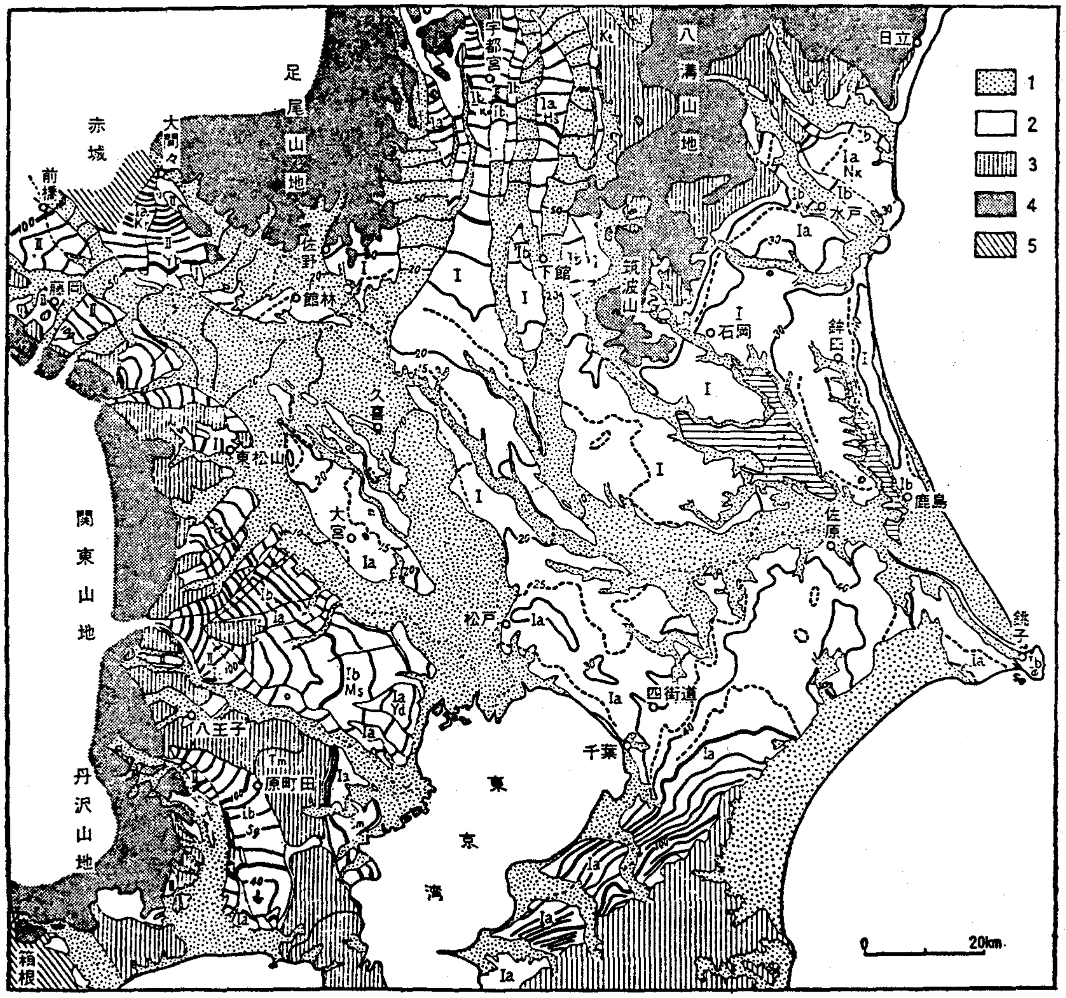


Figure 9.4.1 Geomorphological map of Kwanto District (after Kaizuka). 1, Alluvial lowland; 2, Diluvial upland; 3, Tertiary hill; 4, mountain; 5, volcano.

9.4.3 LAND SUBSIDENCE

In 1923, a severe earthquake occurred near Tokyo, causing widespread damage in the Koto region, east of the city of Tokyo. In order to study the crustal disturbance which might have accompanied this severe earthquake, a precise leveling was rerun in this region. As a result, it was found that the land subsidence was as a whole increasing gradually year by year. It was also found that the extent of the region where the land subsidence was then advancing occupied an area of about 100 km^2, situated between the Sumida and the Arakawa rivers, which flow through the region from north to south.

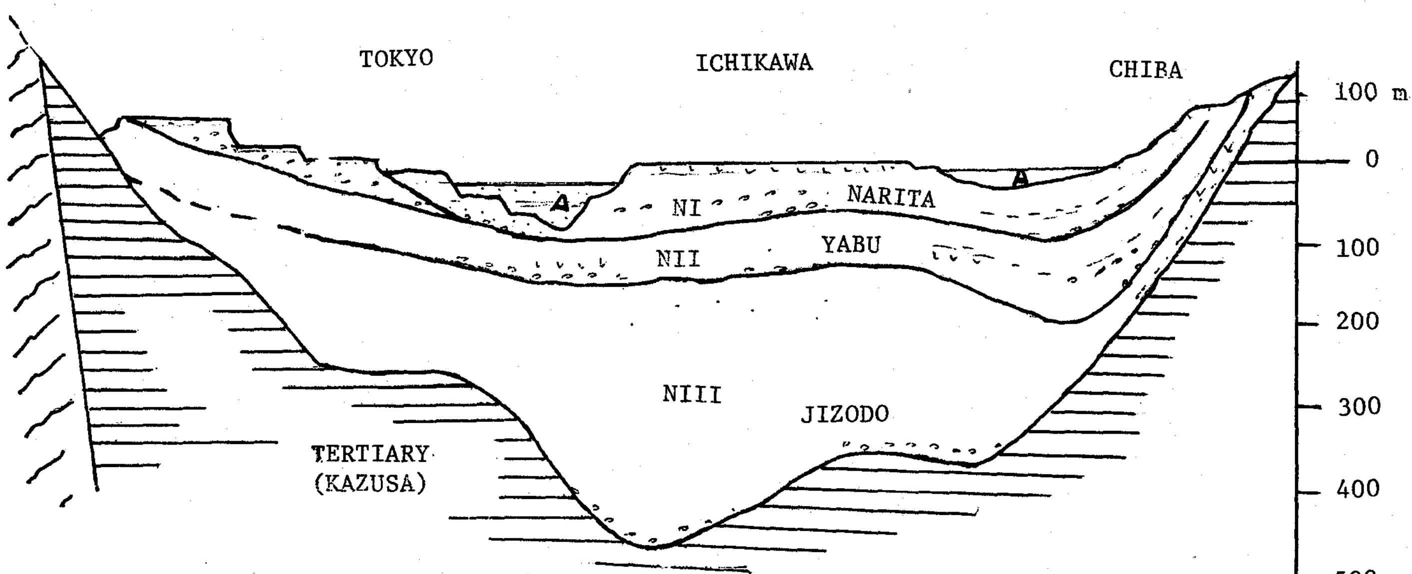
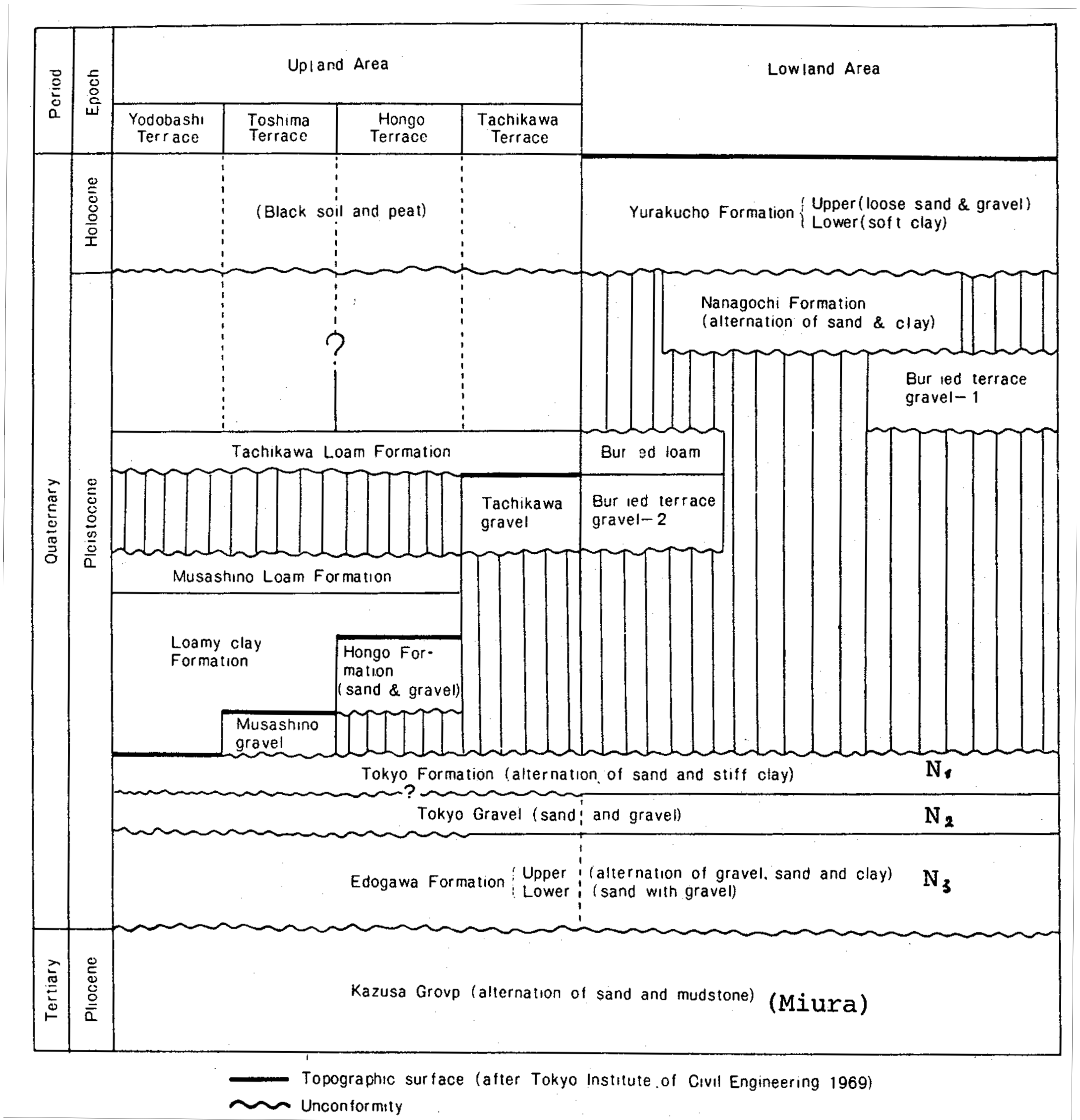


Figure 9.4.2 Schematic geologic cross section of Kwanto Basin.

Table 9.4.1 Stratigraphic succession in Tokyo



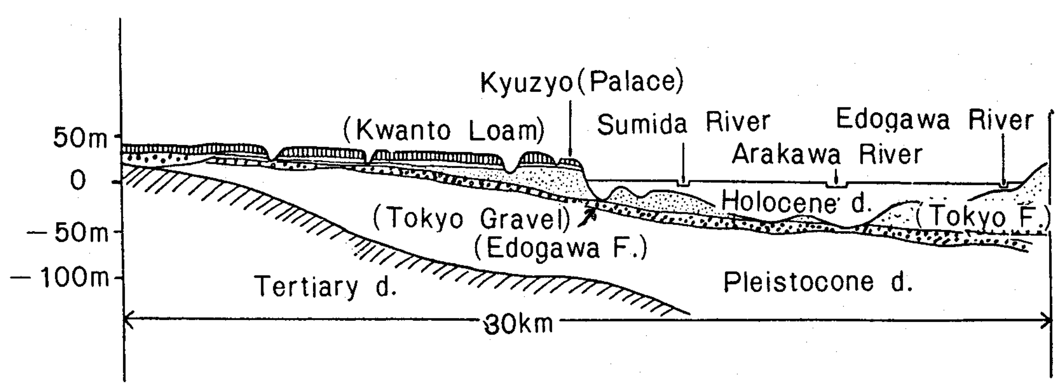


Figure 9.4.3 Geologic cross section.

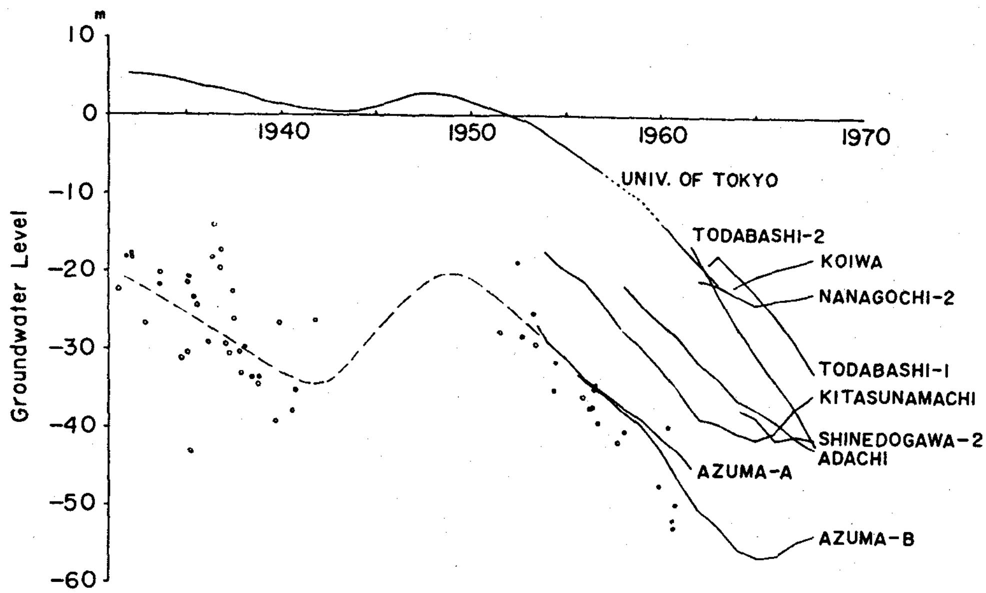


Figure 9.4.4 Secular trend of ground-water levels. Small circles indicate ground-water levels at the time of bore drilling near AZUMA-A and B (node 4)

Table 9.4.2 Amounts of ground-water withdrawal in Tokyo.

Unit: 1000m³/day

Year / Area	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975
23 wards	1162	1017	805	732	679	674	747	550	324	283	195	128
Tama	353	480	536	594	626	765	953	891	853	861	818	790
Total	1515	1497	1341	1326	1305	1439	1700	1441	1177	1144	1013	918

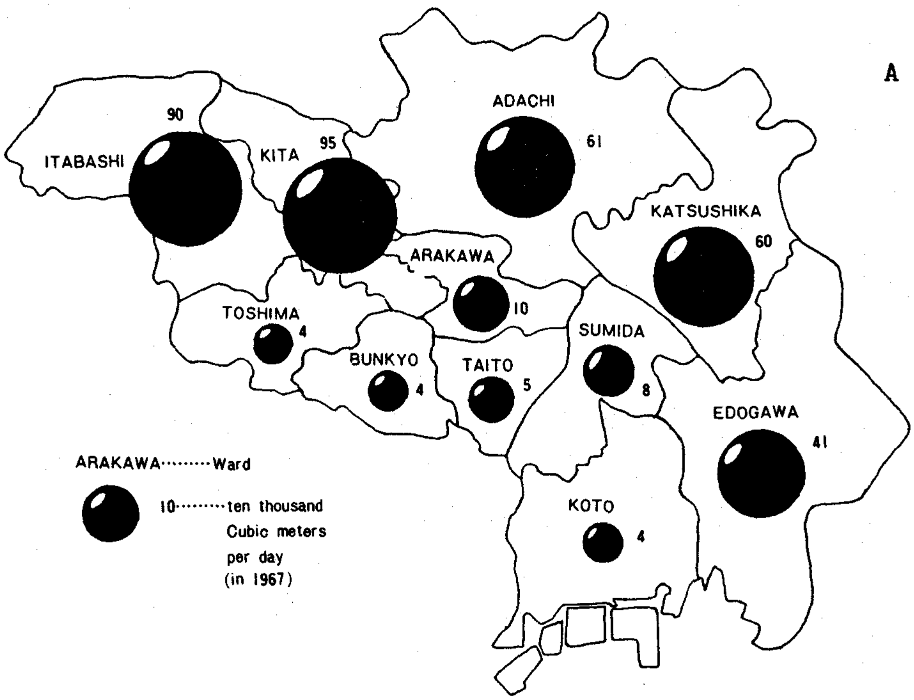
Data from Bureau of Environmental Protection, Tokyo Metropolitan Government

In association with the advancement of such a local subsidence, several remarkable phenomena occurred, such as "lift up" of masonry buildings and well pumps and inundation by rivers and sea tide.

In order to make clear the general features of the subsidence, precise leveling along the network of the leveling routes in Tokyo was started. It takes, however, considerable time to carry out the leveling survey on the network, including all bench-marks in Tokyo. Therefore, the leveling survey has been repeated frequently on the network of bench-marks in the region where the land subsidence is greatest.

In the first stage of study of the subsidence, the leveling was repeated at irregular intervals. Afterwards, it was thought to be inconvenient to work out vertical displacements based on the data of precise levels repeated at irregular intervals, since the amounts of the subsidence became larger and the rates of the subsidence were different from place to place. Therefore, in the Koto region, i.e., the region east of the Sumida river, where the subsidence was greatest, the leveling was repeated every two years, during the period from 1938 to 1946. Since then, leveling has been repeated every year in this region.

First order leveling and observations of the compaction of soil layers and the ground-water levels by means of observation wells were also carried out. As of January, 1976, the area Surveyed by leveling extended to 900 km^2, using 632 bench-marks where the levelings are made



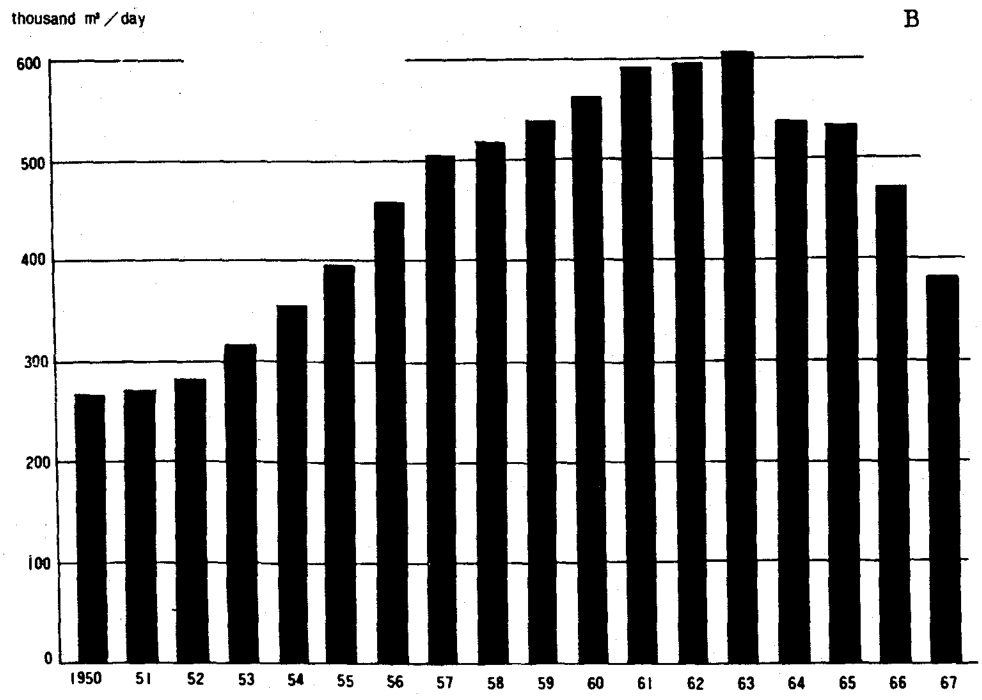


Figure 9.4.5 Amounts of ground-water withdrawal; A, by ward; B, by year.

every year. The compaction of soil layers and the changes of ground-water levels are observed at 68 observation wells located at 34 sites (Figure 9.4.6). The water-level plots are dashed.

Land subsidence has occurred in the Koto district since around 1900 and in the eastern part of the alluvial lowland (Edogawa Ward) since 1920. On the other hand, in the Musashino upland, land subsidence began to occur in the latter half of the 1950's.

The maximum subsidence in Tokyo is about 4.6 m and the maximum rate is 27 cm/yr (Figure 9.4.7). The total subsiding area in Kwanto (Tokyo, Chaiba, Kanagawa, and Saitama) amounts to 2420 km^2 and the area where the subsidence amounts to more than 10 cm/yr is still about 100 km^2.

In order to prevent or abate such a rate of subsidence, the pumping of ground water was restricted as stated above, and thus the rate has dropped year after year since 1972.

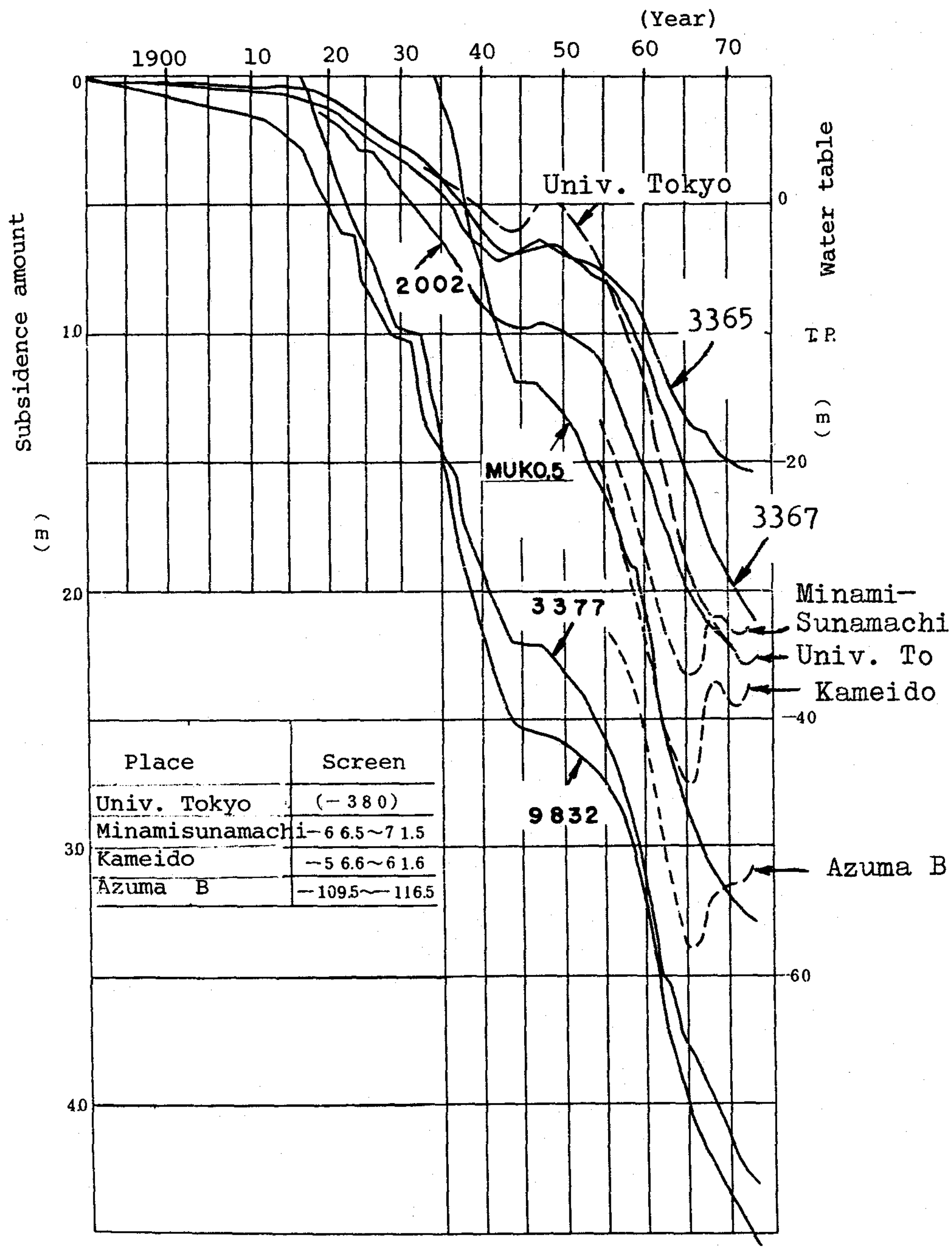


Figure 9.4.6 Secular changes of land subsidence and ground-water levels in Tokyo.

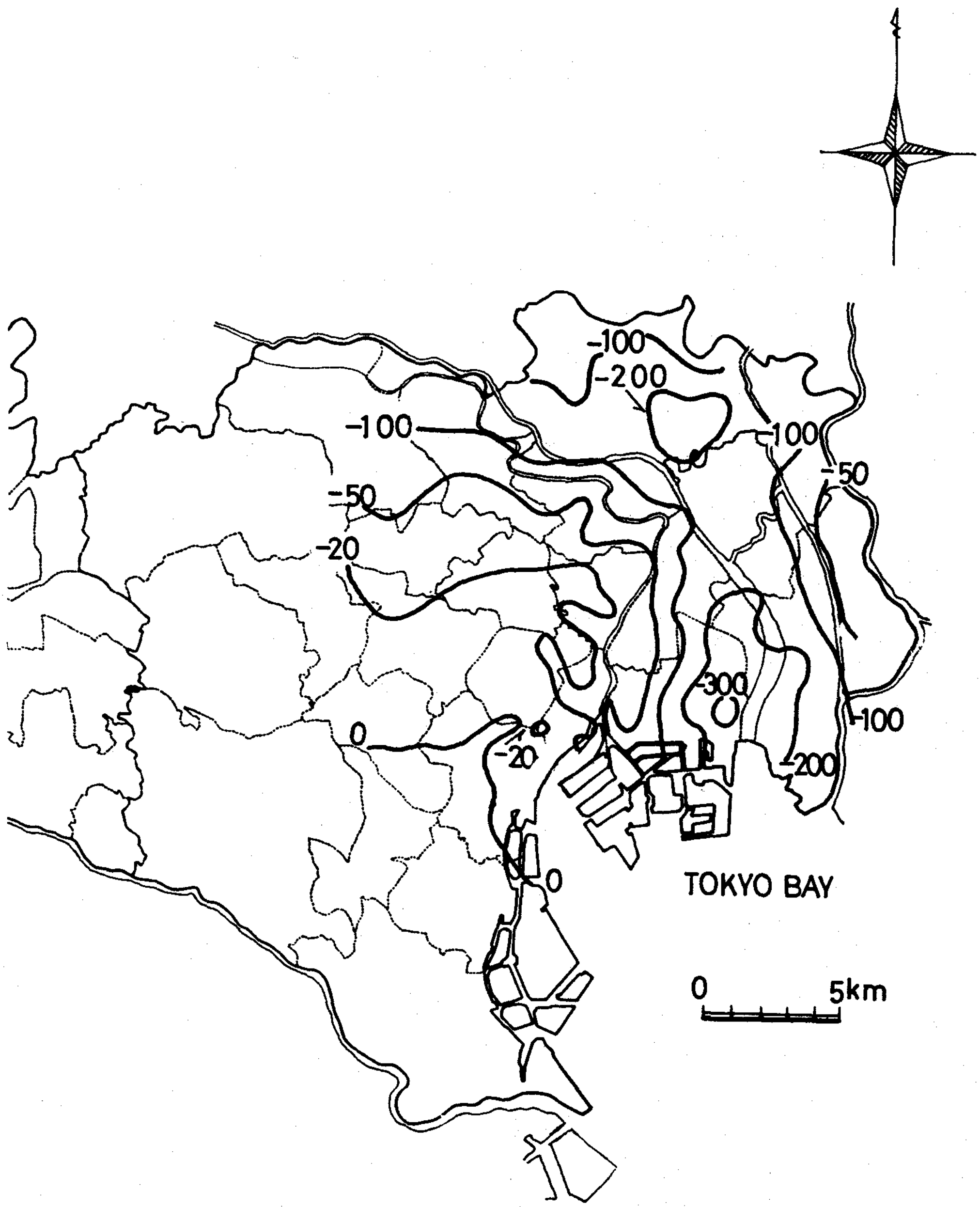


Figure 9.4.7 Total subsidence in Tokyo from 1938 to 1975.

9.4.4 PARAMETERS

Soil tests were carried out on undisturbed core samples. Consolidation tests were made by applying one-directional pressure. C_c value ranged from 0.2 to 1.2 and has the tendency of increasing with increasing water content. The M_v value varies as follows:

Alluvial clay	$2 - 3 \times 10^{-2}$ cm^2/kg
Diluvial clay	$2 - 6 \times 10^{-3}$
Tertiary clay	$1 \times 10^{-3} - 4 \times 10^{-4}$.

K: hydraulic conductivity

Tokyo Group	2.1×10^{-2} cm/sec
Kazusa Group	1.3×10^{-2}

9.4.5 COUNTERMEASURES

In Tokyo, the local government legislated a Metropolitan Ground-Water Law, superposed on the "Industrial Ground-Water Law" and "Building Ground-Water Law." Moreover, they constructed dikes for floods and high tides, pumping installations for drainage, water-supply works for industry, and polder systems. The estimated cost for the countermeasures for the period 1957 through 1970 is about 225 million U.S. dollars.

The regulations for ground-water withdrawal are as follows:

1. Restrictions under the Industrial Water Law.
   The restrictions are designed to reduce the ground-water withdrawals by supplying substitute waters. The main restrictions are described chronologically in the following:

   January 1961: A ban on drilling a new well in the southern part of the alluvial lowland (the Koto district).
   July 1963: A ban on drilling a new well in the northern part of the alluvial lowland (the Johoku district).
   June 1966: Pumping of ground water in the southern part of the alluvial lowland (the Koto district) was restricted.
   December 1971: Pumping of ground water in the northern part of the alluvial lowland (the Johoku district) was restricted.
   April 1975: Pumping of ground water in the eastern part of the alluvial lowland (the Edogawa district) was restricted.

2. Restrictions under the Law Controlling Pumping of Ground Water for Use in Buildings
   The law aims at holding in check the pumping of ground water for air conditioning and other non-drinking purposes in medium- and highrise buildings. The progress of restrictions under the law is described chronologically in the following:

   July 1963: A ban on the drilling of new wells in the alluvial lowland.
   July 1965 and July 1966: Restrictions on the pumping of ground water in the alluvial lowland.
   May 1973: The restriction was extended to the whole area of the 23 wards, and the control of ground-water withdrawals was strengthened.

3. Restrictions under the Tokyo Metropolitan Environmental Pollution Control Ordinance.
   The ordinance restricted the drilling of new wells in areas not covered by the two laws mentioned above.

4. Suspension of drawing ground water containing natural gas
   The Tokyo Metropolitan Government in December 1972 bought the mining rights for water-soluble natural gas extracted in the neighborhood of the Ara River estuary, and thereby suspended the pumping of gas-water (3,000 M^3/day) from the Kazusa Group.

   On the other hand, in the Tama district, the drilling of new wells for industrial water and water for non-drinking purposes (for example, the supply of bath water) is restricted under the Metropolitan Ordinance mentioned in (3) above, but with an increase in population in the district, the demand for ground water climbed from 350,000 m^3/day in 1964 to some 900,000 M^3 /day in 1971.

The replacement drinking water and building water are supplied from the River Tone through the Musashi aqueduct and the industrial water is supplied from the Metropolitan industrial water works on the Tone which is provided by the construction of high dams on the upper part of this river.

9.4.6 SELECTED REFERENCES

MIYABE, N. 1962. Studies in the ground sinking in Tokyo, Rept. Tokyo Institute of Civil Engineering, p. 1-38.

INABA, Y., AOKI, S., ENDO, T., and R. KAIDO. 1969. Reviews of land subsidence researches in Tokyo, IAHS Pub. No. 88, p. 87-98.

ISHI, M., KURAMOCHI, F., and T. ENDO. 1977. Recent tendencies of the land subsidence in Tokyo, IAHS Pub. No. 121, p. 25-34.

Case History No. 9.5. Osaka, Japan, by Soki Yamamoto, Rissho University, Tokyo, Japan

9.5.1 GEOLOGY OF OSAKA

The Osaka basin surrounded by the Rokko and Ikoma Ranges is one of the typical Quaternary basins in the Kinki Triangle. The Rokko elevation reaches more than 900 m in the highest part and the sinking Osaka basin has been filled by the Pleistocene Osaka group and the later sediments which are certainly over 600 m in thickness in the central part of the basin. The horizon belonging to the lower Pleistocene which is the same one recognized at a depth of more than 500 m by boring in the Osaka basin can be confirmed at the height of 500 m in the Rokko Range. The amplitude of the folding of the basement represented by the Osaka basin and the Rokko Range is considered to reach more than 1,000 m since the early Pleistocene (Figure 9.5.1 and 9.5.2).

Complex thrust systems have developed especially along the boundary zones between uplifts and subsidences. The beds older than the middle Pleistocene are divided by thin tuff beds of Ma_0, Ma_l, Ma_2, --- , Ma_{l2}, and have been strongly disturbed by faulting everywhere around the Osaka basin. Terrace deposits have been confirmed to be displaced by faulting at many places. For example, the granite mass of Rokko has thrust up against the higher terrace deposit.

A wide terrace developed in the northern part of the Osaka basin; it is named the Itami terrace, and is the lowest one in this area. The Itami gravels composing this terrace surface gently dip to the center of the Osaka basin. A radiocarbon age determination made on a wood fragment contained in the Itami clay which is overlain by the Itami gravels is 29,800±1,200 years B.P.

The distribution of the "Alluvial deposits" in Osaka has been revealed by boring and the sonic Sparker survey. The deposits indicate the curvature of the surface of the basement and may suggest the shape of basin-forming recent subsidence of the Osaka basin (Table 9.5.1).

9.5.2 HYDROLOGY AND SUBSIDENCE

The first layer below the ground surface is an alluvial layer except on the Uemachi upland running south from the vicinity of Osaka Castle. The average thickness of this alluvial clay layer is about 15 m. The thickness becomes greater as it approaches the coastal zone. Below the Alluvium are very thick Diluvial deposits of Pleistocene age, which consist of alternations of sand and clay layers. Both alluvium and diluvium layers form an excellent aquifer in this district (Figure 9.5.3).

The ground water head was very high in the city until about 50 years ago. It is reported that even flowing artesian wells could be seen at some parts of the city. With the development of industry, however, the use of ground water gradually increased and land subsidence due to the withdrawal of ground water began to appear. Before 1928, the land subsidence in the city was very slight, being at a rate of 6-13 mm/yr. This slight subsidence is considered to be the result of the natural movement of the earth crust and of the natural consolidation of the newly deposited alluvial clay.

After that time, however, a remarkable increase in use of ground water caused an increase in the rate of subsidence (Table 9.5-2).

Since that time, precise leveling for the wider part in the city has been carried out every year by the Osaka Municipal office following the suggestion of M. Imamura. Besides the leveling, the amount of consolidation of soil layer and the artesian heads of various aquifers (O.P.* -33 m, O.P. -62 m, O.P. -176 m) were observed by self-recording apparatus by K. Wadachi at Kujoh Park in 1938. Figure 9.5.4 shows the total amount of subsidence of various bench marks in

---

* O.P. (Osaka Peil) means the lowest low-water level observed in Osaka Port in 1885 and this level is used as the standard datum in Osaka area.

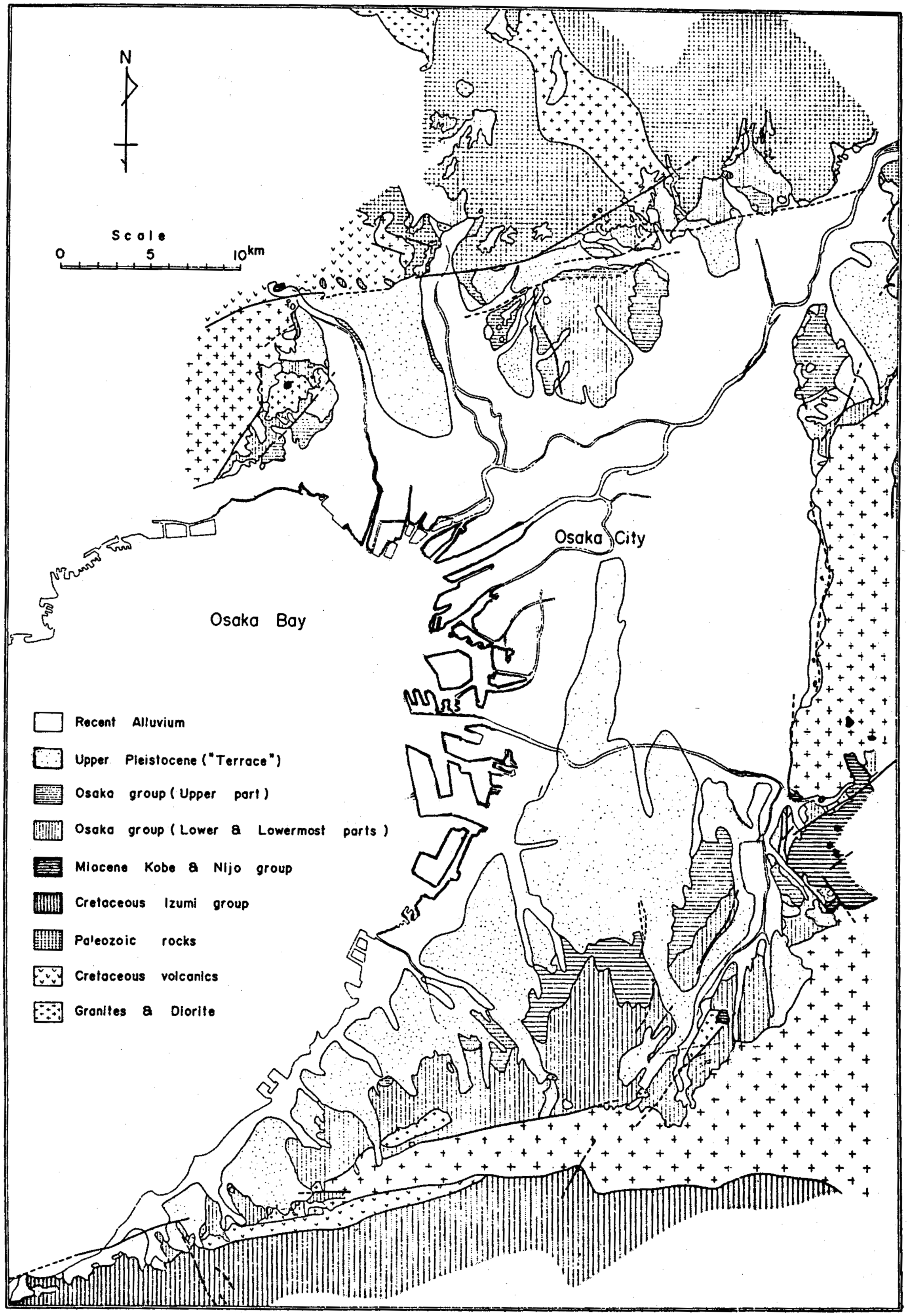


Figure 9.5.1 General geologic map.

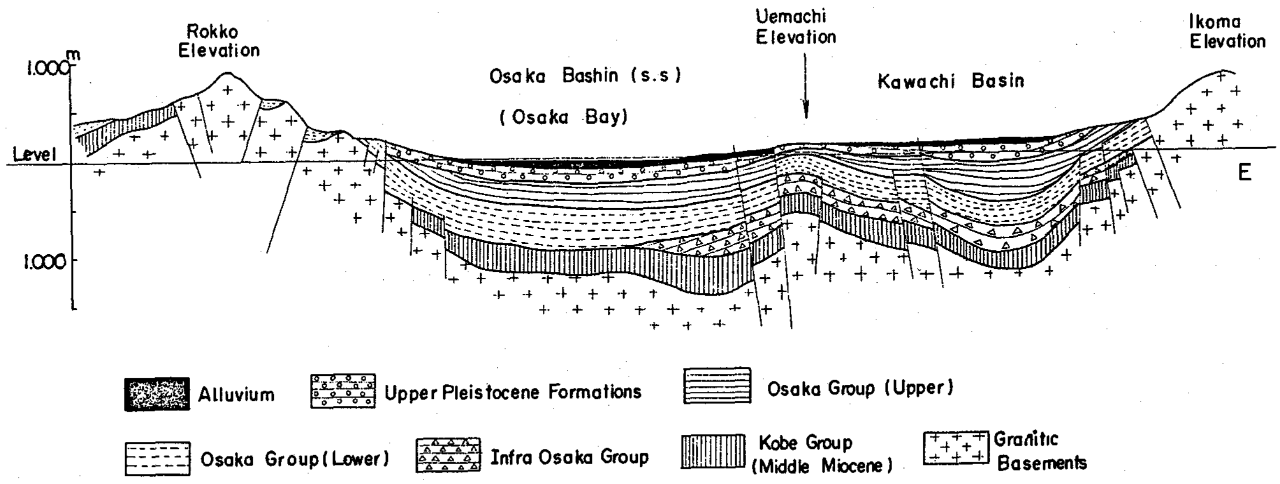


Figure 9.5.2 Diagrammatic profile of the Osaka basin (Ikebe & Takenaka).

western Osaka and secular variation of the artesian head at the Kujoh well 176 m deep. As seen in this figure, the rate of subsidence seems to be classified into four phases. In the first period from 1935 to 1943 the land subsidence was very rapid as the industry in the city developed. In the second period from 1944 to 1951, land subsidence was very slight or sometimes stopped. At that time, industrial activity was greatly depressed by the war disaster and the use of ground water decreased. It began to increase again in the third period from 1952 to 1964 because of the remarkable increase of industrial water use. From 1964 to the present, it has decreased because of the regulations against ground water use. The variation of ground water head is almost similar to that of land subsidence.

The artesian head and rate of subsidence are closely correlated at the Kujoh well. In spite of some unconformity of peak years, however, the general features of the variation in the artesian water head and in the rate of subsidence show a close correspondence and suggest a causal connection between these phenomena. The period when land subsidence stopped corresponds to the period of recovery of the ground water head (Figure 9.5.4).

The total subsidence during 34 years from 1935 to 1968 in Osaka is shown by the isopleths in Figure 9.5.5. In this figure, it can be noticed that the subsidence is larger nearer to the coastal zone, and a zone of little subsidence is left in the upland in the middle of Osaka where no alluvial covering exists. The subsidence area covers about 570 km^2 at present, but is successfully decreasing.

In spite of the success in preventing land subsidence in Osaka City, the subsidence in eastern and northern Osaka has increased remarkably during the recent few years. These regions have been developed lately and many factories which demand much ground water have been built.

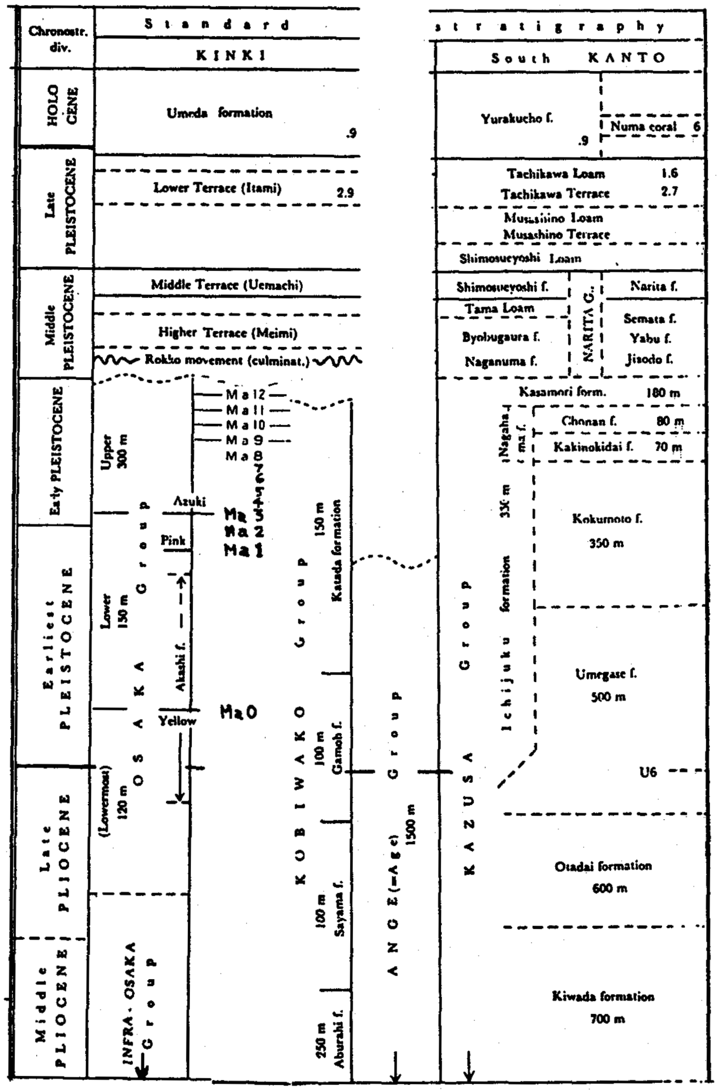
9.5.3 PARAMETERS

The compression index C_c was obtained by the standard oedometer test. The value varies from 0.2 to -1.8 and is proportional to the liquid limit w_L. The equation of the regression line is $C_c = 0.017\ (w_L - 37)$. The preconsolidation pressure varies from 2 to 50 (kg/cm^2). It increases proportionally to the depth and is generally larger than the present effective overburden pressure.

9.5.4 ECONOMIC AND SOCIAL IMPACT

This can be seen from Figure 9.5.6 easily. Total amounts of industrial products are presented in yen deflated in economic appraisal of 1965.

Table 9.5.1 Stratigraphic correlation of geology in Osaka.



9.5.5 COUNTERMEASURES WITH LEGAL REGULATION

Due to land subsidence, the ground surface of a part of western Osaka has sunk below sea level and the city is exposed to the danger of floods caused by storm surges in Osaka Bay. Subsidence areas below high tide have extended to about 100 km^2.

In 1934 a very large flood occurred, caused by the Muroto Typhoon, the biggest typhoon that ever attacked Osaka. An area of about 49 km^2 was flooded by the storm surge of O.P. + 4.20 m. In order to prevent further disasters, the dikes have been repaired and built more satisfactorily.

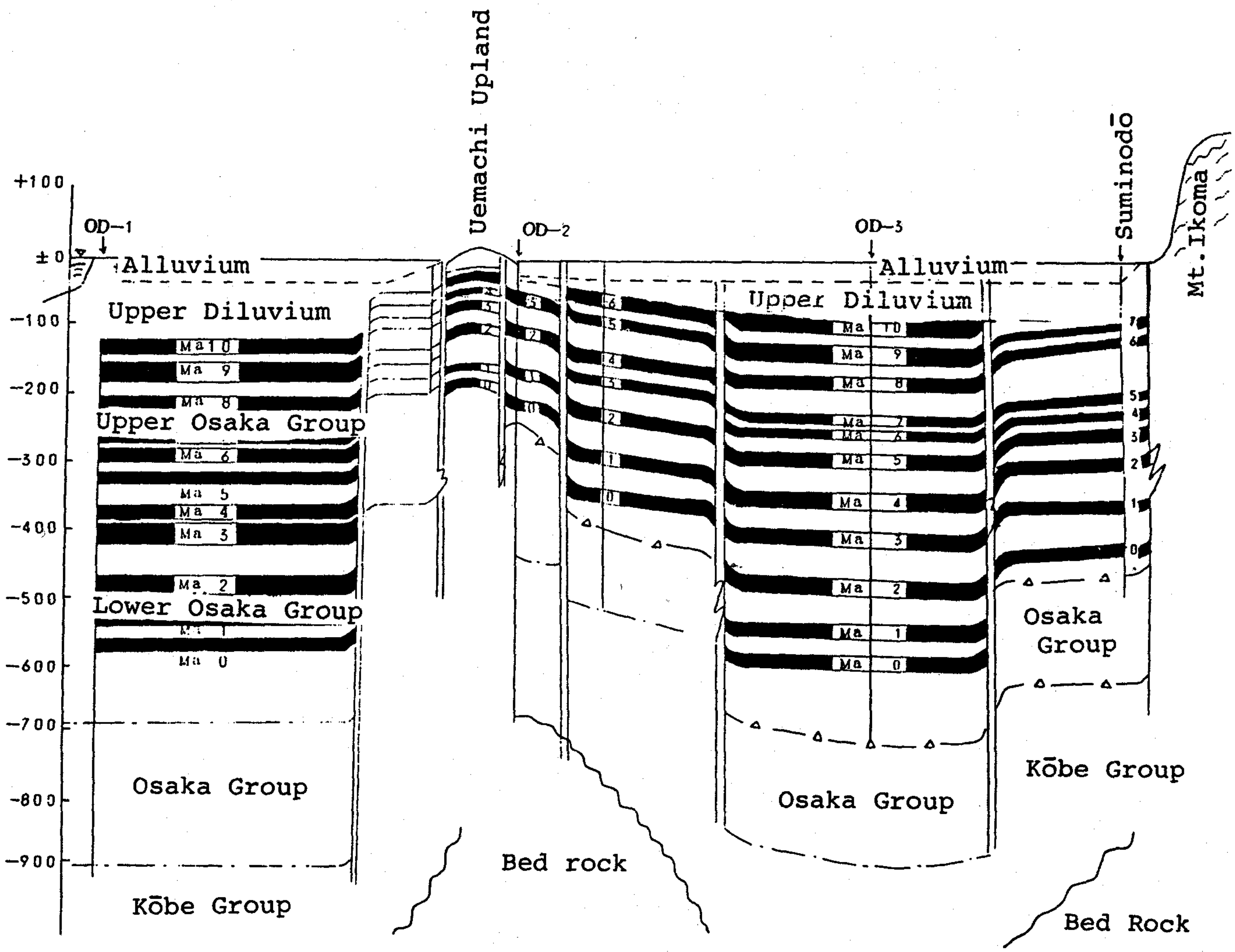


Figure 9.5.3 Geologic cross section of Osaka.

Table 9.5.2. Change of ground water withdrawal in Osaka

Year	Discharge amount x 10^3 m^3/year	Year	Discharge amount x 10^3 m^3/year	Year	Discharge amount x 10^3 m^3/year
1920	956	1956	65060	1963	94780
1930	6386	1957	93120	1964	79190
1940	18997	1958	110720	1965	56120
1945	19309	1959	130420	1966	34410
1953	30000	1960	144100	1967	15400
1954	37440	1961	130420	1968	4660
1955	57440	1962	123680	1974	42

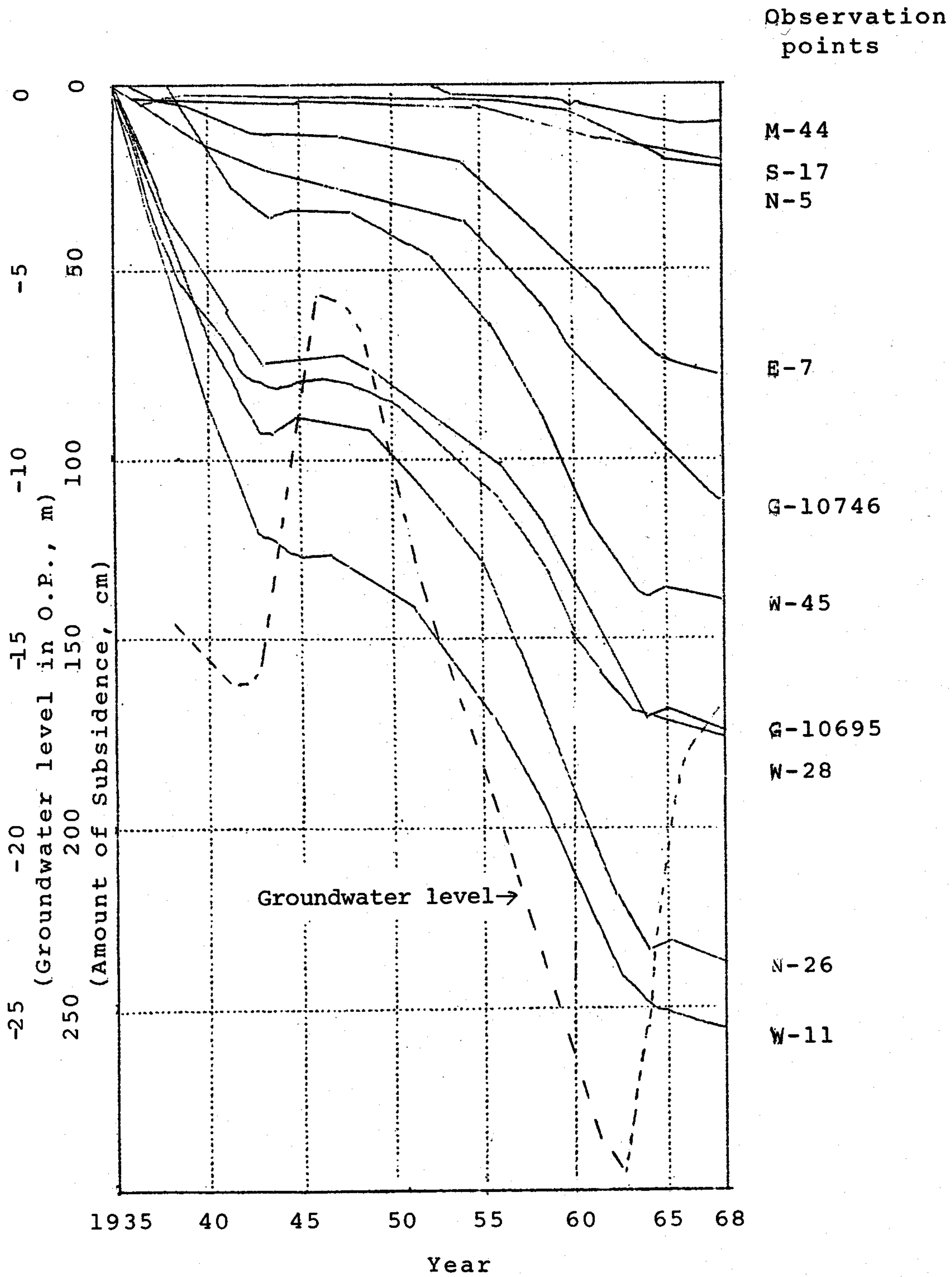


Figure 9.5.4 Secular changes of land subsidence and ground-water level in Osaka.

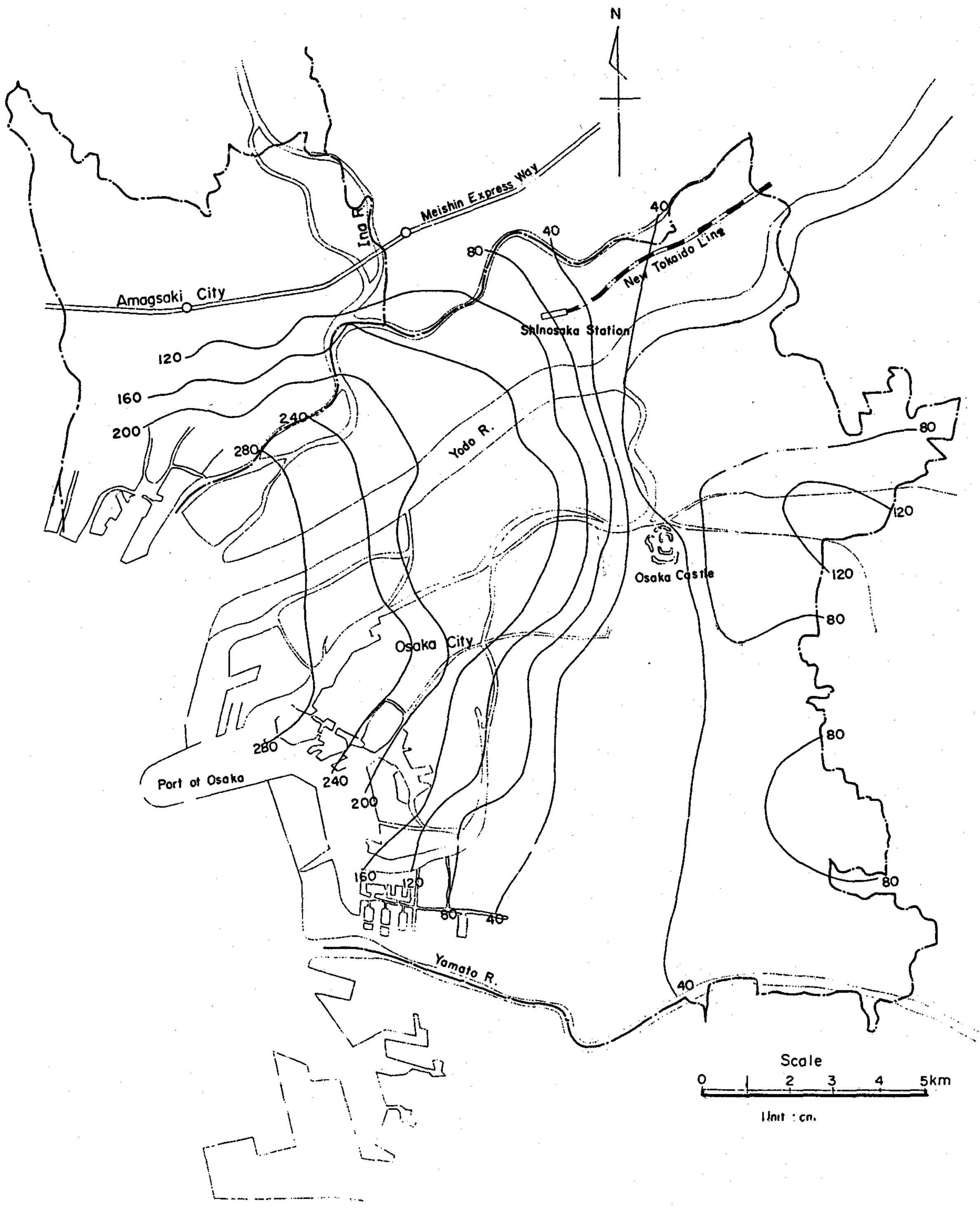


Figure 9.5.5 Isopleths of amount of land subsidence in Osaka area from 1935 to 1968

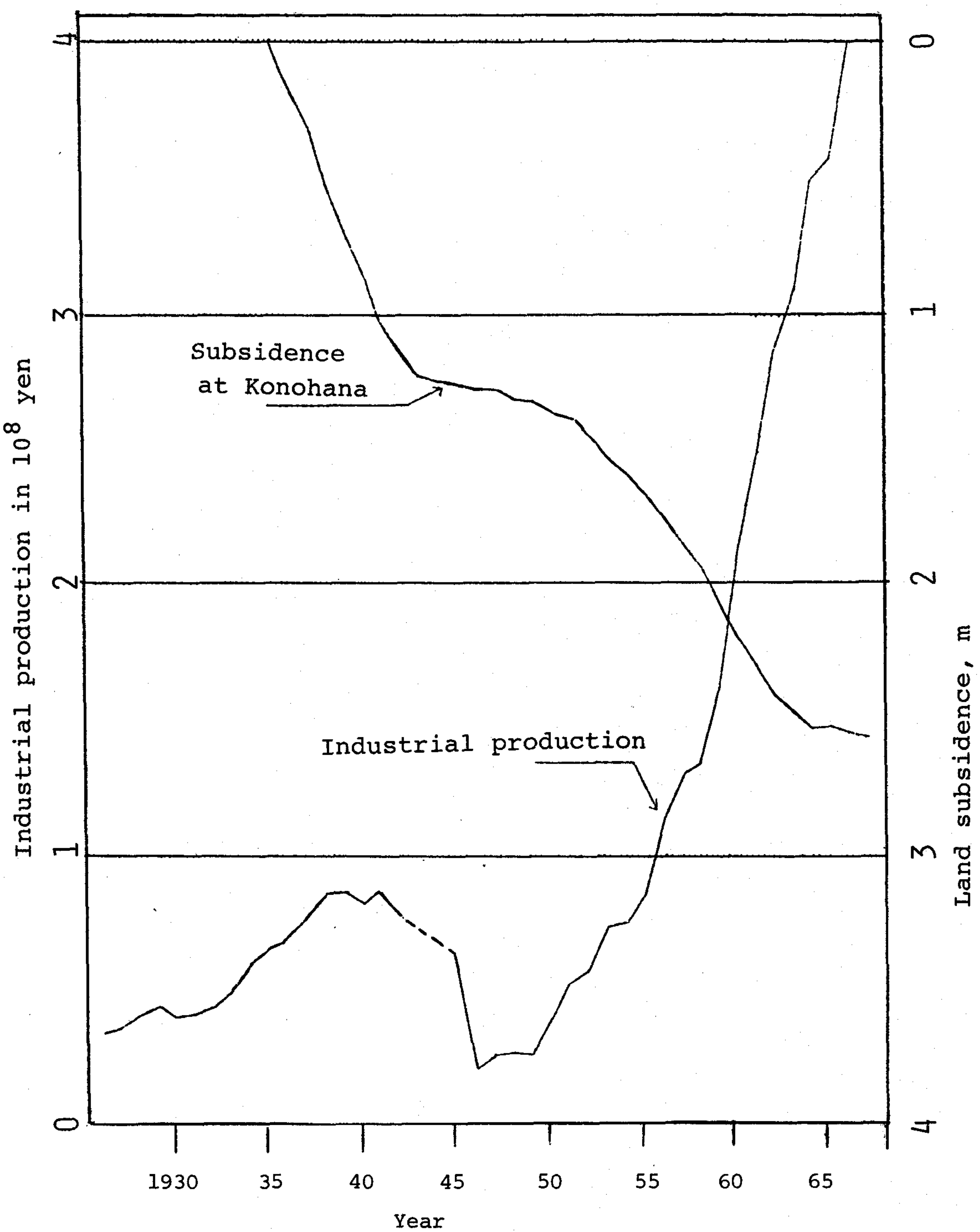


Figure 9.5.6 Land subsidence and industrial production in Osaka Prefecture. (Industrial production is inflated to the value of 1965.)

Besides these prevention works, the use of ground water has been gradually regulated in accordance with the progress of the industrial water supply works for delivering surface water, which was planned as the substitute for ground water. In 1962, Osaka City constructed industrial water supply works and in 1970, the Osaka prefectural government also constructed industrial water supply works introducing water from the River Yodo which started from Lake Biwa, the biggest lake in Japan.

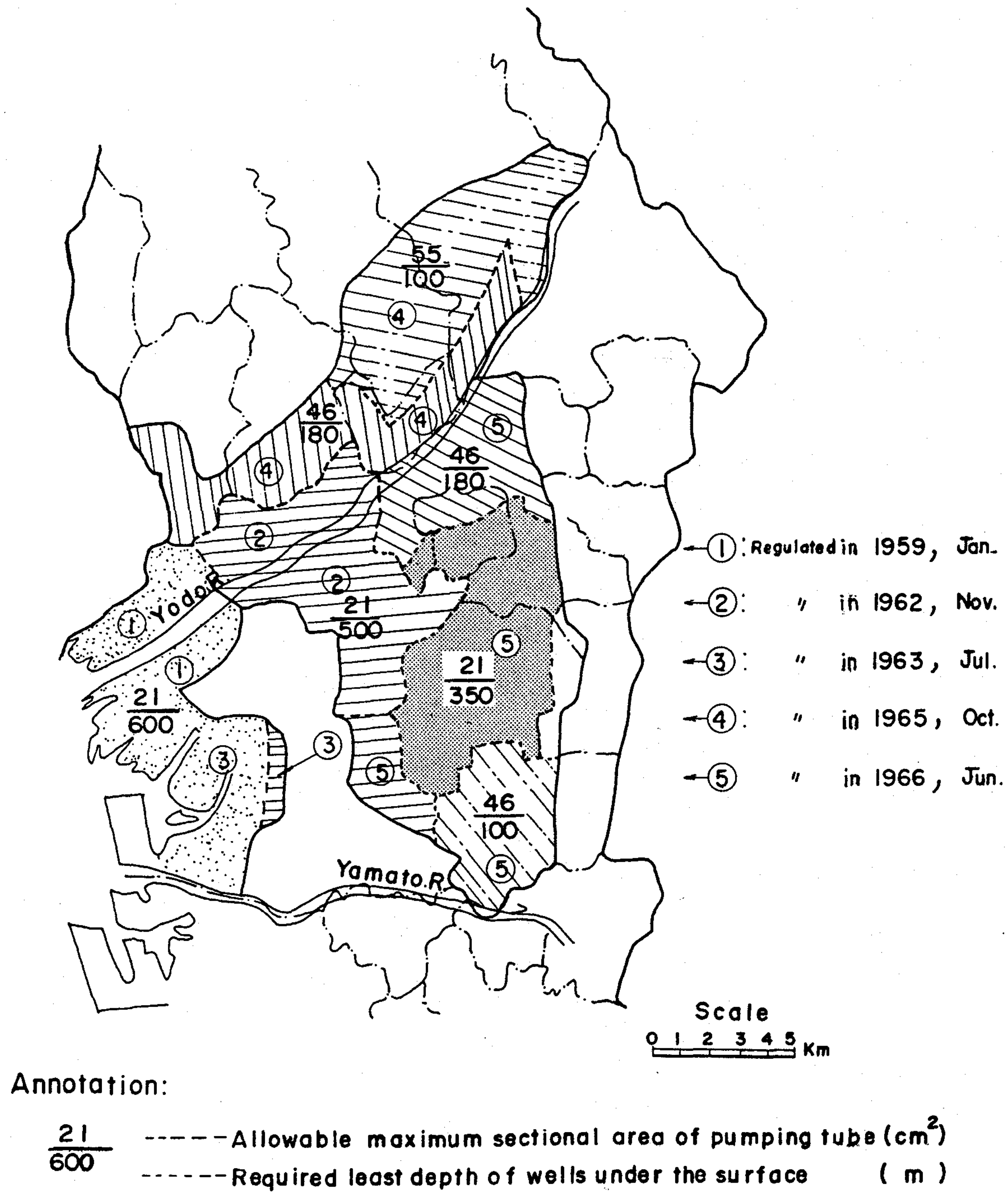


Figure 9.5.7 Regulated areas against use of ground water for industry.

Because of the regulation against the use of ground water in Osaka City land subsidence in the city gradually decreased and has almost stopped at present (Figures 9.5.7 and 9.5.8).

9.5.6 SELECTED REFERENCES

MURAYAMA, S. 1969. Land subsidence in Osaka, IASH Pub. No. 88, p. 105-130

NAKAMACHI, H. 1977.Land subsidence in Osaka, Soil and Foundation, 25-6, p. 61-67.

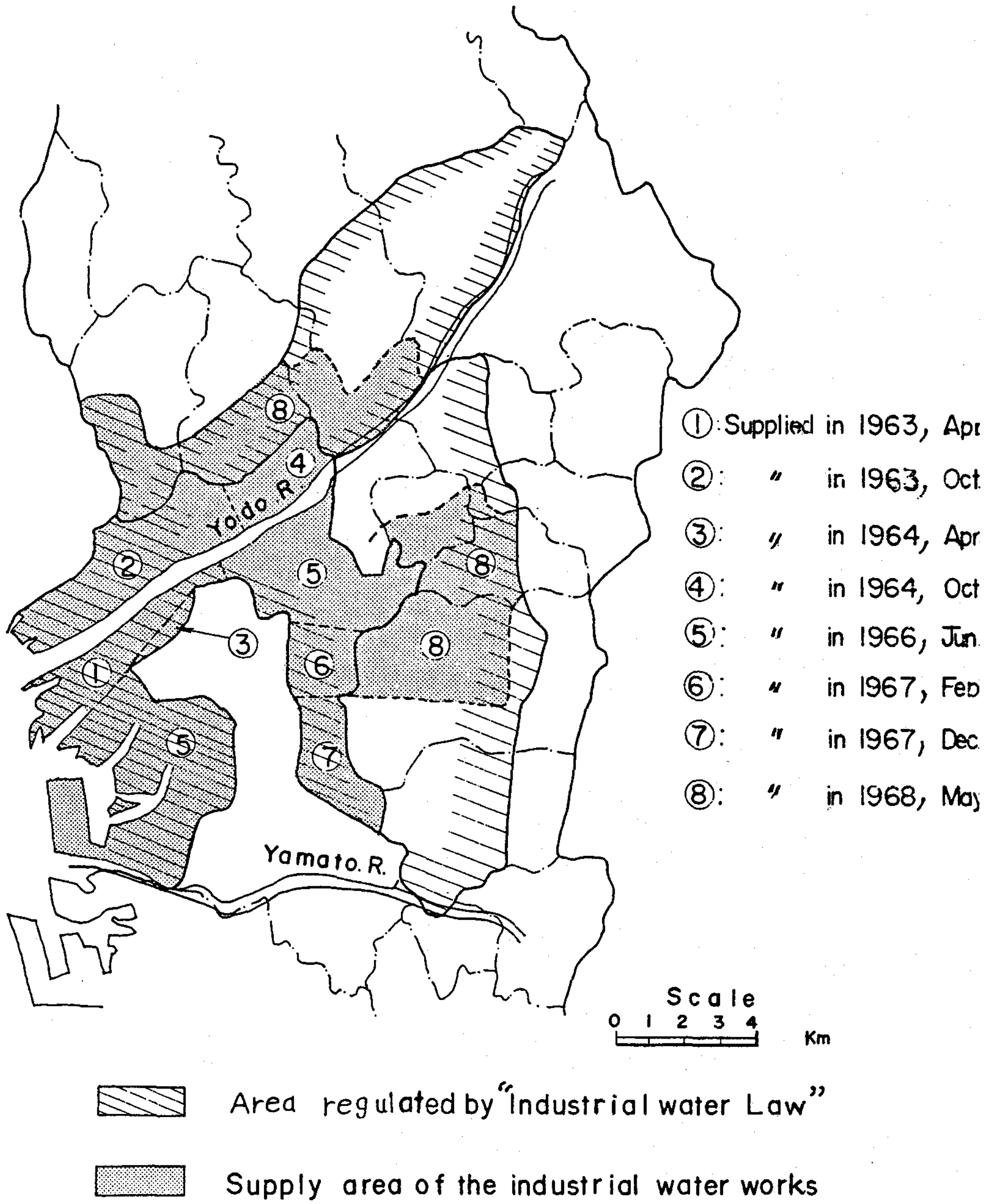


Figure 9.5.8 Supply area of industrial waterworks.

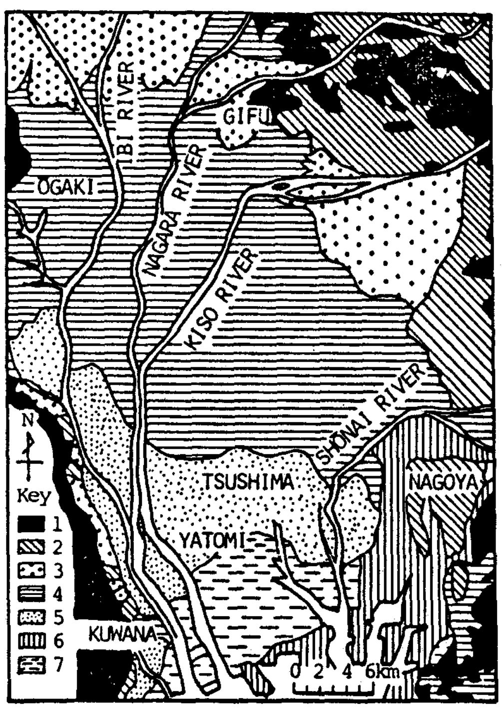
Case History No. 9.6. Nobi Plain, Japan, by Soki Yamamoto, Rissho University, Tokyo, Japan

9.6.1 TOPOGRAPHY AND GEOLOGY OF NOBI PLAIN

The Nobi Plain underlain by young sediments is situated in the central part of Japan and is about 1800 km^2 in area. This plain faces Ise Bay, where the Ibi, Nagara, Kiso, and Shonai rivers discharge, and is composed of alluvial fans, flood plains, deltaic plains, terraces, reclaimed lands, and filled-up ground (Figure 9.6.1).

The west-east profile of the southern part of this area is illustrated in Figure 9.6.2. The basement block in the Nobi Plain area bounded by the Yoro fault has tilted and is covered with sediments dipping westward. The depth of strata is about 2,000 m to the basement rocks.

The subsurface stratigraphy of these sediments in the plain has been explored on the basis of borehole material obtained from several thousands of water wells and test borings. The



1. Mountain and Hill 2. Terrace
3. Alluvial Fan and Cone
4. Flood Plain 5. Deltaic Plain
6. Filled-up Ground
7. Reclaimed Land

Figure 9.6.1 Topographic features of the Nobi Plain.

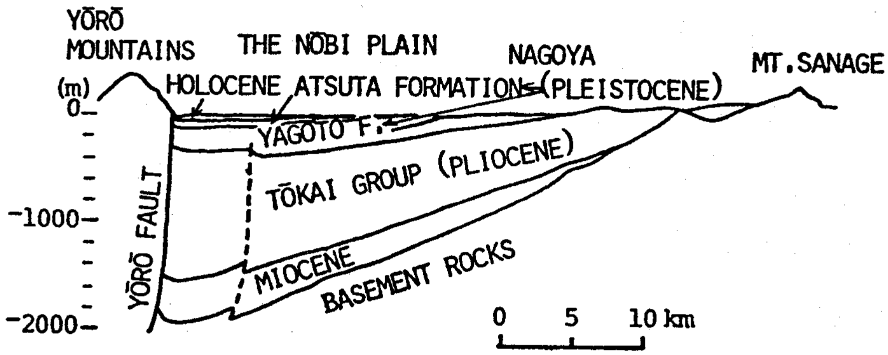


Figure 9.6.2 West-east profile of the Nobi plain.

Table 9.6.1 The Subsurface Stratigraphy in the Nobi Plain

		(Thickness)
HOLOCENE	NANYO FORMATION (H) (loose upper sand bed and very soft marine clay bed)	10-60 m
PLEISTOCENE	NOBI FORMATION (N) (alternation of sand and silt bed)	10-20 m
	DAICHI GRAVEL BED (G1)	10-30 m
	ATSUTA FORMATION (D3) (upper sand and clay beds and unconsolidated lower marine clay bed)	10-100 m
	DAINI GRAVEL BED (G2)	5-30 m
	AMA FORMATION GROUP (alternations of semiconsolidated sand, clay and gravel beds)	30-100 m
	PRE-AMA FORMATION GROUPS (alternations of semiconsolidated sand, clay and gravel beds)	30-70 m
PLIOCENE	TOKAI GROUP (alternations of semiconsolidated clay, sand and gravel beds)	200-1000 m
	M I O C E N E S E R I E S	
P R E - T E R T I A R Y B A S E M E N T R O C K S		

geological succession of these sediments is shown in Table 9.6.1, and a geologic cross section in Figure 9.6.3.

The middle Pleistocene and younger sediments are composed of an alternation of clay, sand and gravel beds. Changes in sedimentary environments and climatic fluctuations of these sediments have been studied by means of microfossil analyses of numerous core samples (Nobi Plain Quaternary Research Group, 1976).

Semiconsolidated fresh-water lacustrine clay beds and fluvial sand and gravel beds are interbedded in the lower horizon of the Pleistocene sediments, the so-called Pre-Ama Formation Groups. The Ama Formation Group and the younger sediments are composed of alternations of fluvial sand or gravel beds and unconsolidated marine clay beds, deposited under inner bay conditions. Each of these marine clay beds shows a sedimentary cycle from transgressive to regressive phase, and represents a relatively warm period, interglacial epoch or interstadial. In colder periods, the gravel beds have been deposited as either terrace or river-bed gravels

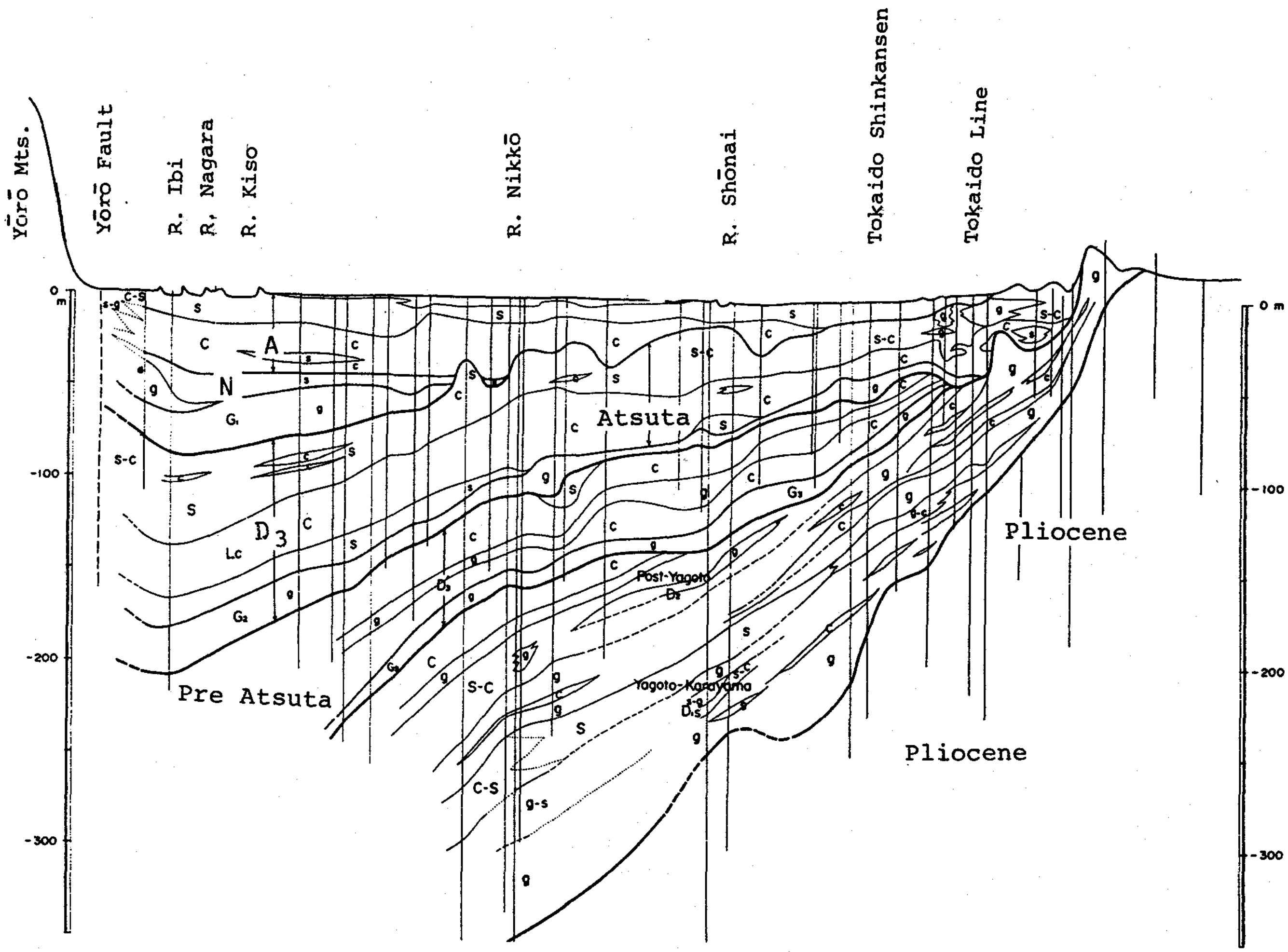


Figure 9.6.3 Geologic cross sectiopn of Nobi Plain.

in the valleys formed during the period of low sea level falling. The river-bed gravel deposited in the bottom of the valleys during a maximum stage of sea level falling reaches 20 m or more in thickness. The terrace gravel beds are generally thinner than the valley bottom gravel beds. These two types of gravel beds are distributed under almost the whole area of the Nobi Plain. Buried topography such as hills, terrace, and valleys formed in the process of sea-level lowering is depicted in the base contour map of these gravel beds. The buried topography and types of gravel beds affect ground-water yield in this plain. The marine clay beds overlying the gravel beds have attained more than 30 m in thickness in the valleys, and spread far and wide under the plain area except for the alluvial fan area, where the clay beds thin out and grade into sand or gravel beds (Figure 9.6.3).

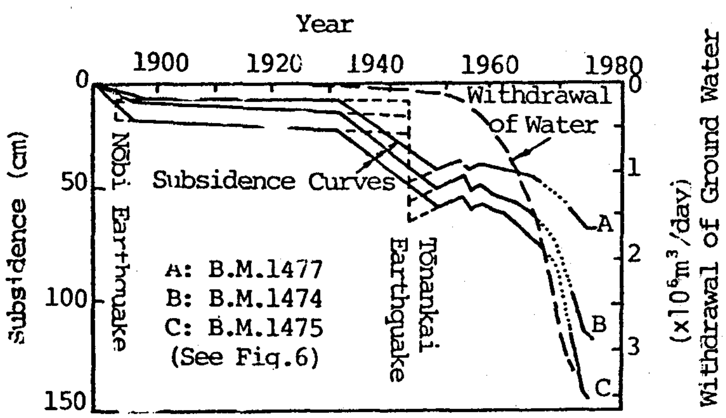


Figure 9.6.4 Subsidence of bench marks and withdrawal of ground water in the Nobi Plain.

Table 9.6.2 Withdrawal of ground water in the Nobi Plain.

Year	m^3/day
1925	1638
1945	129088
1950	154040
1955	360301
1960	849338
1965	1552764
1970	3002128
1973	3514195

9.6.2 HYDROLOGY

In the alluvial fan area, precipitation and surface water percolate downward through these permeable sand and gravel beds, and recharge ground water in the Nobi Plain. A superficial sand bed, the upper member of the Nanyo Formation, as much as 15 m thick, contains unconfined ground water which is recharged directly from precipitation and infiltration of irrigated water. The gravel beds, Gl, G2, and others interbedded in clay beds, are artesian aquifers and supply a large quantity of ground water. Before the pumping was largely developed, many flowing wells tapped these gravel beds in most of the Nobi Plain.

The increasing withdrawal of ground water in the Nobi Plain is shown in Tables 9.6.2 and 9.6.3. The use for industry amounts to 60 per cent of the total.

Recent annual withdrawals of ground water in the Nobi Plain equal 32 per cent of the annual rainfall on this plain. This volume is much larger than the natural recharge of ground water.

In the 1920’s, the piezometric levels of confined aquifers were above the ground surface in most of this plain. In the 1940’s, flowing wells were still observed in Ogaki, Kanie, and Kasugai districts. But since then, the piezometric levels have declined due to the increase in the number of artesian wells.

Table 9.6.3 Use of ground water for each purpose in the Nobi Plain (1973).

Item	Total	Industry	Buildings	Water Supply	Agriculture
Withdrawal of Ground Water (m^3/day)	3,802,293	2,290,015	343,025	477,028	692,225
Percentage	100	60	9	13	18

9.6.3 LAND SUBSIDENCE

The records of three bench marks depicted in Figure 9.6.4 show the evolution of land subsidence in this area. During the period from 1950 to 1973, the subsidence increased exponentially and some areas subsided more than 20 cm in 1973.

Figure 9.6.5 shows a comparison between the areas subsiding more than 2 cm/yr from Feb., 1961 to February, 1962, and from November, 1972 to November, 1973. This figure illustrates how the subsiding area in this plain enlarged from 1961 to 1973.

The subsidence of this plain during about 15 years from February, 1961 to November, 1975 is shown in Figure 9.6.6. The southern part of Nagashima facing the Ise Bay settled 147 cm during these 15 years. The total subsiding area is 1140 km^2 (Environment Agency, Japan, 1976). By 1973, 363 km^2 had become lower than the mean high-sea level (1.1 m higher than the mean sea level), 248 km^2 lower than the mean sea level, and 37 km^2 lower than the mean low-sea level (1. 4 m lower than the mean sea level). The area below mean sea level enlarged from 186 km^2 in 1961 to 248 km^2 in 1973.

Regulations for withdrawal of ground water in the Nobi Plain are as follows:

1. The Industrial Water Law (established in 1956)
   The areas designated by the Industrial Water Law are supplied with industrial water from surface sources instead of restriction on pumping of ground water. The regulations for these areas are shown in Table 9.6.4.

2. Regulations by Ordinance of Aichi Prefecture
   <u>Aichi Regulation Zone I</u> (enforced on 30 September 1974)
   This regulation zone was decided considering the rate of subsidence greater than 5 cm/year in 1972 and/or 1973. In this zone, a newly bored well must meet the following conditions;
   a. The depth of strainer should not be greater than 10 m.
   b. The inside area of discharge pipe should be less than 19 cm^2.
   c. The power of motor should be less than 2.2 kw.
   d. Total discharge should be less than 350 m^3/day.

   Concerning the wells that existed before the regulation, flow meters were installed in them and the discharge records are reported to the Prefectural office every year. Since the lst of January, 1976, the withdrawal of ground water from existing wells was restricted within 80 per cent of the discharge in the past (Figure 9.6.7).
   <u>Aichi Regulation Zones II and III</u> (enforced on 1 April 1976)
   The regulations for newly bored wells and existing wells are the same as those for Zone I, except that for the existing wells in Zone II, the withdrawal from the lst of April, 1977 is going to be restricted within 80 per cent of the discharge in the past, and for the existing wells in Zone III, the withdrawal in future is going to be restricted within the discharge in the past.

3. Regulations by Ordinance of Mie Prefecture (enforced on 1 April 1975)
   The regulations for newly bored wells and existing wells are the same as those explained for Aichi Regulation Zones II and III, where Mie

Table 9.6.4 Regulations for the Areas Designated by the Industrial Water Law

Area	Zone (See Fig. 9.6.7)	Allowed Depth of Strainer	Inside Area of Discharge Pipe
Southern	N_1	deeper than 80 m	less than 46 cm^2
Industrial Area of		deeper than 300 m	greater than 46 cm^2
Nagoya	N_2	deeper than 90 m	less than 46 cm^2
designated in 1960		deeper than 180 m	greater than 46 cm^2
Industrial		deeper than 100 m	less than 21 cm^2
Area of	Y_l	deeper than 230 m	from 21 cm^2 to 46 cm^2
Yokkaichi designated			
in 1957	Y_2	deeper than 50 m	less than 21 cm^2
and 1963		deeper than 150 m	from 21 cm^2 to 46 cm^2

Table 9.6.5 Dealing with existing wells in the case where the strainers are deeper than 10 m in Nagoya

Zone	Wells for Buildings	Wells for Industry
I	Changed to use city water since 16 Nov. 1975	Change to the industrial water supply as soon as possible
II	Changed to use city water since 16 Nov. 1976	
III	Increasing withdrawal is forbidden	

Regulation Zones I and II correspond to Aichi Regulation Zones II and III, respectively.

4. Regulation by Ordinance of Nagoya City (enforced on 16 November 1974) According to the ordinance of Nagoya City, a new well can be allowed only in the case where the strainer is not deeper than 10 m and the inside area of discharge pipe is less than 19 cm^2. Existing deep wells are being treated as shown in Table 9.6.5. The regulation zones of Nagoya City are overlapped by the regulation zones of Aichi Prefecture. Therefore, the withdrawal of ground water in Nagoya is controlled by ordinances of Nagoya City and Aichi Prefecture.

Figure 9.6.8 shows the subsidence of Nagashima during the recent ten years and piezometric levels of the lst confined aquifer (G_l) and the 2nd confined aquifer (G_2) measured at Matsunaka observation well during the recent five years. The confined aquifers G_1 and G_2 are located at depths between 40 m to 60 m and 100 m to 115 m respectively at the observation site. Concerning the piezometric levels, seasonal changes are superposed on total trends of ground water levels. The seasonal drops of piezometric levels are caused by the increase of pumpage for cooling and

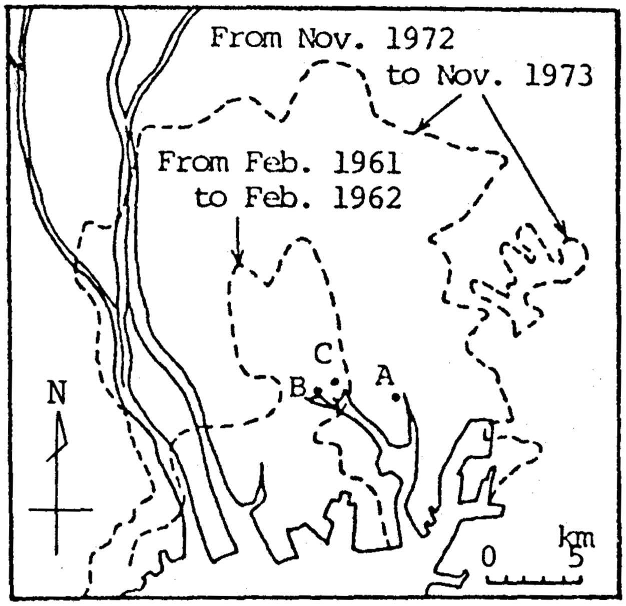


Figure 9.6.5 Enlargement of the area subsiding more than 2 cm/yr.

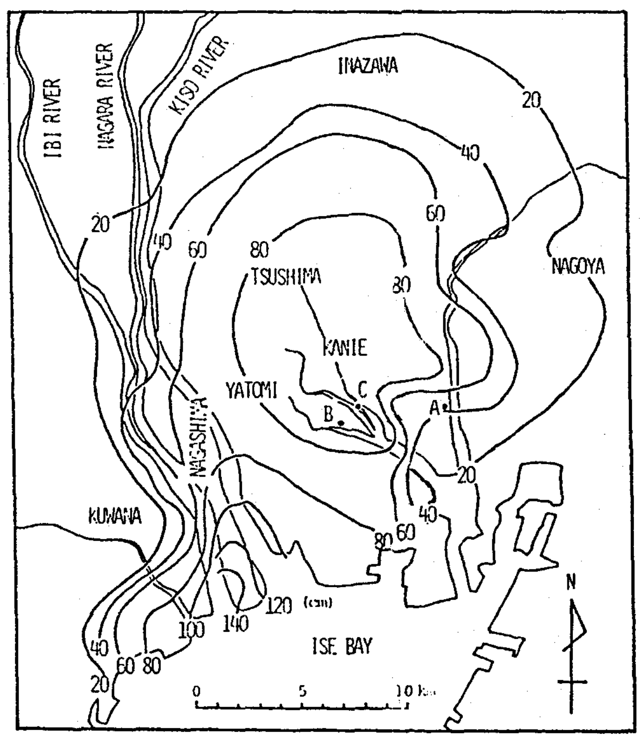


Figure 9.6.6 Subsidence for 15 years from February 1961 to November 1975 in the Nobi Plain.

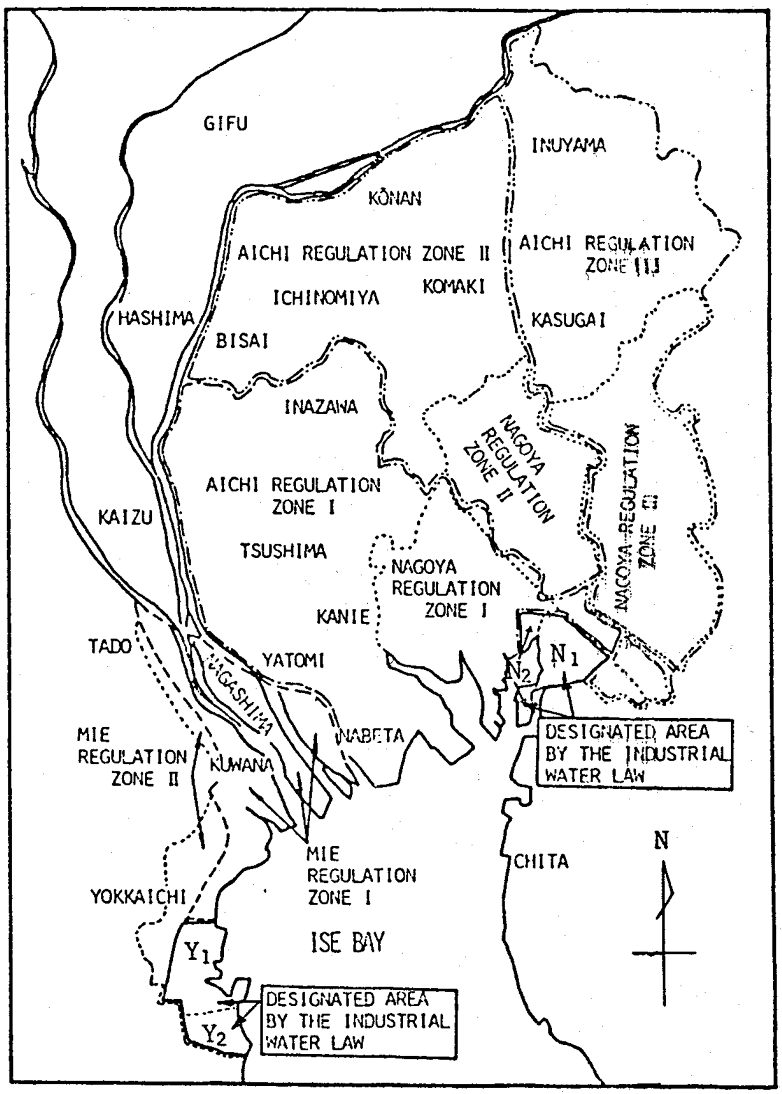


Figure 9.6.7 Restriction of withdrawal of ground water in the Nobi Plain.

irrigation in summer. The piezometric level of the deeper confined aquifer is lower than the one of the shallower confined aquifer, because ground water of better quality is pumped up in plenty from the deep aquifer.

The piezometric levels of aquifers show, recovering trends since 1974. Reflecting this favorable turn in the ground water situation, the subsiding area became smaller and the rate of subsidence decreased.

Moreover, several rebounding areas appeared around the area having reduced subsidence. (See Figure 9.6.9.)

9.6.4 PARAMETERS

The compression index C_c varies vertically and horizontally in clay layers and the values are closely related to the sedimentary facies. The value is the largest in the middle horizon of finer materials. Lateral distribution of the mean C_c value calculated by averaging vertical

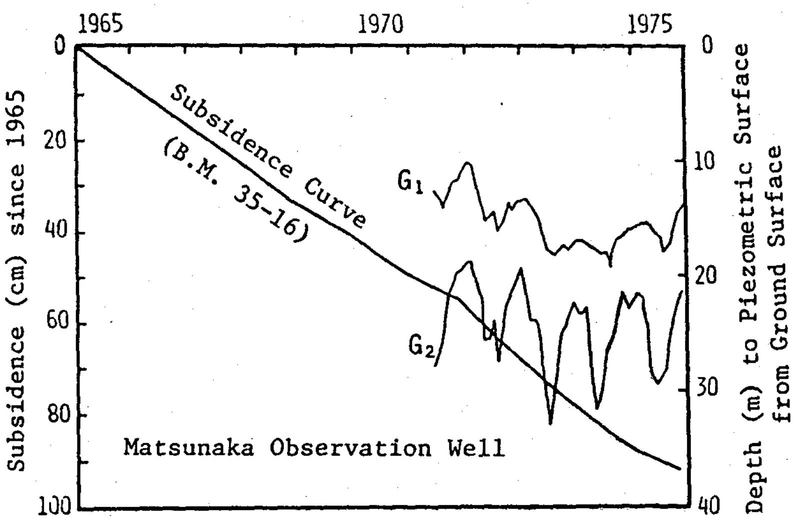


Figure 9.6.8 Subsidence and ground water conditions at Nagashima during recent years.

variations reflects the sedimentary environment of the clay bed. It varies from 0.3 to 0.8 from the margin of this plain to the central area. There is a good correlation between e_o and C_c which can be expressed as follows:

$$C_c = 0.5 (e_o - 0.5)$$

where e_o is the natural void ratio of clay.

9.6.5 SELECTED REFERENCES

IIDA, K., SAZANAMI, T., KUWAHARA, T., and K. UESHITA. 1977. Subsidence of the Nobi Plain, IAHS Pub. No. 121, p. 47-54.

KUWAHARA, T., UESHITA, K., and K. IIDA. 1977. Analysis of land subsidence in the Nobi Plain, IAHS Pub. No. 121, p. 55-64.

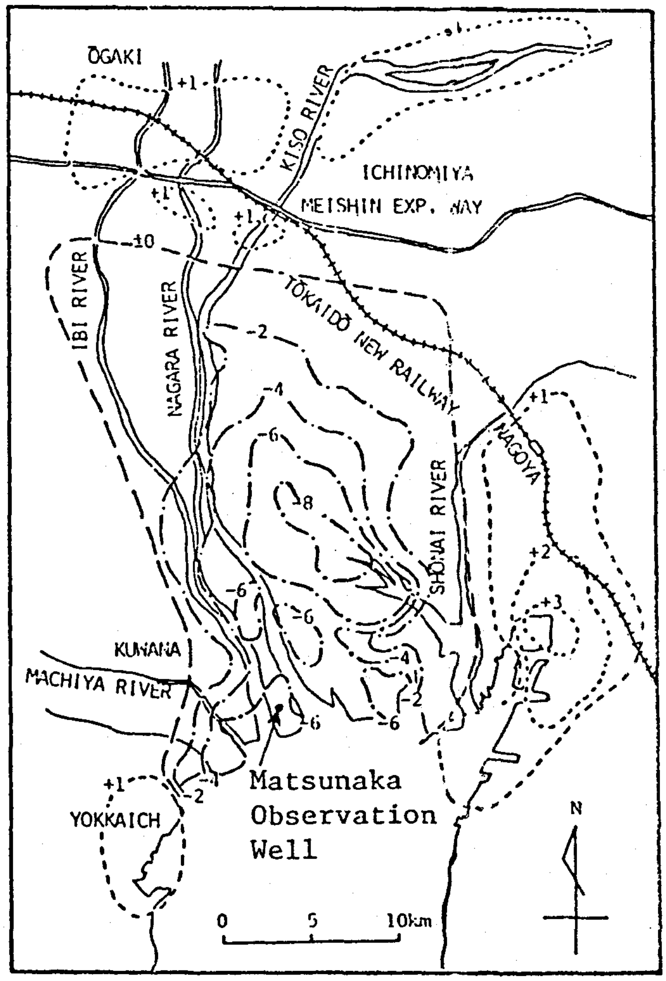


Figure 9.6.9 Subsidence and rebound of the Nobi Plain from November 1974 to November 1975 (unit: cm/yr).

Case History No. 9.7. Niigata, Japan, by Soki Yamamoto, Rissho University, Tokyo, Japan

9.7.1 TOPOGRAPHY AND GEOLOGY

Niigata Plain is the largest coastal plain along the Japan seacoast. It is bounded on the east by mountains, on the south and west by hills and on the north by the Japan Sea, with coastal sand dunes. Through the center of this plain the R. Shinano, the R. Agano and their branches flow down from south to north into the Japan Sea. Along these rivers, especially on down reaches, there are many back marshes, swamps, and lakes (Figure 9.7.1.).

This area is a typical synclinal basin with deposits of Cenozoic age underlain by a basement complex (Figure 9.7.2.). The subsurface geology of this area consists of Holocene, Pleistocene and Neocene deposits. The stratigraphic correlation is as shown in Table 9.7.1.

9.7.2 HYDROLOGY

It had been well known that there was abundant methane gas in this area. The major gas reservoirs in the Niigata gas field belong to the Uonuma Group of Pleistocene age which is characterized by the alternation of clay, sand and gravel beds. Gas reservoirs, i.e., confined aquifers consisting of sand and gravel, are filled with brackish to saline water.

Large quantities of saline ground water containing dissolved gas are pumped up from wells as much as 1000 metres deep, tapping relatively unconsolidated segments of the G_l, G_2,....G_7 layers of Cenozoic age (Figure 9.7.3.). Shallow aquifer G_1 is exploited for domestic use, but the others are exploited for industrial uses (Figures 9.7.4. and 9.7.5.). About 200 million cubic metres of methane was produced in 1958, 60 per cent of the methane gas production in Japan.

The producing wells increased rapidly to 1959 near the mouth of the R. Shinano along the coast of the Japan Sea. With increase of the amount of ground-water withdrawal from gas wells, the rapid lowering of the groundwater level was recognized. The permeability of the gas reservoirs varies from 50 to 200 millidarcys.

9.7.3 SUBSIDENCE

Although no attention had been paid, about 1930 the land subsidence in Niigata was indicated by geologists as "Pseudo-sinking" of sea coast. Subsidence of the Niigata area, especially the harbor district, became noticeable by 1955 (Figure 9.7.6.). Most of the harbor area which initially was only 1-2 m above mean sea level had been damaged by inundation from the sea.

The results of first-order leveling over this area were reexamined and extraordinary subsidence was recognized. Another area of surface subsidence had occurred in the inland part of the Niigata Plain, in the southern part of Niigata city. In 1959, the center of the subsidence was located near Shirone town, and the subsidence rate was about 14 cm in ten months. Based on the data of compaction meters, it was concluded that the compaction was located in the zone shallower than 120 m depth, which is correlated with the Holocene and younger Pleistocene deposits. As shown in Figure 9.7.7, about 10,000 wells producing natural gas for domestic use are located in this area. The amount of the ground-water withdrawal from these wells in 1960 was estimated to be approximately 60,000 m^3/day.

First-order level surveys have been made in Niigata and its vicinity by the Japan Geographical Survey Institute in 1898, 1930, 1951, 1955 and 1957, and leveling has been conducted at 6-month intervals since 1957. (See Figure 2.2.) The subsidence of 20 cm at the harbor mouth in 6 months indicates an annual rate of 40 cm. By 1959, the annual rate had increased to 54 cm a year.

Long-term graphs of elevation change for five bench marks which are representative of the subsidence trend are shown in Figure 9.7.8. They show a slow subsidence of about one half a centimetre per year from 1898 to 1952 and a rapid acceleration since 1955. The cause of the slow

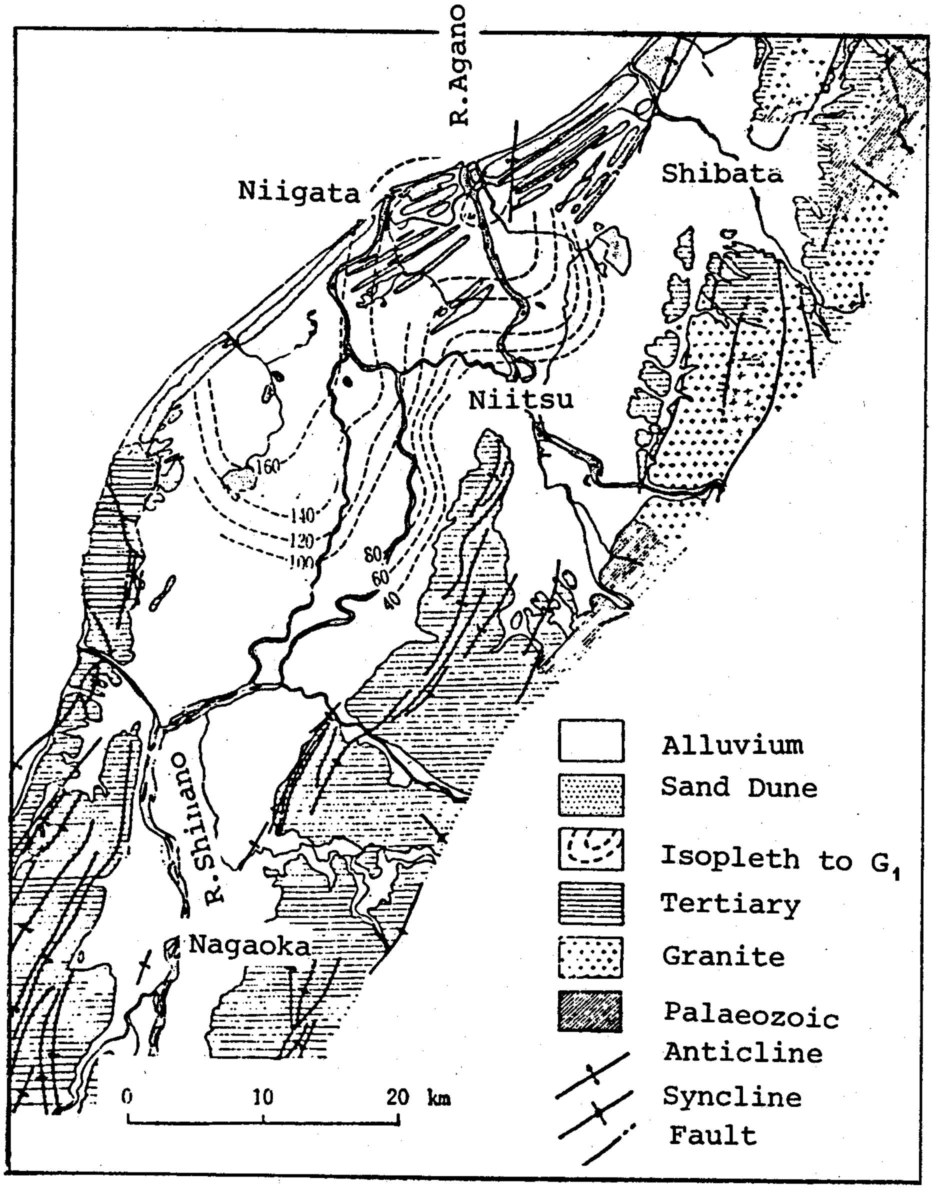


Figure 9.7.1 Physiographic map of Niigata area.

subsidence prior to 1952 has not been explained. Natural gas production, shown in Figures 9.7.4 and 9.7.5, began about 1947 and increased rapidly in the fifties. The volume of gas-bearing water pumped reportedly has been about equal to the volume of gas recovered. Because of the coincidence in time and place of the increase in rate of subsidence and of fluid withdrawal, we concluded that the cause of the accelerated subsidence is the withdrawal of the water and gas. This withdrawal has decreased the fluid pressure in the gas-bearing zones and has caused the compaction of sediments.

The rate and distribution of compaction in depth is being observed by means of 12 novel compaction recording wells, installed in 1958-1959, and ranging in depth from 20 to 1190 m. When the results obtained from these observation wells were analyzed, they showed a remarkable contraction of the layer from 380 to 610 m depth. The subsiding area of Niigata districts is about 430 km^2.

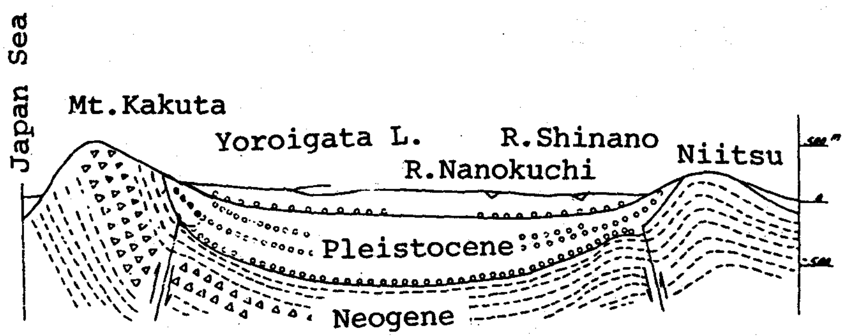


Figure 9.7.2 Diagrammatic cross section of Niigata.

9.7.4 PARAMETERS

M_V	1×10^{-2}	-	5×10^{-2}	CM^2/kg
C_V	1×10^{-1}	-	1×10^{0}	CM^2/min.

9.7.5 LEGAL ASPECTS AND COUNTERMEASURES

Since 1960, control of ground-water withdrawal has been undertaken by putting area "A" completely under the ban of gas production (Figure 9.7.9). They also established area "B," where extraction of gas from shallower reservoirs than G_6 is prohibited, and "C," where gas production is permitted within the limit of the past production record. Over all this area, new drilling is prohibited.

Besides legal restrictions, hydrogeologists had carried on experiments of water injection into gas reservoirs since 1966 or so. The injectivity index is usually less than a quarter of the productivity index. After 1965, the gas company started to inject saline water into four reservoirs (G_{4-1}, G_5, G_{5-1}, G_6). The change of ground-water level and compaction of these layers has been observed in this area (Figures 9.7.10, 9.7.11, 9.7.12, and 9.7.13). Since 1973, all the pumped water, about 110,000 cubic metres per day, after gas separation, has been injected into the gas reservoirs through injection wells, with no surface drainage.

The total estimated cost of countermeasures over the whole Niigata area is difficult to estimate but the direct cost is estimated as $12 million for the period 1957 to 1974.

9.7.6 SELECTED REFERENCES

AOKI, S. 1977. Land subsidence in Niigata. IAHS Pub. No. 121, p. 629-634.

HOKURIKU AGRICULTURAL BUREAU. 1965. Land subsidence of agricultural land in Niigata, pp. 1-485.

ISHIWADA, Y. 1969. Experiments on water injection in the Niigata gas field. IASH Pub. No. 88, pp. 629-634.

Table 9.7.1 Geological correlation and gas reservoir.

Age	Division	Lithology	Thickness
HOLOCENE	I	Silt & clay alternation	120 - 160 m
	II	Silty clay	
	III	Silt, clay and sand	
	IV	Peat, clay & sand alternation	
	V	Clay-silt, Gravel, sand and silt G_1	
PLEISTOCENE (Uonuma - Kanbara)	upper	G_2 Sand & gravel G_3	310 - 660 m
	middle	G_4 Mudstone,Conglomerate and sandstone	
	lower	G_{4-1} Mudstone G_5 sandstone & conglomerate	
NEOGENE	Haizume	G_{5-1} Conglomerate G_{5-2} Conglomerate G_6 Conglomerate	500 - 700 m
	Nishiyama	G_7 Sandstone G_8 Sand & gravel	

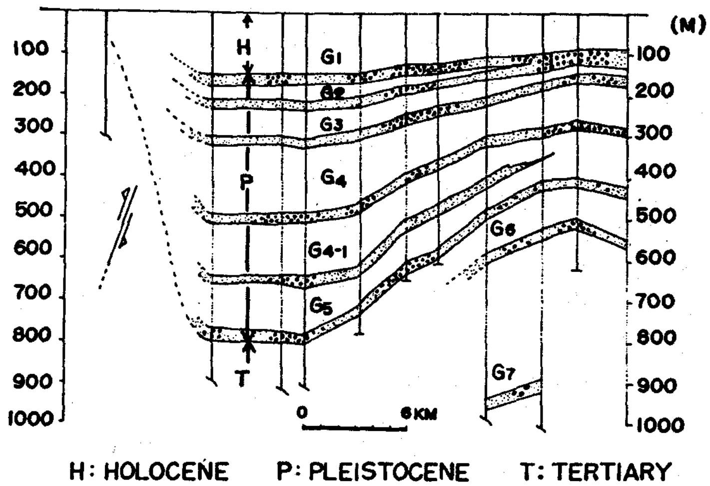


Figure 9.7.3 Geologic profile.

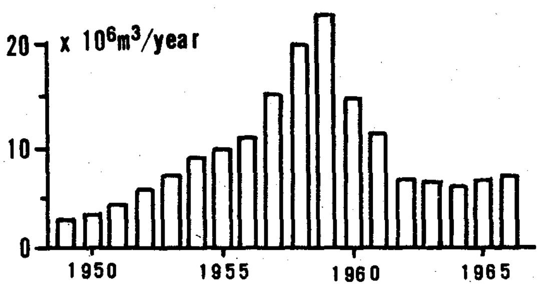


Figure 9.7.4 Annual amount of withdrawal of gas water for industrial use.

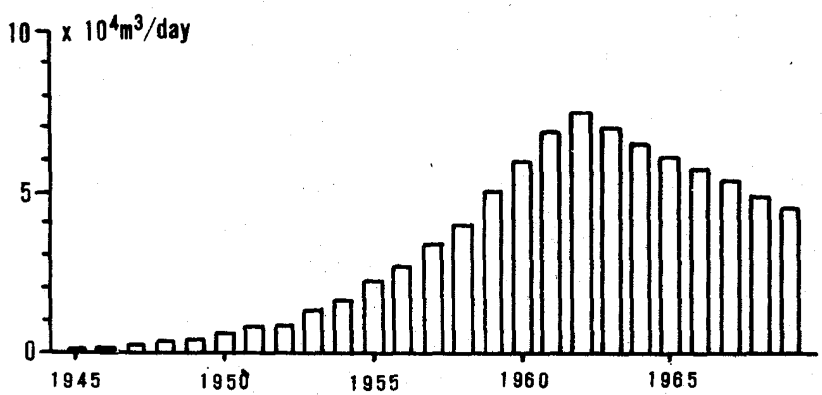


Figure 9.7.5 Daily amount of withdrawal of gas water for domestic use.

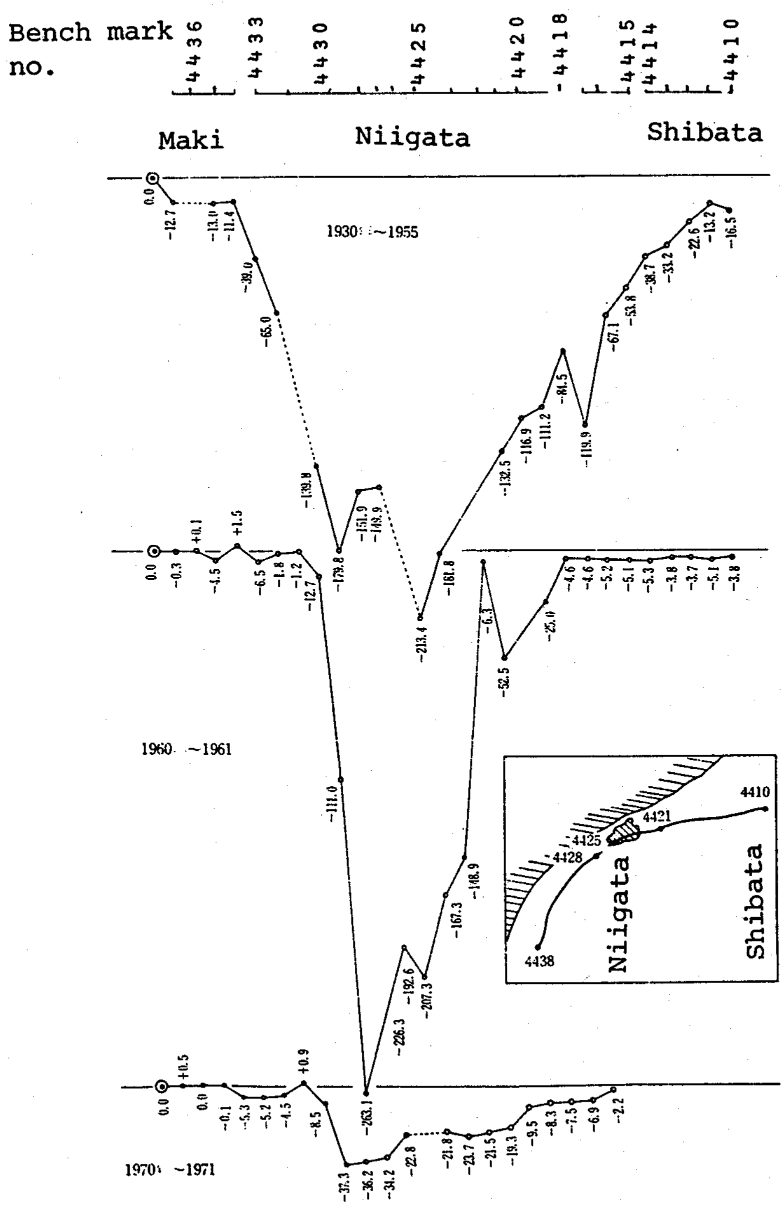


Figure 9.7.6 Subsidence of bench marks on selected points in Niigata and vicinity, in millimetres.

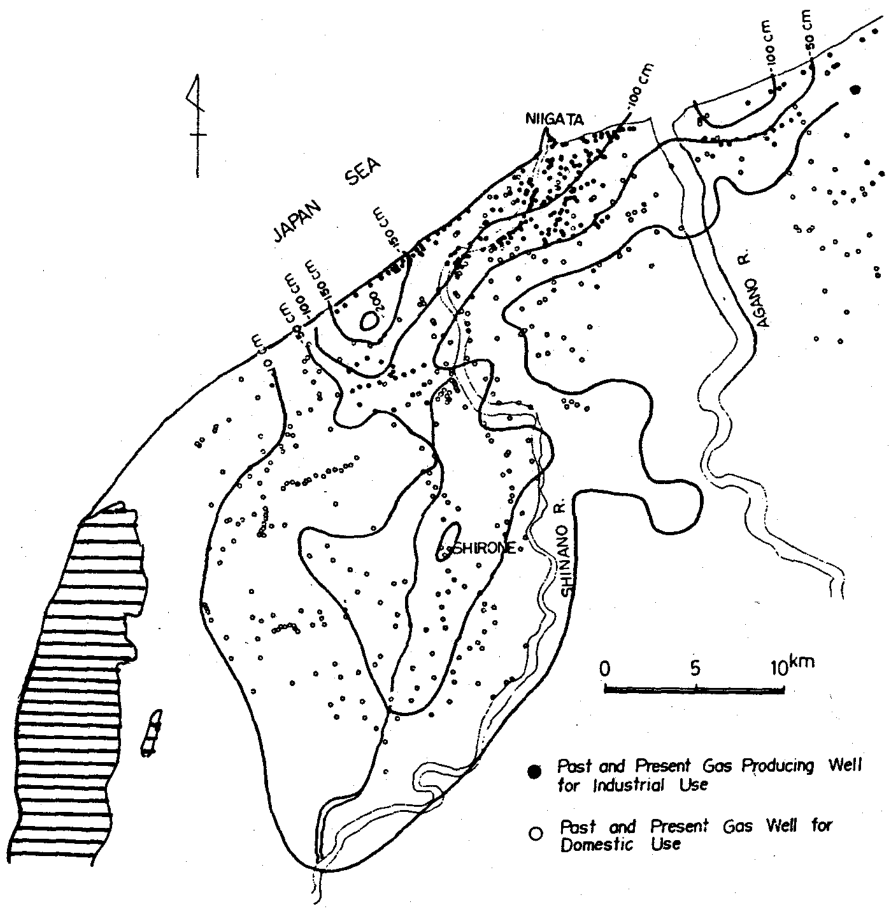


Figure 9.7.7 Distribution of gas wells and total amount of land subsidence during the period 1959-74.

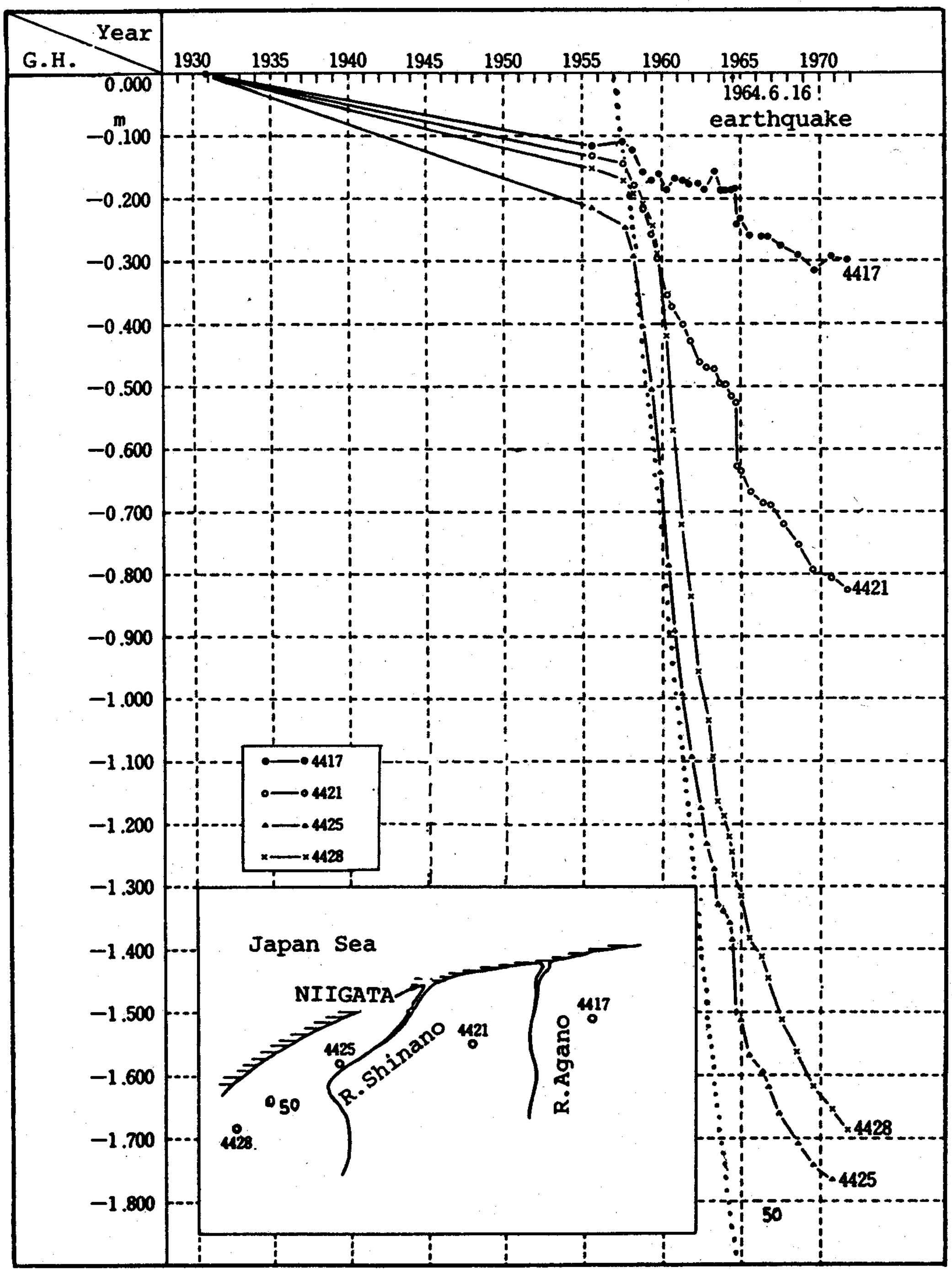


Figure 9.7.8 Change of ground height in Niigata (m).

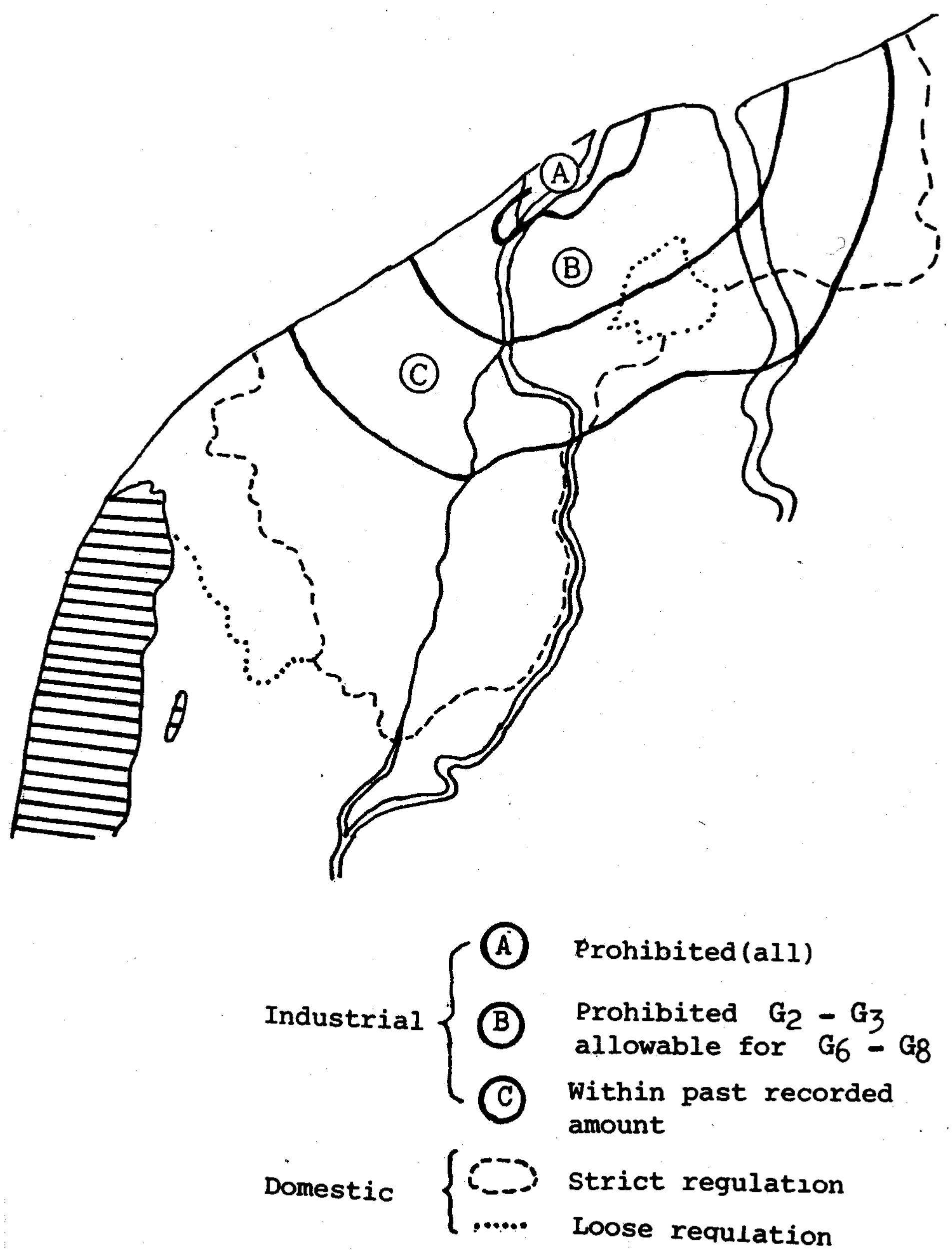


Figure 9.7.9 Regulation by law in Niigata area.

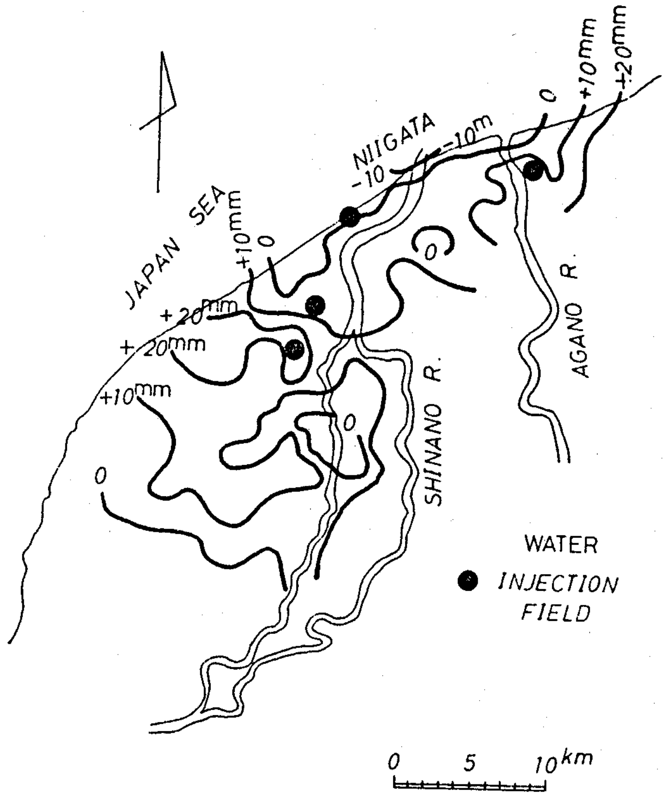


Figure 9.7.10 Land subsidence during 1973-74.

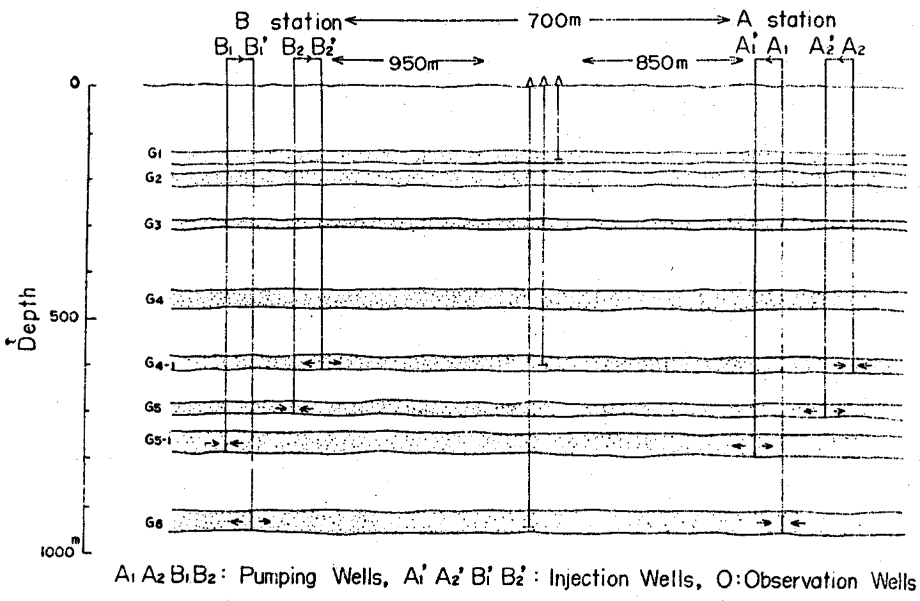


Figure 9.7.11 Profile of water injection experimental station (at Kurosaki).

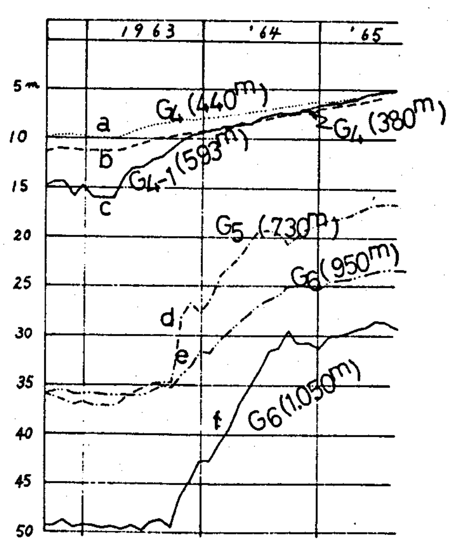


Figure 9.7.12 Change in ground-water levels at observation wells (Kurosaki).

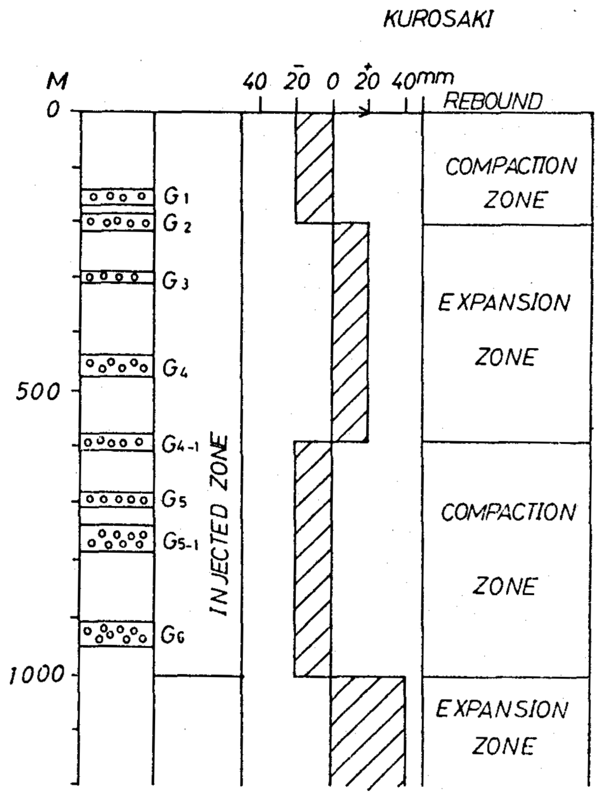


Figure 9.7.13 Columnar section showing the position of expansion and compaction layers at Kurosaki.

Case History No. 9.8. Mexico., D. F., Mexico, by Germán E. Figueroa Vega, Comisión de Aguas del Valle de México, Mexico, D., F.

9.8.1 GEOLOGY

Mexico City is located in the southwestern portion of the Valley of Mexico. The general geological features of the zone are shown in Figure 9.8.1. in which it may be seen that the most ancient outcrops, in the upper part of the western and northern ranges of the zone, are andesitic and dacitic formations of the middle Tertiary, overlain on their slopes by volcanic and alluvial formations of the upper Tertiary and Quaternary (Comisión Hydrológica de la Cuenca del Valle de México, 1961b).

The southern range is almost completely covered by Quaternary basaltic emissions and the flatter portion of the city is constituted by Quaternary lacustrine clays. These clays overlie Quaternary clastics that constitute the aquifer whose overdraft has caused the subsidence. The definition of the symbols which appear in Figure 9.8.1. is given in Table 9.8.1.

The clayey formation has a variable thickness from 0 to 50 m. with some intercalations of fine sands and silts, void ratios up to 15 and water contents up to 650 per cent. As a consequence, its shearing strength is very low and its compressibility very high.

The aquifer contains thick strata of gravel and sand of good permeability. Wells are generally of high yield (180 to 360 m^3/hr or more) with specific capacities ranging between 18 and 36 m^3/hr/m and more.

The mechanical properties of Mexico city clays, especially their low shearing strength, make it necessary to carry out special soil mechanics studies practically in all types of foundations.

In general, foundations by continuous slabs are possible only in buildings with no more than 4 or 5 stories. in any building higher than this, it is necessary to resort to the use of compensated foundations which present difficulties due to stability problems in slopes and in bottoms of excavations. An easier alternative is the use of friction or point piles. In this way it has been possible to construct buildings up to 42 stories high.

Because of similar problems in the construction of sewage tunnels and their shafts, it has been necessary to use shields and compressed air and, in some cases, very special construction methods.

The Mexico City clays have been studied from a mineralogical standpoint by nuclear spectrography, electronic microscopy, and interchange of cations and thermic differential analysis to determine their composition. Their approximate composition is 80 per cent montmorillonite and 15 per cent kaolinite, with some beidellite, illite, and halloysite. The clayey materials are mixed with 2 to 20 per cent of the total weight of solids (mixtures of sands and fossils) to which some investigators attribute the elastic properties of clays (Marsal and Mazari, 1959).

9.8.2 HYDROLOGY

The portion of the valley which contains the City of Mexico has an area of approximately 958 km^2. The annual precipitation ranges from 60 mm in the lower zone to 1300 mm in the higher zone, with an average on the order of 890 mm per year.

The potential evaporation ranges between 1900 mm per year for the lower zone and 900 mm for the higher zone, with an average of the order of 1300 mm.

The mean runoff of the period 1948-60, within the 958 km^2, including the urban area of the city, was 20.5 per cent and in the nonurbanized area (238 km^2), 14.6 per cent (Comisión Hydrológica de la Cuenca del Valle de México, 1963).

Figure 9.8.2, which shows a north-south stratigraphic profile of the city, and Figure 9.8.3, which shows an east-west stratigraphic profile (Marsal and Mazari, 1959), allow us to appreciate that the permeable outcrops are in the slopes of the mountains. This is why those zones are the main recharge areas of the aquifer under exploitation. In spite of this, infiltration may occur in the clayey zone as happens in the northern part of the city which has

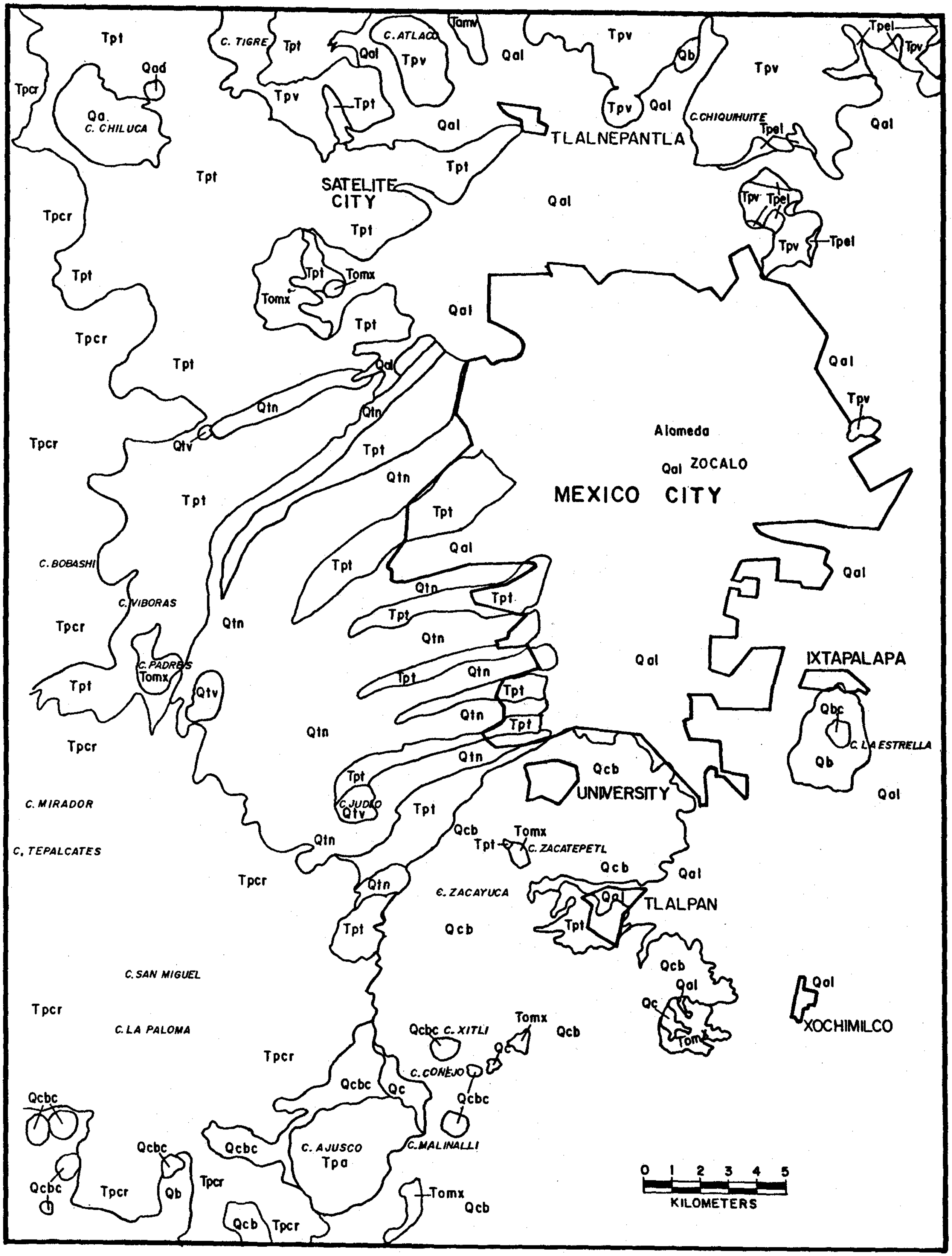


Figure 9.8.1 Geologic map (after Comisión Hydrológica de la Cuenca del Valle de México, 1961).

Table 9.8.1 Geological symbols and units.

Qal	Alluvials, lacustrine and clastic deposits	
Qb	Interstratification of lavas and tuffs	
Qbc	Ash cones	Quaternary basalt-andesite volcanic series
Qad	Andesite lava domes	
Qa	Andesite lava	
Qcb	Interstratification of lavas and basalt tuffs	Quaternary Chichinautzin volcanic series
Qcbc	Ash cones	
Qtn	Nuees ardents, peleans, lahars,	
Qtv	Fluvial conglomerates, pumice horizons, soils and tuffs	Quaternary Upper Tarango formation
Tpt	Nuees ardents of ashlar stone type, pumice horizons, soils and tuffs	Tertiary Lower Tarango formation
Tpel	Eluvial deposits	
TpV	Undifferentiated volcanic rocks Tepozotlan range andesite. Guadalupe range dacites. Range deposits.	Tertiary volcanic rocks
Tpa	Ajusco andesite	
Tpcr	Andesite series of the Cruces range.	
Tomx	Undifferentiated volcanic series of the Xonchitepec range.	

a similar stratigraphy. Here there is a solar evaporator for the industrial exploitation of brines. Water remains all year on the surface and it has been determined by careful balances that the yearly infiltration loss is 20 cm. As it is estimated that in the city the fresh water loss in the net leakage is almost 30 per cent, there may be a local infiltration on the order of 2 m^3/s or more.

It is rather difficult to estimate the historical development of local pumpage because, even now, no flow measurements are made in most of the wells.

Taking into consideration the existent fractional information and the reported population at different dates, it has been estimated that the extraction, which began around 1850, is presently on the order of 12 m^3/s. The approximate development is shown in Table 9.8.2 (Figueroa Vega, 1973a and 1977).

In regard to deep piezometric developments, there are similar problems, since their detailed measurement has been made only during the last 10 years. Notwithstanding, from existent data in the Well Register it has been possible to reconstruct partially the evolution as shown in Figures 9.8.4 and 9.8.5. Evolutions prior to 1948 may be estimated only by the fact that, according to old local drillers, many wells within the Lake zone of the city were still flowing wells at the beginning of this century. The water table has remained nearly constant 1 to 2 m below the land surface throughout the period of ground-water development.

9.8.3 LAND SUBSIDENCE

The subsidence of Mexico City is one of the most remarkable cases in all the world.

The phenomenon, which began during the past century, was discovered casually as a result of a polemic about the subsidence of the gates of San Lazaro, at the beginning of the main sewage channel of the city.

In February 1925, Roberto Gayol, author of the project of the sewage net of the city and director of its construction, demonstrated before the Association of Engineers and Architects of Mexico that the problem was just the result of the general subsidence of the bottom of the valley, presenting as evidence two precision levelings, made in 1877 and 1924, of a monument located near the Cathedral (Gayol, 1925). Gayol attributed the phenomenon to the effect of the recently built drainage system.

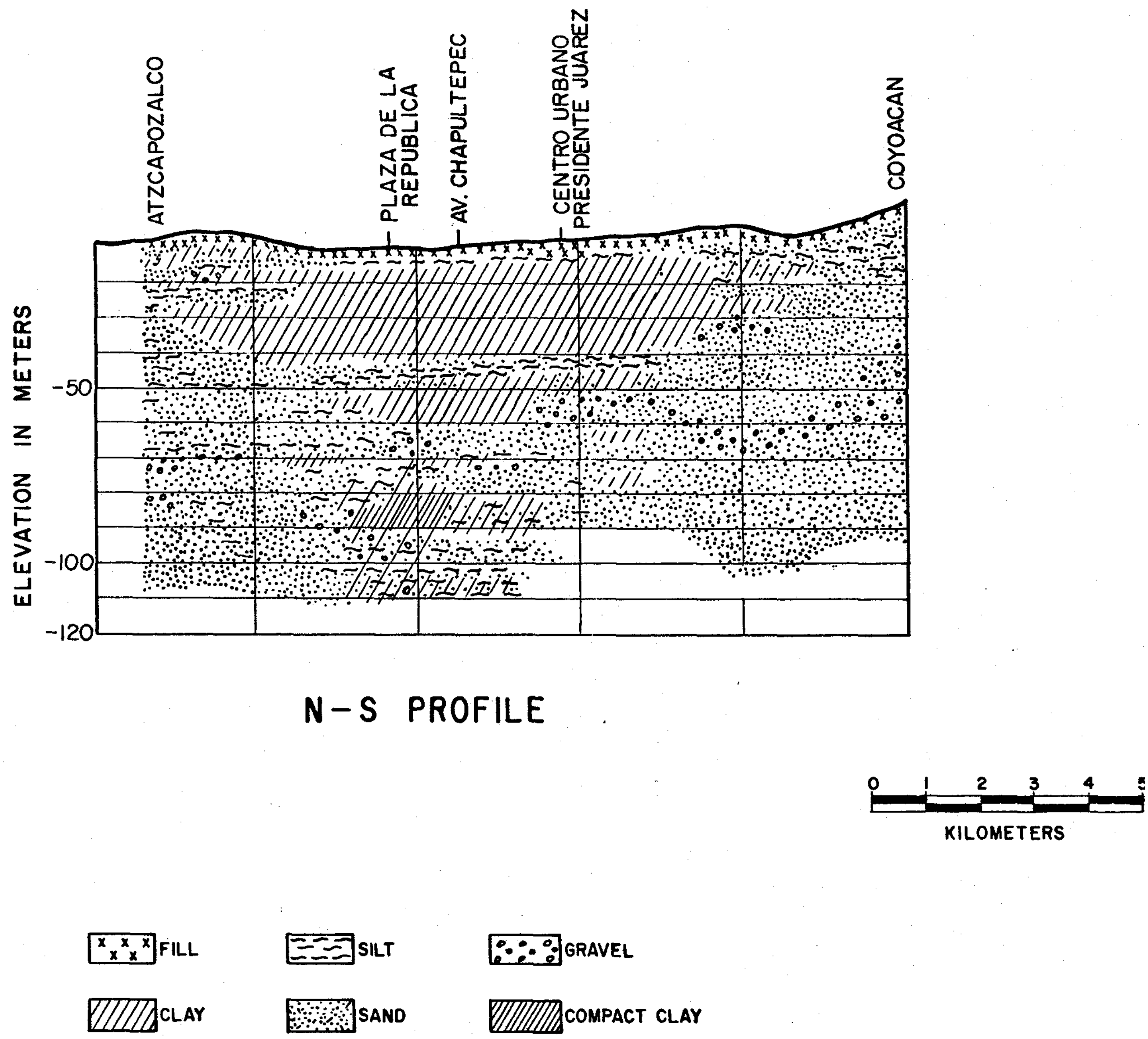


Figure 9.8.2 North-south geological profile (after Marsal and Mazari, 1959).

In spite of the importance of the discovery, 23 years elapsed before Nabor Carrillo demonstrated that the main cause of the subsidence was the extraction of ground-water by wells for municipal use (Carrillo, 1948).

Carrillo, using a profile consisting of an aquifer overlain by clayey strata and assuming a lineal distribution within the clays for the neutral pressures at the beginning and end of the process of consolidation, found the evolution of neutral pressures in the aquifer, corresponding to a constant subsidence velocity of the surface of the clay (Carrillo, 1948).

After that, other investigators continued developing these ideas (Marsal, Hiriart, and Sandoval, 1951). By collecting all the available information regarding precision levelings and mechanical properties of the local clays, they reconstructed the history of the subsidence and made a first prediction about its probable future total magnitude, as shown in Figure 9.8.6 (Marsal, 1952).

At the same time, bench marks and piezometric stations were installed for the observation of subsidence and the evolution of the neutral pressures at different depths. In accordance with the consolidation theory, the neutral pressures are directly related with the phenomenon,

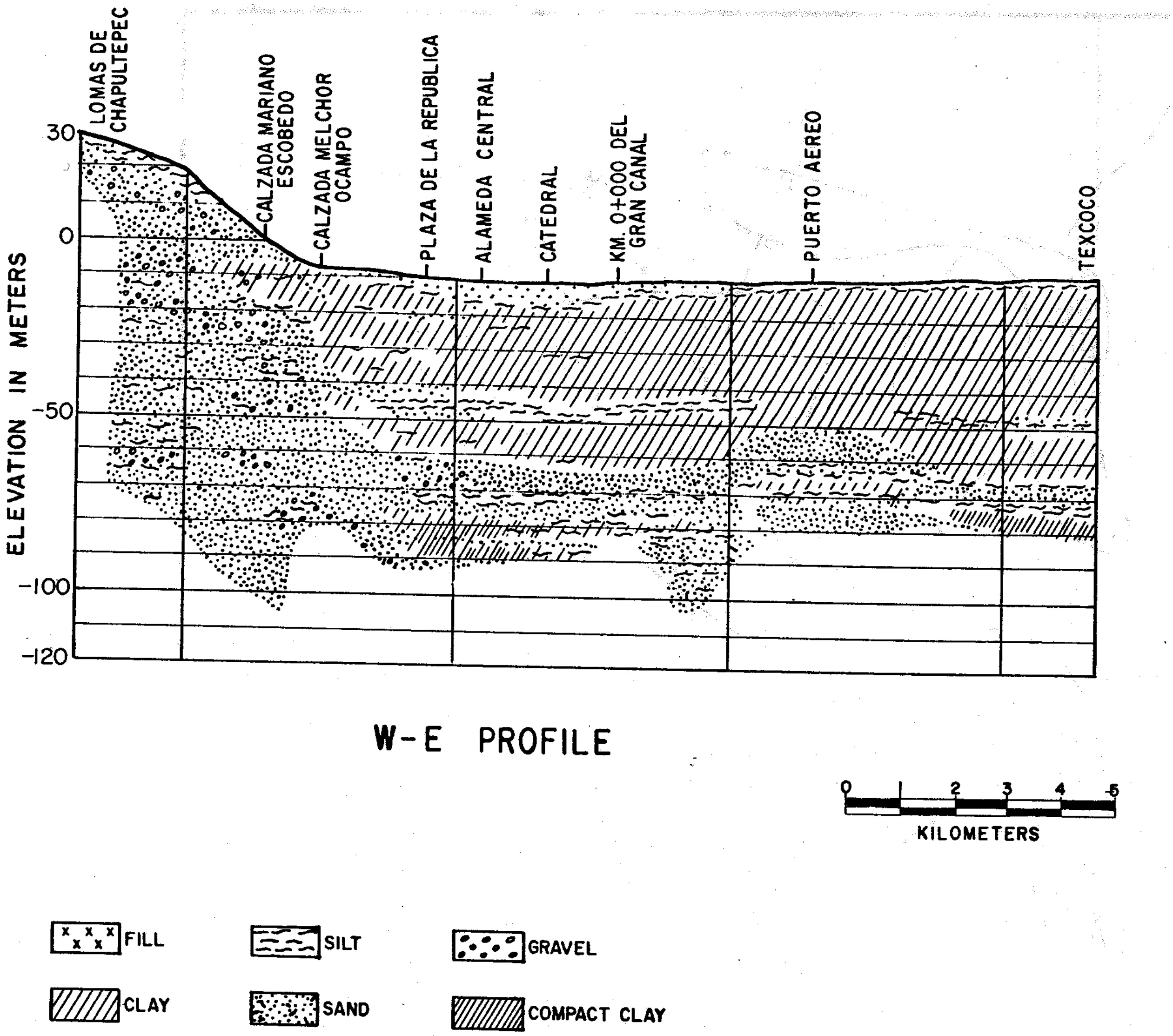


Figure 9.8.3 East-west geological profile (after Marsal and Mazari, 1959).

Table 9.8.2. Orders of magnitude of ground water pumpage in Mexico City.

Year	Pumping Rate (m3/s)
1860	0.0
1910	0.5
1930	1.5
1940	6.0
1950	9.0
1960	9.0
1970	9.0
1974	12.0

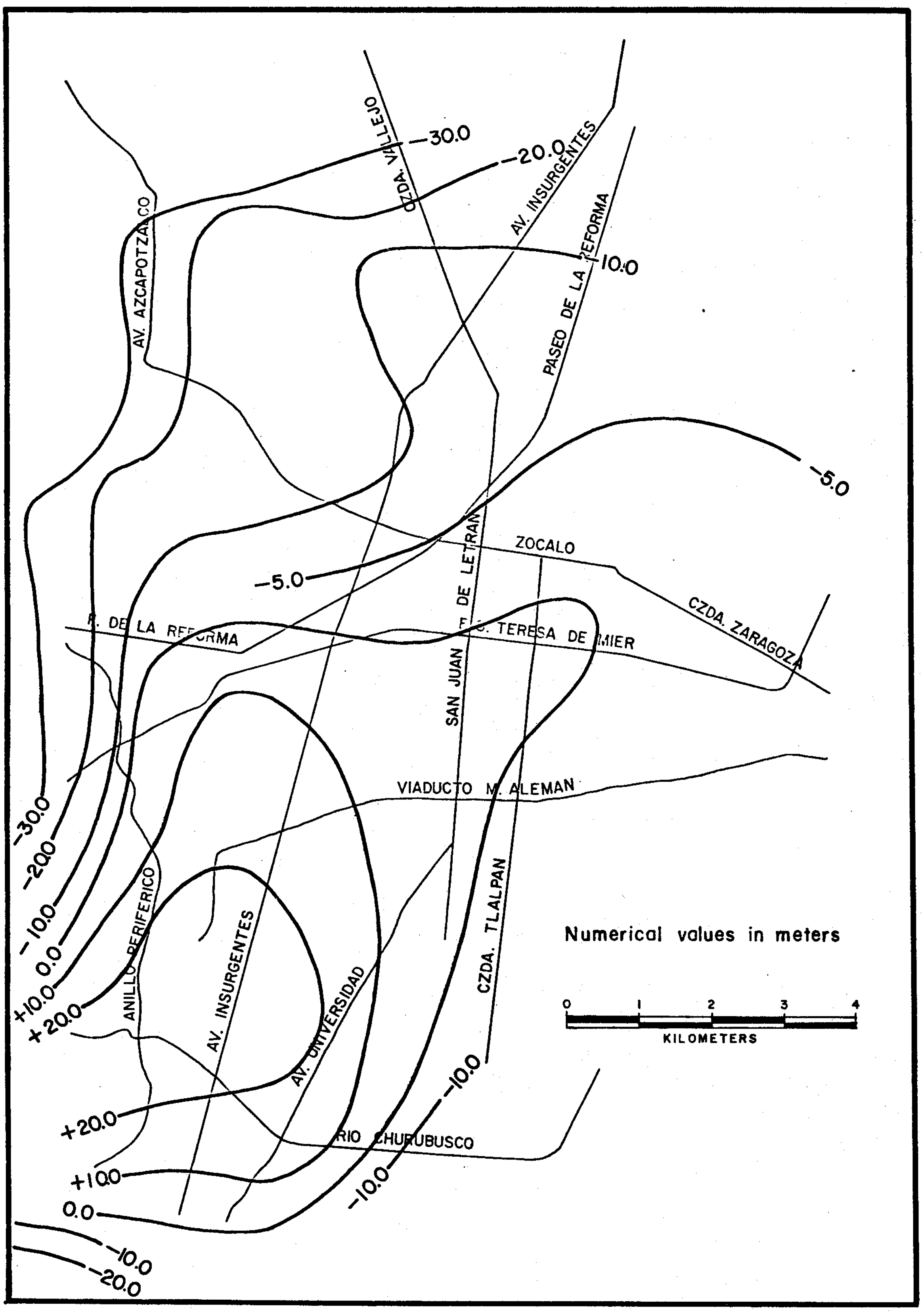


Figure 9.8.4 Change in ground-water level, 1948-1975.

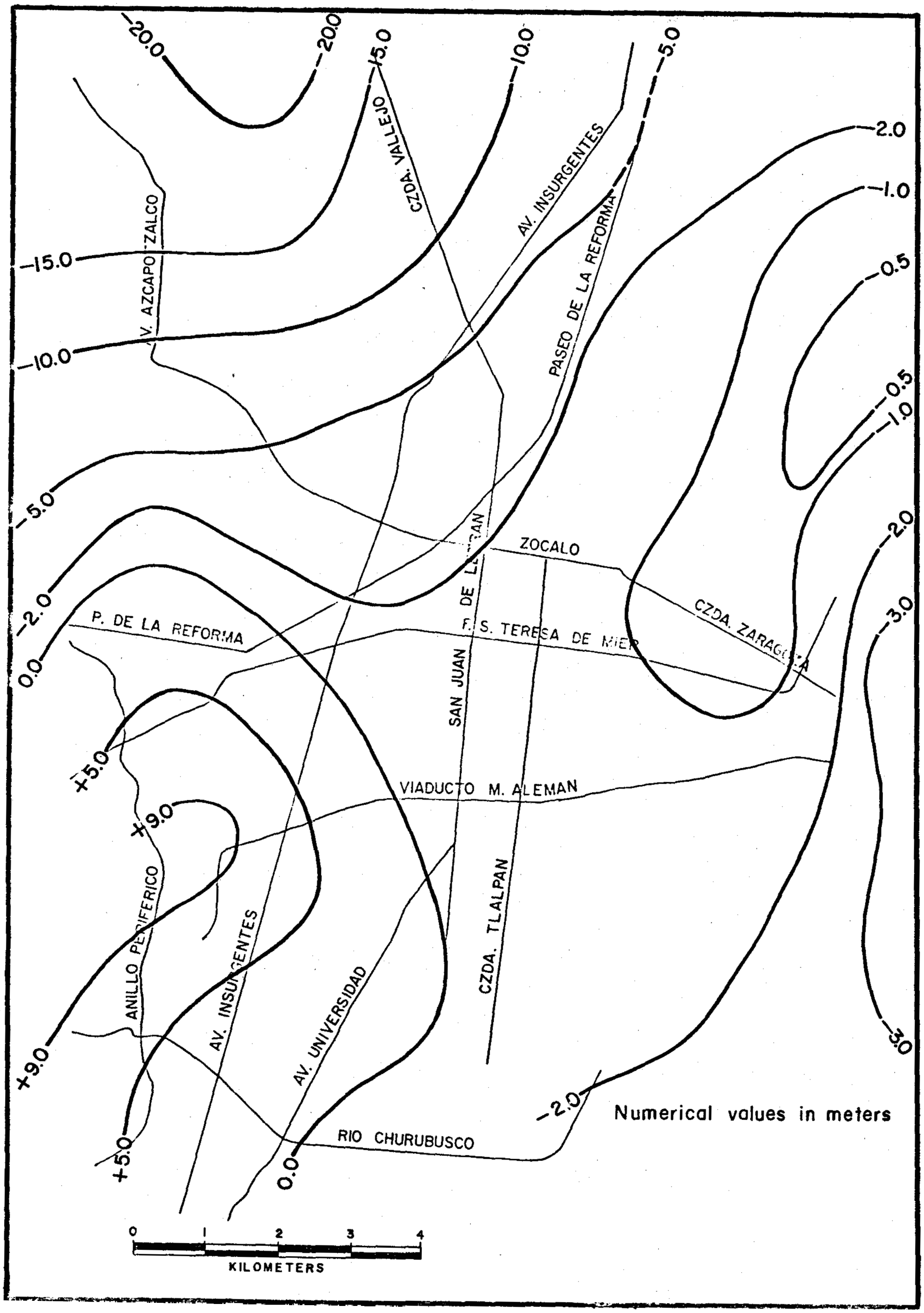


Figure 9.8.5 Change in ground-water level, 1969-1975.

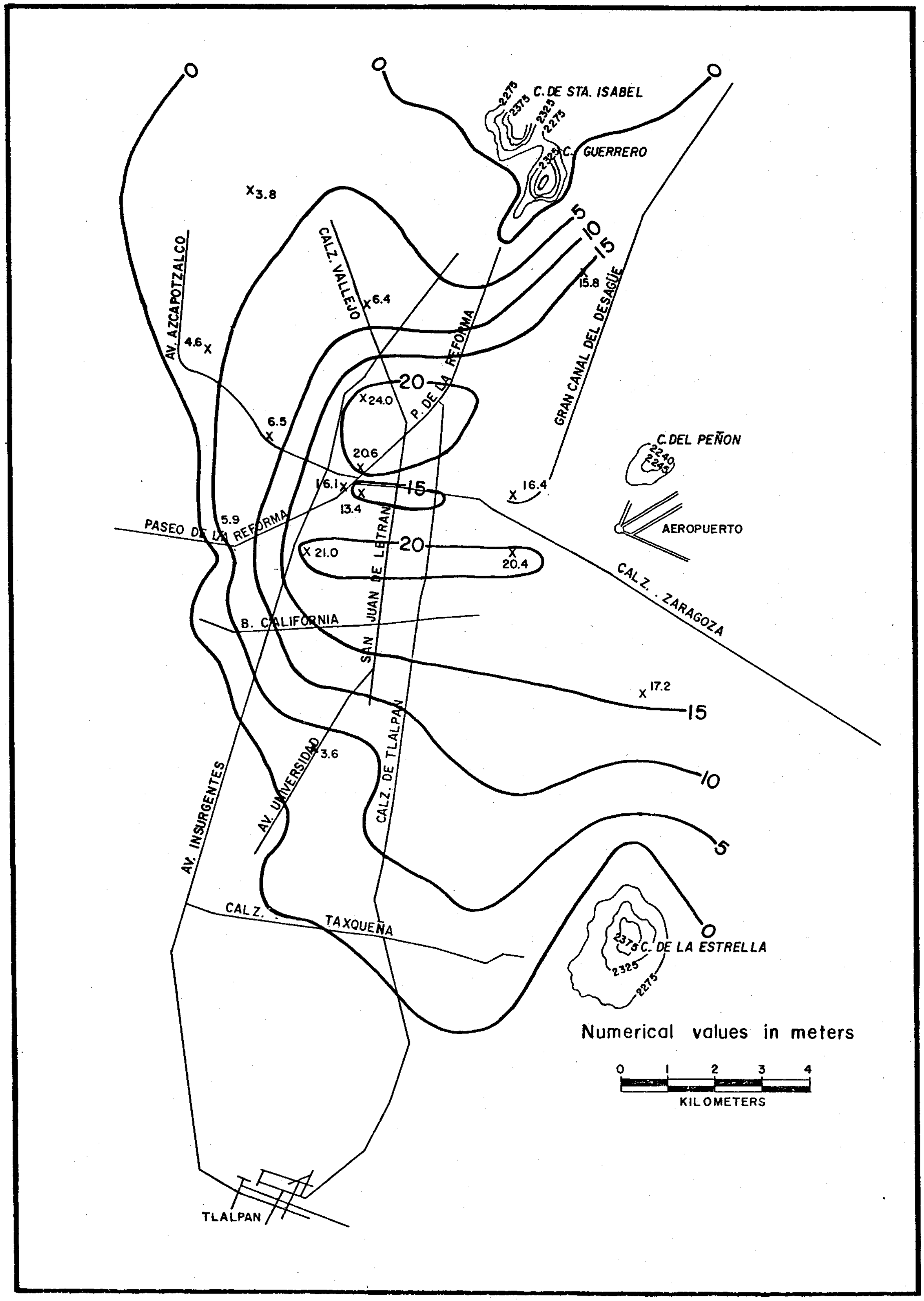


Figure 9.8.6 Potential upper limits of subsidence (after Marsal, 1952).

since their reduction causes a load transfer to the soil structure, with its consequent reduction of volume and resulting subsidence of its surface.

As for the mechanical properties of the clays in the city, a huge quantity of information has been collected, giving rise to a statistical presentation of the existing data and to a stratigraphic zoning of the city, which may be seen in Figure 9.8.7 (Marsal and Mazari, 1959).

Here, in the presence of three main zones, may be noticed: the Hills Zone, located over tuffs of low compressibility, the Transition Zone, and the Lake Zone, located over clays of high compressibility.

In 1954 the Hydrological Commission of the Valley of Mexico, which is now the Water Commission of the Valley of Mexico, took charge of the observation of the subsidence, adopting for this the already established practices. Since then more piezometric stations have been installed, and new precision levellings performed, as well as other observations to be mentioned later in this paper.

The data relative to the above have been published previously (Comisión Hydrológica de la Cuenca del Valle de México, Boletín de Mecánica de Suelos Num. 1, 1953; 2, 1958; 3, 1961a; 4, 1965; 5, 1967; 6, 1970; and 7, 1975).

Accordingly, the subsidence of Mexico City has been known since 1891 for the old part of the city and since 1952 for the total city area.

For the purposes of the present paper, some other figures have been selected (Figures 9.8.8 through 9.8.11) showing, for the old part of the city, the subsidence during the periods 1891-1952, 1952-1973, and 1891-1973, and for all the city during the period 1952-1973. In the same way, Figure 9.8.12 shows the observed subsidence through time of several selected points.

On the other hand, Tables 9.8.3 and 9.8.4 show the mean velocity of subsidence in the old part of the city and in the total area for different periods. The general evolution of subsidence in Mexico City can be visualized through the maps, graphs, and tables included herein. It may be seen that at some places it has almost reached 9 metres. Figure 9.8.12 shows the general trend of subsidence, which has evolved as an inverted "S" of asymptotic nature, with a remarkable diminution in recent years.

In addition to the subsidence, superficial cracks have been observed in two zones: along Paseo de la Reforma and a parallel street, within the clayey zone, and in the northwestern part of the City, in the tuffaceous zone. Those of the first zone have brought about the demolition of several houses and a part of a school and also caused serious damage to the abutments of a recently built bridge. The latter are even more impressive.

The subsidence of the city has also been noticed through the protrusion of well casings. Table 9.8.5 shows a comparison between observed protrusions and measured subsidences in several wells.

It has been shown by correlation studies that for the period 1970 - 1973, approximately 75 per cent of the total subsidence was due to consolidation of the clayey strata and the remaining 25 per cent to the compression of the materials of the deep strata that constitute the aquifer.

There is no doubt about the main cause of Mexico City's subsidence: the overdraft of the aquifer. As a rough estimate the weight of the buildings contribute only 10 to 15 per cent of the total subsidence.

Since 1972 a digital model has been developed to simulate the subsidence of Mexico City. The central idea is to reduce the system of partial differential equations which represent the behavior of the coupled system aquifer-consolidating strata to an integrodifferential equation for the aquifer alone, including the inputs by consolidation through a convolution or memory term (Figueroa Vega, 1973b and 1977).

Some preliminary results show that the simulation is possible, within the limitations imposed by the employed simplifications.

The model is presently in its calibration stage, which has been impaired because data pertaining to the aquifer are relatively scarce (Figueroa Vega, 1977).

9.8.4 ECONOMIC AND SOCIAL IMPACT OF SUBSIDENCE

It is difficult to estimate the economic and social impact of the subsidence of Mexico City. Among the main resulting damages are those to buildings, sidewalks, and pavements, not to mention the continuous dislocation of the freshwater and sewage nets.

On the other hand, the sewage of the city, which originally drained by gravity, has been eliminated by pumping since the flood which occurred during 1951.

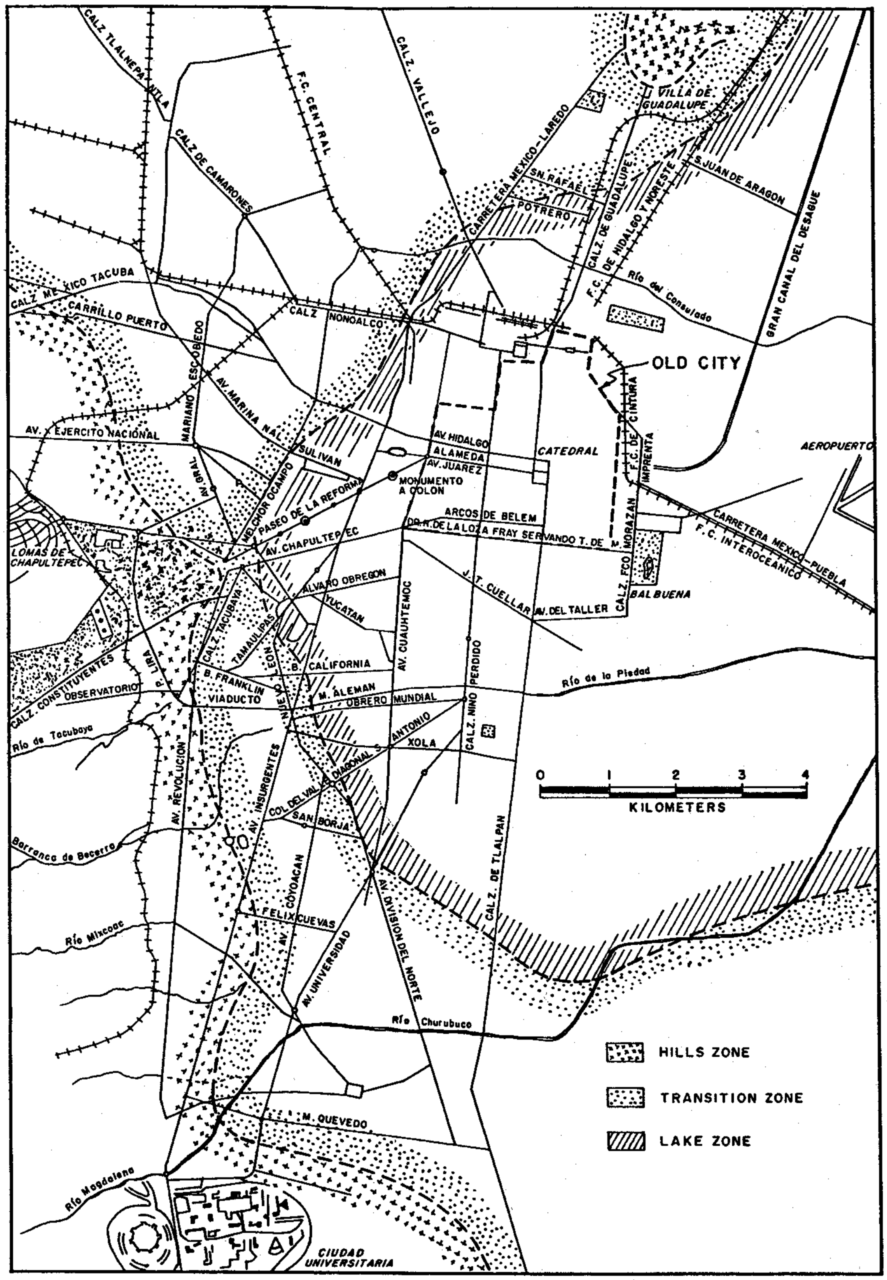


Figure 9.8.7 Zonification of the city (after Marsal and Mazari, 1959).

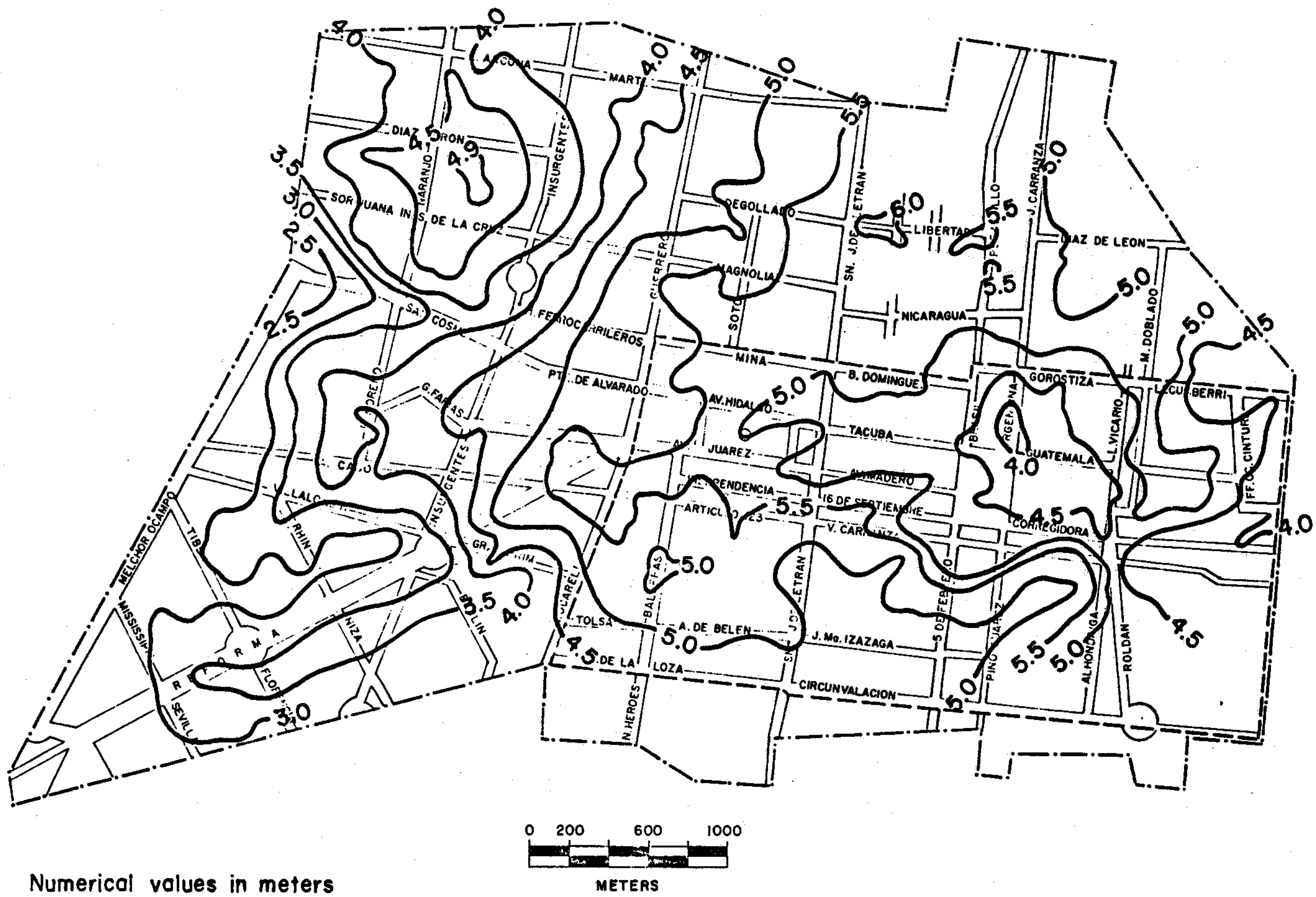


Figure 9.8.8 Subsidence (Old City), 1891-1952 (after Comisión Hidrológica de la Cuenca del Valle de México, 1953).

The constant danger of new floods in case of an electric system, failure, compelled the city authorities to build a Deep Sewage System, with a capacity of 200 m^3/s and a length on the order of 60 km. Complementary collectors are presently under construction.

The total cost of the project would have been much less, if it had been possible to eliminate the sewage by gravity. On the other hand, the overexploitation and the consequent ground-water declines have raised the cost of ground-water extraction, and the loss of water due to dislocation of the distribution net has been estimated up to 15 m^3/s.

Because of above-mentioned factors, the subsidence of Mexico City could conceivably be more expensive than bringing water from other watersheds to avoid the overexploitation of the local aquifer.

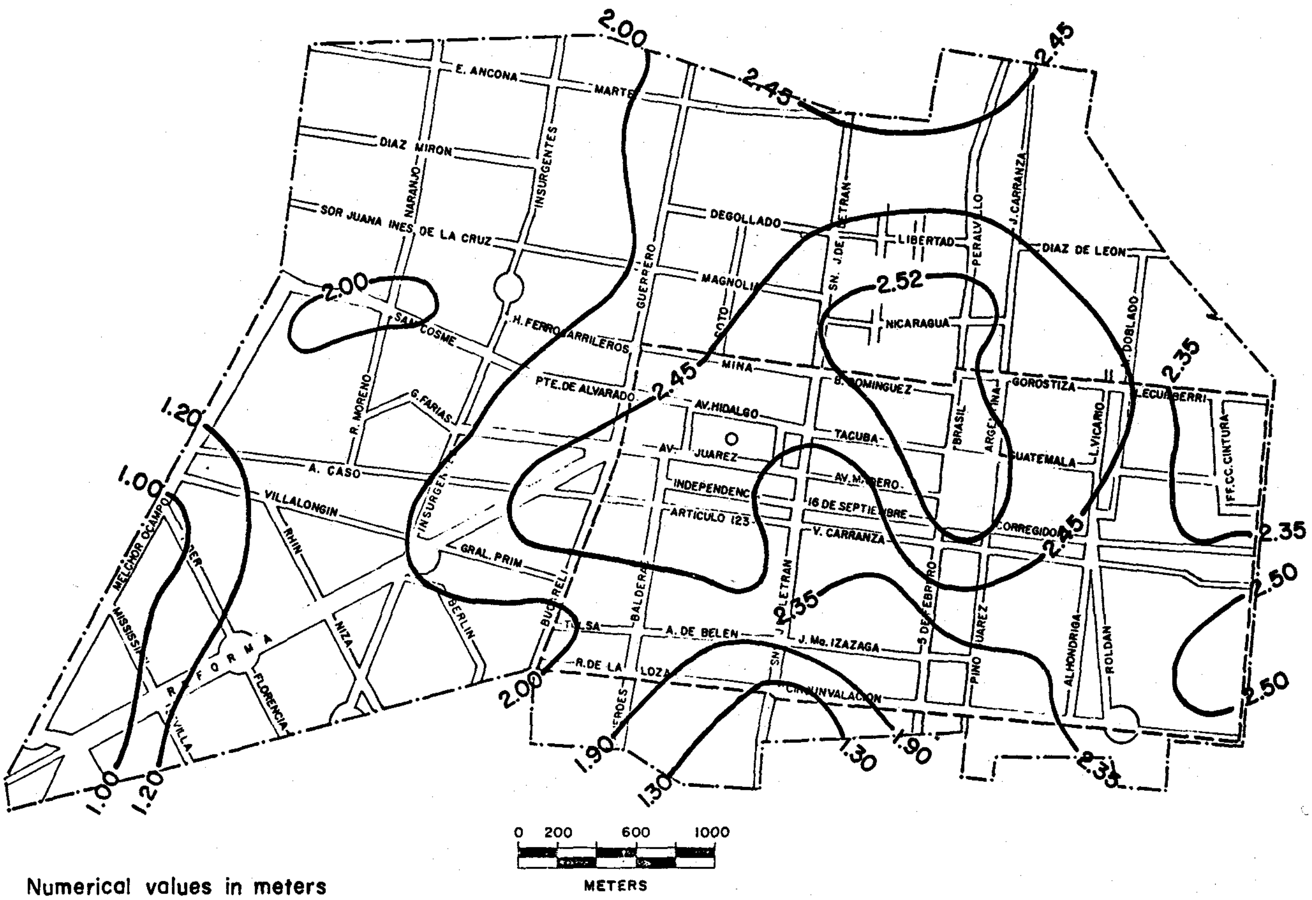


Figure 9.8.9 Subsidence (Old City), 1952-1973 (after Comisión de Aguas del Valle de México, 1975).

9.8.5 LEGAL ASPECTS

From a legal standpoint, the ground water in Mexico belongs to the nation and for this reason no legal action is taken against its overexploitation. As a result, social costs originating from overdraft are normally covered through taxation and water rates.

9.8.6 MEASURES TAKEN TO CONTROL OR AMELIORATE SUBSIDENCE

Soon after the floods of 1951, the City authorities began bringing water from other sources outside the Basin and managed to keep the local extraction constant for many years, as shown in Table 9.8.2. The effect of this may be appreciated in the final portion of the curves of Figure 9.8.12.

The accelerated growth of the city in the last years, which has been an average on the order of 5 per cent annually, has made it necessary to increase slightly the local extractions, as

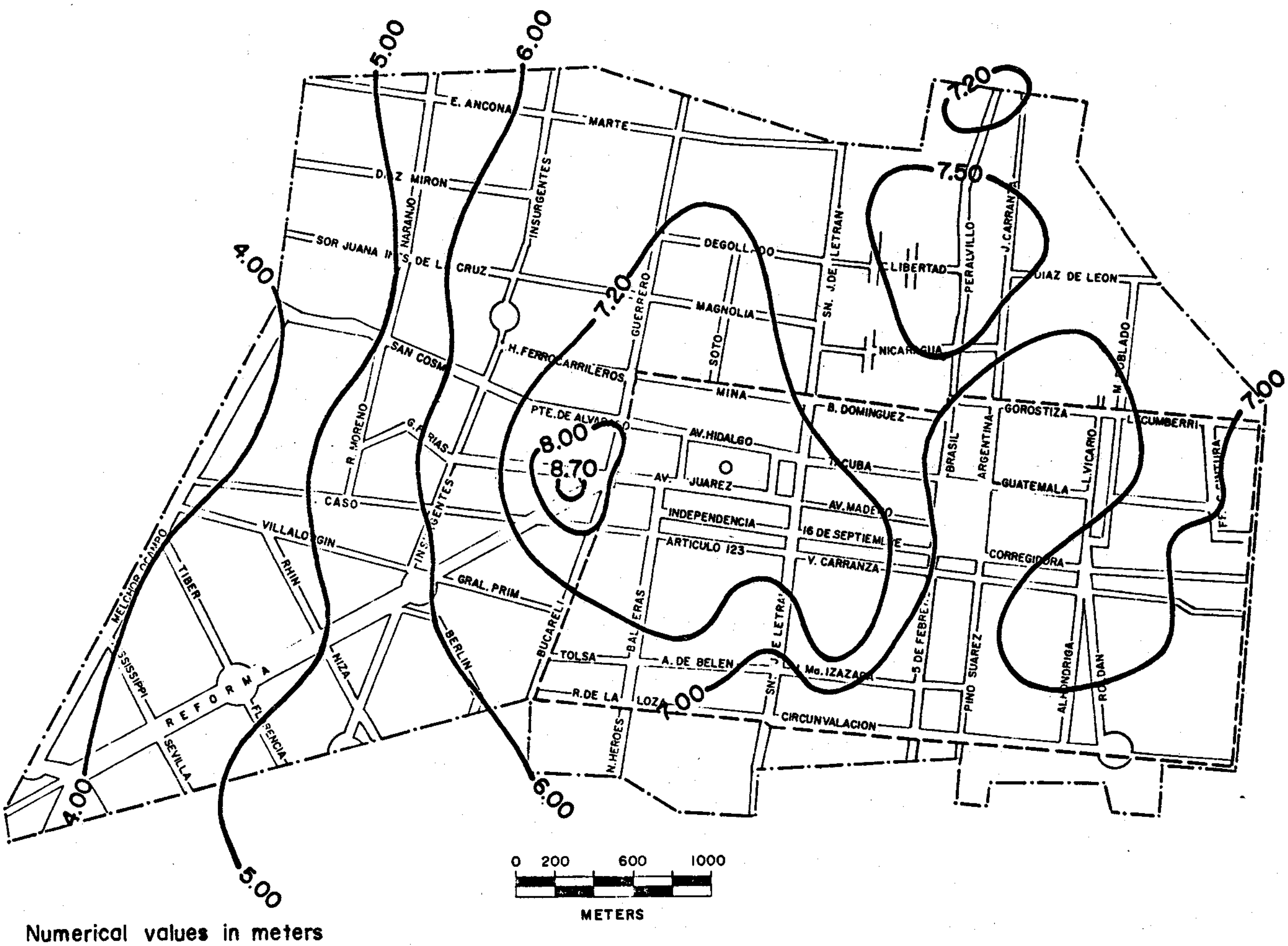


Figure 9.8.10 Subsidence (Old City), 1891-1973 (after Comisión de Aguas del Valle de México, 1975).

shown in the same table. Nevertheless, large projects to bring water from other watersheds are now under way in order to satisfy the future demands and, if possible, to be able to diminish the local extraction in order to solve the subsidence problem. Additionally, the construction of sewage treatment plants for industrial use is now under way and recirculation of water in industries is being made mandatory in order to shut down some of the wells employed by the industries, as these consume almost 30 per cent of the water used by the city. It is estimated that in the near future the substitution of treated sewage water for ground water for industrial use could be of the order of 5 to 7 m^3/s.

The effect of these measures will undoubtedly reduce or cancel the subsidence of the City of Mexico. The schedule for this depends now on political decisions and on availability of funds.

9.8.7 REFERENCES

CARILLO, N. 1948. Influence of artesian wells on the sinking of Mexico City: Proc. of the Int. Conf. on Soil Mechanics, Holland.

COMISION HIDROLOGICA DE LA CUENCA DEL VALLE DE MEXICO. 1953. Boletín de Mecánica de Suelos Num. 1, México, D.F.

___________. 1958. Boletín de Mecinica de Suelos Num. 2, México, D.P.

___________. 1961a. Boletín de Mecánica de Suelos Num. 3, México, D.F.

___________. 1961b. Informe sobre la geologia de la Cuenca del Valle de México y zonas colindantes.

___________. 1963. Hidrologia de la Cuenca del Valle de México, Tomos II y V.

___________. 965. Boletín de Mecánica de Suelos Num. 4, México, D.F.

___________. 1967. Boletín de Mecánica de Suelos Num. 5, México, D.F.

___________. 1970. Boletín de Mecánica de Suelos Num. 6, México, D.F.

COMISION DE AGUAS DEL VALLE DE MEXICO. 1975. Boletín de Mecánica de Suelos Num. 7, Mexico, D.F.

FIGUEROA VEGA, G. E. 1973a. El Hundimiento de la Ciudad de México. Breve Descripción; Recursos Hidráulicos. Vol. II, num. 4; pp. 525-534.

___________. 1973b. Aquifer and subsidence model for Mexico City. 85th annual meeting of The Geological Society of America; v. 5, no. 7, p. 620.

___________. 1977. Subsidence of the City of Mexico, A Historical Review; Publication No. 121 of The International Association of Hydrological Sciences, pp. 35-38.

GAYOL, R. 1925. Estudio de las pertubaciones que en el fondo, de la Ciudad de México ha producido el drenaje de las aquas del subsuelo, por las obras del desaque y rectificación de los errores a que ha dado lugar una incorrecta interpretación de los efectos producidos. Revista Mexicana de Ingeniería y Arquitectura, Vol. III, Num. 2. pp. 96-132.

MARSAL, R. J. 1952. Estudios relativos al comportamiento del subsuelo de la Ciudad de México. Instituto Nacional de Investigacion Cientifical México, D-F,

MARSAL, R. J., HIRIART, F., and SANDOVAL, R. 1951. Hundimiento de la Ciudad de México. Congreso Cientifico Mexicano. México, D.P.

MARSAL, R. J., and MAZARI, M. .1959. El Subsuelo de la Ciudad de México. Primer Congreso Panamericano de Mecánica de Suelos y Cimentaciones, México, D.F. Republished in 1969 with English translation.

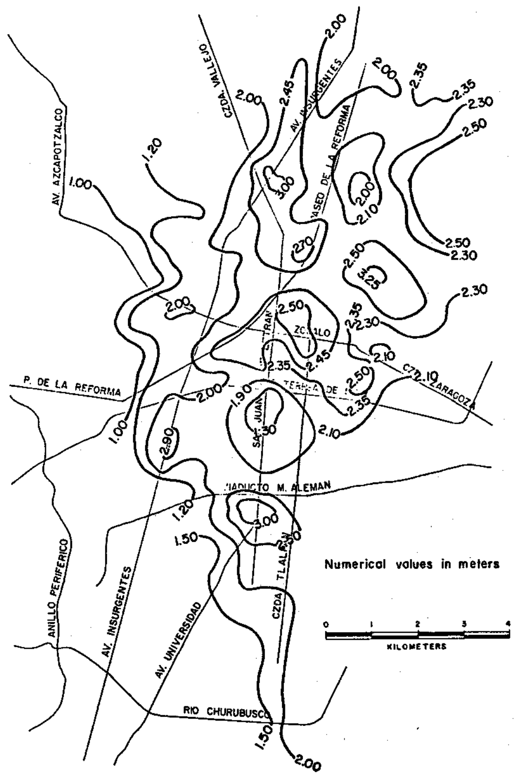


Figure 9.8.11 Subsidence (total area), 1952-1973 (after Comisión de Aguas del Valle de México).

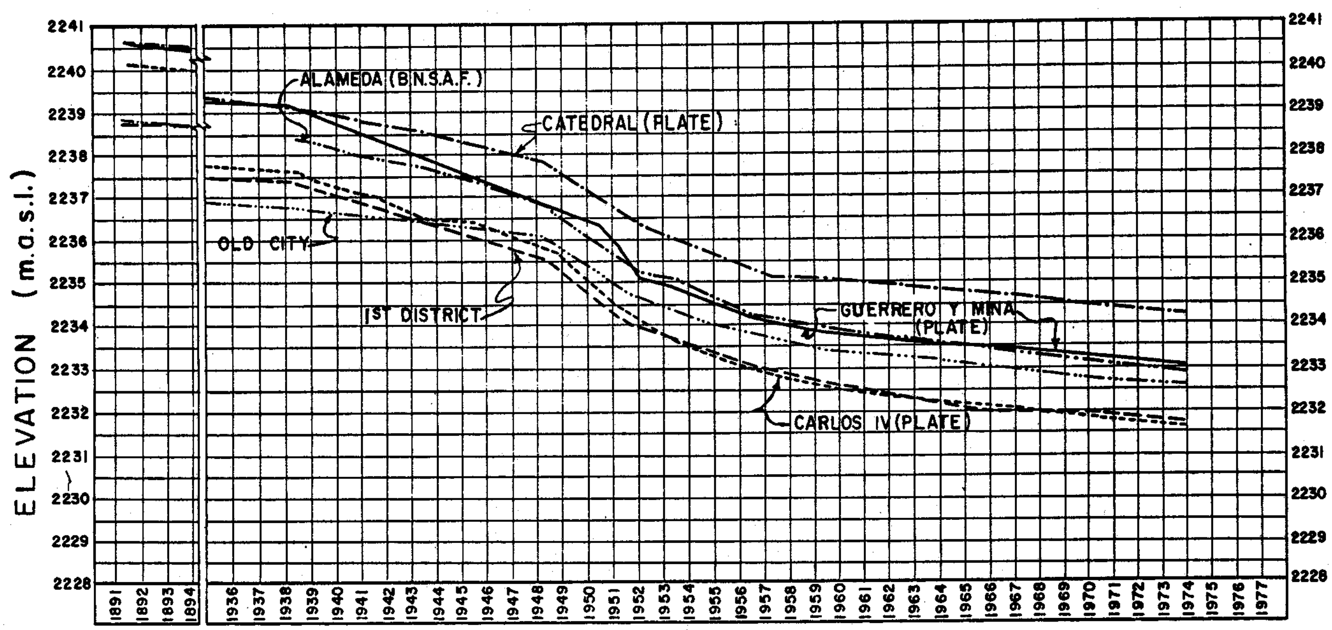


Figure 9.8.12 Subsidence evolution at selected sites (after Comisión de Aguas del Valle de México).

Table 9.8.3 Mexico City subsidence (older part) (from Figueroa Vega, 1977, Table 2).

From - to	Total subsidence (m)	Average (m/year)
1891 - 1938	2.12	0.045
1938 - 1948	0.76	0.076
1948 - 1950	0.88	0.440
1950 - 1951	0.46	0.460
1951 - 1952	0.15	0.150
1952 - 1953	0.26	0.260
1953 - 1957	0.68	0.170
1957 - 1959	0.24	0.120
1959 - 1963	0.22	0.055
1963 - 1966	0.21	0.070
1966 - 1970	0.28	0.070
1970 - 1973	0.17	0.051

Table 9.8.4 Mexico City subsidence (total area) (from Figueroa Vega, 1977, Table 3).

From - to	Total Subsidence (m)	Average (m/year)
1952 - 1959	1.014	0.140
1959 - 1963	0.440	0.110
1963 - 1966	0.254	0.080
1966 - 1970	0.260	0.065
1970 - 1973	0.203	0.059

Table 9.8.5 Well casings protrusion (from Figueroa Vega, 1977, Table 4).

Well	Protrusion 1970 - 1973 (m)	Subsidence 1970 - 1973 (m)
San Juan de Aragón Campamento	0.304	0.440
Czda. Guadalupe	0.130	0.172
Sta. Isabel Tola	0.259	0.320
Monumento de la Revolución Frontón México	0.179	0.200
Jardin de los Angeles No. 2	0.076	0.145
Insurgentes Norte 1407	0.199	0.283
Penitenciaria Jardin	0.233	-
Gómez Farías No. 61	0.113	0.140
Monumento de la Revolución Procuraduría	0.323	-
Jardin de los Angeles No. 3	0.100	0.145
Jardin de los Angeles No. 1	0.146	0.150

Case History No. 9.9. The Wairakei Geothermal Field, New Zealand, by Paul F. Bixley, Ministry of Works and Development, Wairakei, New Zealand

9.9.1 INTRODUCTION

The Wairakei geothermal area is located 8 km north of Lake Taupo in the center of the North Island of New Zealand (Figure 9.9.1). Geothermal investigations at Wairakei began in 1950 and culminated with the commissioning of the first stage of the power station in 1958 and the second stage in 1964 bringing the installed capacity to 192 MWe. Steam is supplied to the power station by 64 wells, most of which produce a steam-water mixture at the wellhead. The mixture is separated, the steam piped to the power station and waste water dumped into the Waikato River. The 64 production wells cover an area of 2 km^2 referred to in this paper as the "production field."

The first indication of ground movement came in 1956 when discrepancies were found in the levels of several benchmarks since the previous survey in 1950 (Hatton, 1970). One benchmark had subsided 76 mm. A levelling network was established and gradually expanded until by 1971, the most recent comprehensive survey, the area of subsiding ground was found to exceed 30 km^2. Within this area were two zones of relatively rapid subsidence; one immediately north of the eastern production field and the other at Karapiti, a region of natural thermal activity about 3 km south of the production field. Economic interest has centered on the zone of rapid subsidence northeast of the production field, as steam mains to the power house, and channels carrying separated water pass across this zone. Benchmarks in this zone are levelled to third order standards annually (third order accuracy is within 12 mm$\sqrt{\text{km}}$, where km is kilometres of line traversed).

9.9.2 GEOLOGY

The geology of the Wairakei geothermal field has been discussed in detail by Grindley (1965). The production field is underlain by a near flat sequence of acid volcanics, consisting of six basic units down to 1.2 km (most wells are drilled to 600-1200 m, one well is drilled to 2.5 km). These units are: Recent Pumice, Wairakei Breccia, Huka Falls Formation, Haparangi Rhyolite, Waiora Formation and Wairakei Ignimbrites. Almost all production comes from within the Waiora Formation where active faults have been intercepted by drillholes (Grindley, 1965). There is also evidence for a permeable zone in the Waiora Breccia just above the Wairakei Ignimbrite contact (Bolton, 1970).

WAIRAKEI IGNIMBRITES: Hard welded ash flow tuff, thickness 950 m in the single hole penetrating this formation.
WAIORA FORMATION: Pumice sandstone, pumice breccia and thin (up to 70 m) ignimbrite sheets. Total thickness 400 m in the western section of the production field, thickening rapidly to the east to greater than 750 m.
HAPARANGI RHYOLITE: An extensive rhyolite sill, intruded into the Waiora Formation to the west of the production area. Maximum thickness 450 m.
HUKA FALLS FORMATION: Bedded mudstone and tuffaceous sandstone; thickness 60-220 m.
WAIRAKEI BRECCIA: Chalazoidite and vitric tuff conformably overlying the Huka Falls Formation; differentiated from the Huka Falls Formation by the incoming of chalazoidites. Maximum thickness 170 m.
RECENT PUMICE: Superficial deposits up to 30 m thick, consisting of alluvium derived from the dissection of underlying formations together with ash and pumice/lapilli shower material.

The relationship between these units is shown on the cross section ABC (Figure 9.9.2).

The age of the above sequence ranges from lower-mid Pleistocene for the Wairakei Ignimbrites to possibly as young as 20,000 years BP for the Wairakei Breccia (Browne,1973).

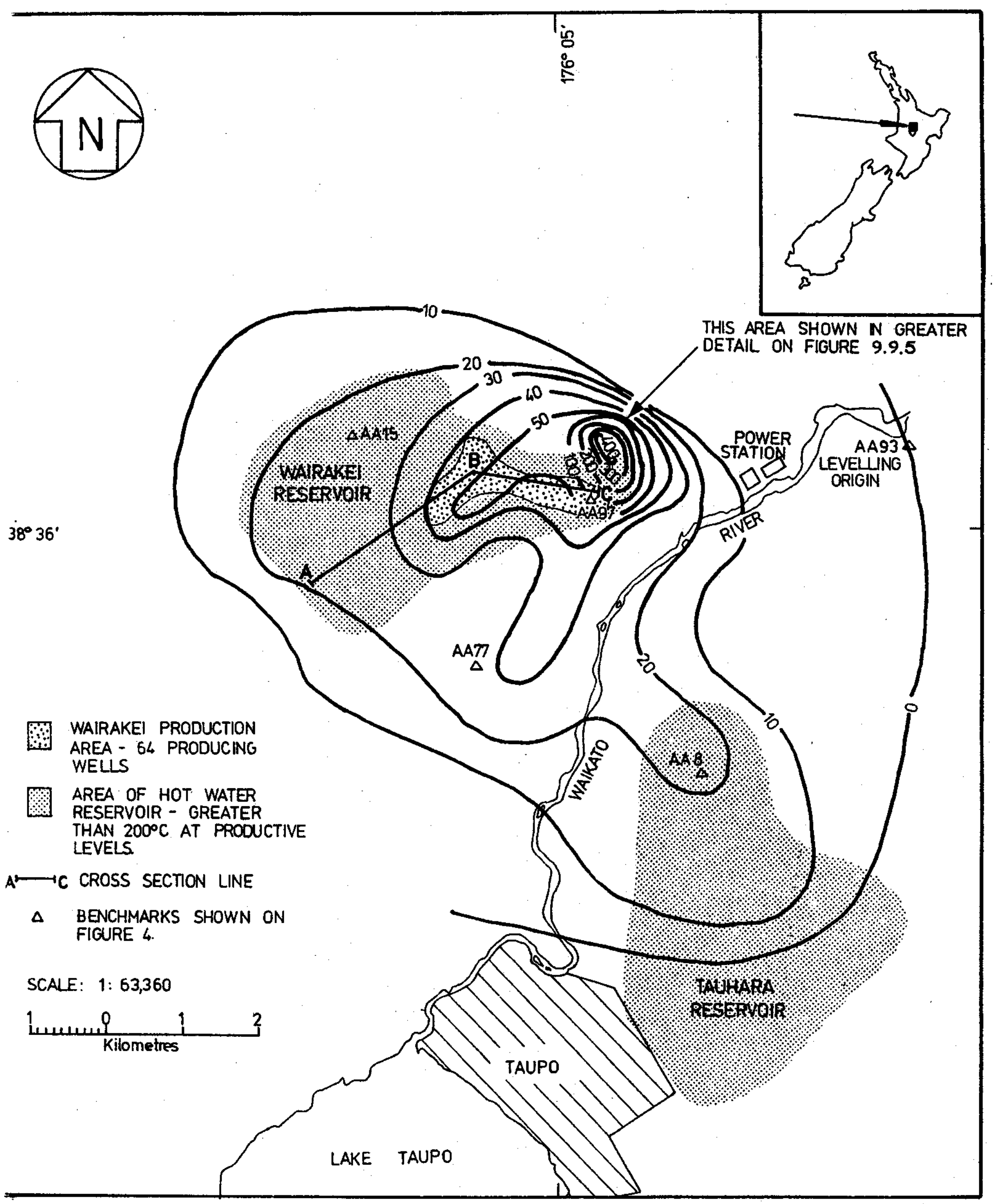


Figure 9.9.1 Land subsidence 1956 to 1971, of Wairakei-Tauhara geothermal areas, New Zealand. Lines of equal subsidence in millimetres per year. Reservoir boundaries are taken as the estimated 2000 temperature contour at the production level, based on surface resistivity measurements and downhole temperature profiles.

The Haparangi Rhyolite at Wairakei is considered tentatively to be an intrusive rhyolite of late Huka age (Grindley 1965).

9.9.3 STRUCTURE

The Wairakei production field is located on a structural high situated between the major Taupo-Reporoa Basin to the east and a series of smaller block and basin structures to the west. Structure is largely controlled by a series of normal faults which strike northeast, parallel to the trend of the Taupo Volcanic Zone. Most of these faults are still active.

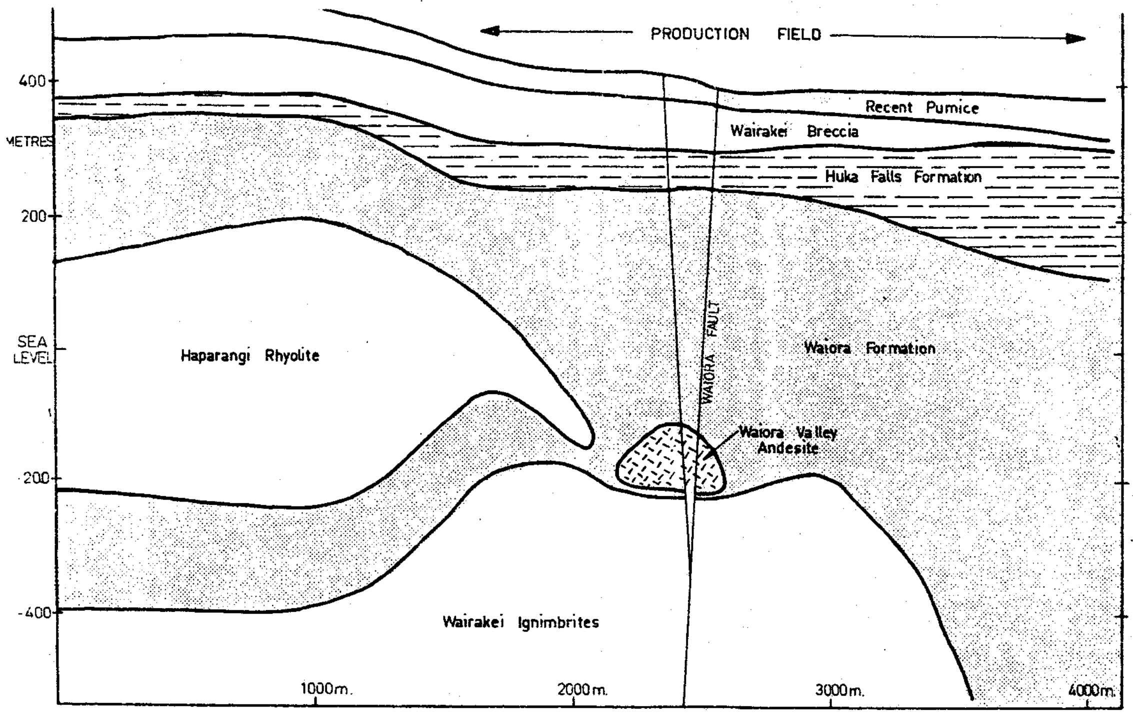


Figure 9.9.2 Cross section ABC through Wairakei geothermal reservoir showing geological structure. Vertical scale twice the horizontal scale.

9.9.4 ROCK PROPERTIES

Cores taken from investigation wells drilled into the hot water reservoir at Wairakei have been extensively tested for wet and dry bulk densities, particle density and porosity.

In 1975 Terra Tek Inc. conducted a series of comprehensive tests on selected cores from Wairakei for Systems Science and Software Inc. (Pritchett, 1977). Cores used for these tests had remained drying in the core shed for ten years before testing. Bulk density and particle density measurements agreed with those previously done on fresh cores. Results of these tests are tabulated below:

Table 9.9.1. Properties for Wairakei Hot Water Reservoir Rocks. Densities are tonnes/m^3 and effective porosity per cent by volume.

Formation	Bulk density wet	dry	Effective porosity	Grain density
Surface Pumice	1.88	1.39	49	2.71
Huka Falls Formation	1.99	1.59	40	2.70
Waiora Formation	2.02	1.64	39	2.72
Wairakei Ignimbrite	2.36	2.22	14	2.69

The linear coefficient of expansion for the Waiora Formation was found to be 8.2×10^{-6} m/m/°C and the dry specific heat 0.71 to 0.75 J/g°C. Thermal conductivity increased with increasing stratigraphic depth of the formation: measured values (saturated) were: Surface Pumice 1.03, Huka Falls Formation 1.28, Waiora Formation 1.56, and Wairakei Ignimbrite 2.11 W/m°c.

9.9.5 HYDROLOGY

The production field covers an area of 2 km^2 but this is only part of a much larger reservoir, as shown on Figure 9.9.1. Down to depths explored by drilling (1.2 km, and one well to 2.5 km) the Waiora Formation, a heterogeneous mixture of acid volcanic pyroclastics, vitric tuffs, sediments, and a rhyolite sill, forms the aquifer system from which geothermal fluids are withdrawn. A hot water reservoir (over 200°C) covering an area of 11 km^2 has been delineated by drilling and geophysical exploration.

At Wairakei 100 wells have been completed in the production area, 16 deep exploration wells in the hot water reservoir outside the production area and four deep and two shallower wells completed in the "cold" area outside the hot water reservoir.

Exploratory wells drilled outside the hot water reservoir have shown that there are hydrological connections between these "cold" wells and the hot water reservoir. Well 223, a "cold" well 5 km west of the production area, reacts almost immediately to changes in drawoff rate in the production field (Bolton, 1970). Bolton used the steep pressure gradients between cold wells and the hot water reservoir as evidence for some kind of low permeability barrier between the hot water reservoir and the surrounding cold water, down to depths of at least 1 km.

The Wairakei hot water reservoir is connected hydrologically to another hot water reservoir of about the same size located 8 km to the SE at Tauhara (Figure 9.9.1). This reservoir has not been exploited.

The intensive pattern of active, northeast-striking faults through the Wairakei reservoir is the major control on fluid flow. These faults penetrate the Huka Falls Formation "caprock" allowing the escape of hot fluids to feed natural thermal features, and when pressure conditions are suitable they may allow cold water from the surface water table to penetrate the hot water system. The faults also provide channels for vertical inflow of hot water into the aquifer system from below. Within the hot water filled aquifer system faults allow rapid propagation of pressure changes.

Thus the hot water aquifer system at Wairakei is unconfined, in that the "caprock" is penetrated by active faults and the same faults provide channels for inflow of hot water into the system from below. In addition, although there seem to be impermeable barriers around the hot water systems down to depths of at least 1 km, the cold wells located outside the hot water reservoir are affected by pressure change within the reservoir.

9.9.6 HISTORIC DEVELOPMENT OF MASS WITHDRAWAL

The withdrawal history from the Wairakei reservoir is shown in Figure 9.9.3. Output from the "western" wells includes all investigation wells located outside the production field and well 204 which blew out in 1960 and continued to discharge uncontrolled until 1973 when discharge ceased. Heat withdrawal from the production field is currently 1570 MW, and natural heat discharge is of the order of 400 MW (both relative to 12°C) - Current mass withdrawal rate is about 5500 t/h.

9.9.7 CHANGE OF PRESSURE IN THE AQUIFER SYSTEM

Average pressures at 152 m below sea level in the production field are plotted on Figure 9.9.3. Pressure changes due to withdrawal of mass and heat are discussed in detail by Bolton (1970) and Pritchett (1977). Bolton pointed out that the behavior of the Wairakei reservoir is primarily governed by the saturation pressure-temperature relation for water. Hydrostatic pressures throughout the reservoir have followed the trends shown on Figure 9.9.3. However, in the upper parts of the aquifer system below the production field a zone filled with saturated steam has developed. Pressure drop in this zone depends on changes in steam temperature.

9.9.8 LAND SUBSIDENCE

Surveys show an area of over 30 km^2 is subsiding at more than 10 mm/year (Figure 9.9.1). Within this area are two smaller zones each of about 1 km^2 which have subsided comparatively rapidly. The zone at Karapiti, 3 km south of the production field, was the most rapidly subsiding part of the survey network until about 1963, when the subsidence rate decreased to the same rate as for the surrounding ground surface. Subsidence at bench mark AA77 within this zone is plotted on Figure 9.9.4.

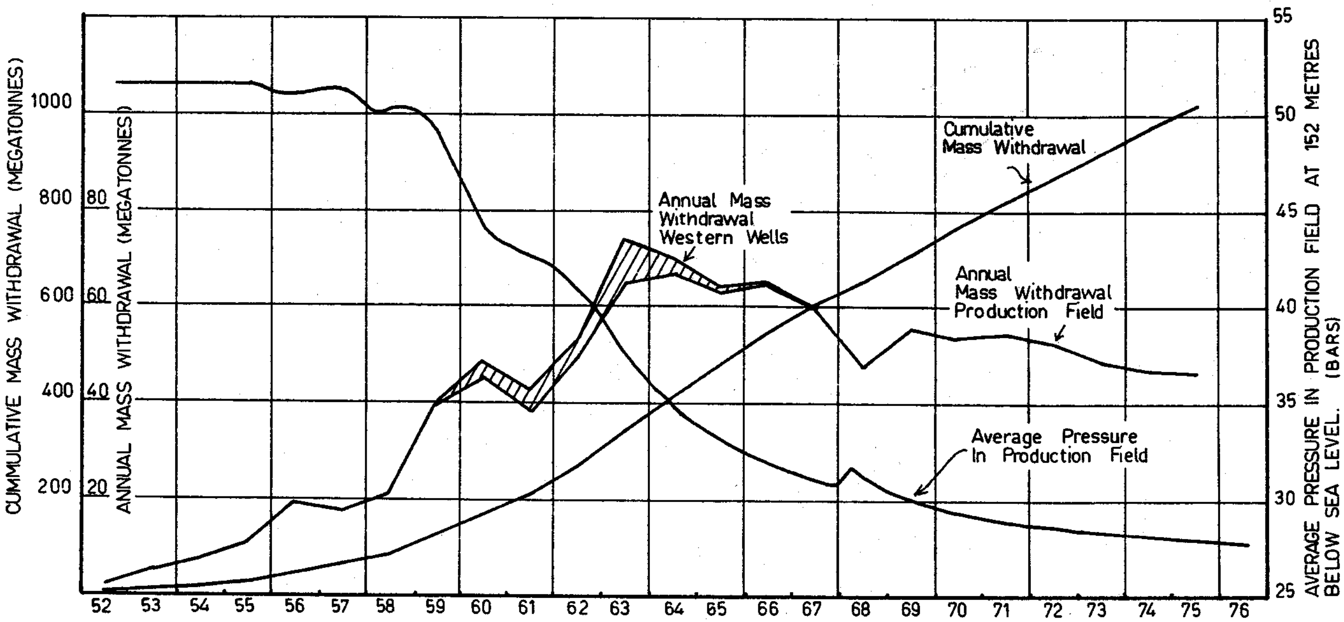


Figure 9.9.3 Wairakei hot water reservoir: mass withdrawal and aquifer pressure, 1952-1976. (Pre-1962 pressures after Bolton, 1970.)

About 1960 the subsidence rate at bench mark A97 began to increase and over the next few years the zone of rapid subsidence immediately north of the eastern production field shown on Figure 9.9.5 was delineated. Subsidence at bench mark A97 in this zone is shown on Figure 9.9.4. Subsidence of A97 between 1971 and 76 continued at 135 mm/year compared with 138 mm/year between 1966 and 71 as shown on Figure 9.9.4. Economic interest centres on this zone of subsidence as both the steam and waste water channels from the production field cross the subsiding basin. Bench marks in this zone are surveyed annually to third order standards.

Subsidence at bench marks AA8, at Tauhara, and AA15 to the west of the Wairakei production field are also shown on Figure 9.9.4.

9.9.9 HORIZONTAL MOVEMENT

The network which has been set up to measure horizontal movement at Wairakei has been described by Stilwell (1975), and Hatton (1970) showed the calculated and measured horizontal strain along the steam mains due to subsidence. The horizontal control network was re-surveyed in 1977 and vector directions shown by Stilwell were confirmed. Vector movement is generally toward the center of subsidence. Annual horizontal movement between 1968 and 77 was about 110 mm/year at a radius of 250 m from the centre of subsidence, decreasing to about 15 mm/year at 750 m radius.

9.9.10 CAUSE OF SUBSIDENCE

Subsidence in the area shown on Figure 9.9.5 must be related to the withdrawal of geothermal fluids. However, the more widespread subsidence as shown on Figure 9.9.1, although probably related to the underground hot water system, may be the result of natural events rather than withdrawal of fluids in the production area. Browne (1973) pointed out that at the Broadlands geothermal area (20 km NE of Wairakei) a natural rate of subsidence of 3.6 mm/year may have been occurring for the last 3400 years. Withdrawal of fluid at Wairakei has resulted in a number of continuing changes. The most significant of these is the overall lowering of hydrostatic pressures in the aquifer and the creation of a steam zone in the upper part of the aquifer in the production field. Computer modelling by Pritchett (1977) suggests that the gradual lowering of temperatures in this zone of saturated steam has been a major factor in controlling the

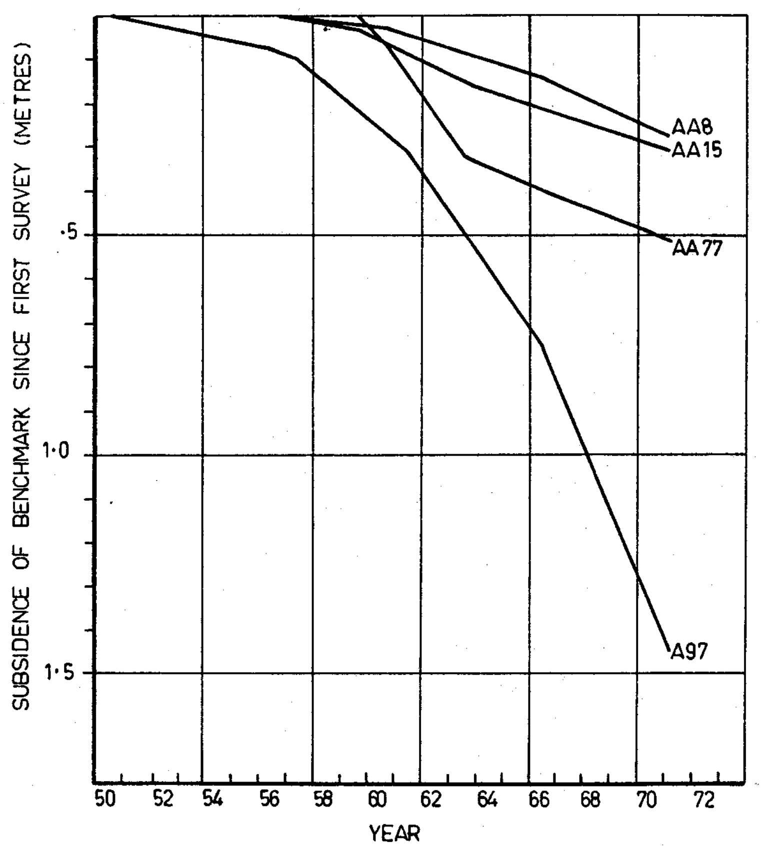


Figure 9.9.4 Subsidence at selected bench marks. For locations see Figure 9.9.1. Benchmark A97 first surveyed 1950, AA8 and AA15 1956, and A77 1959.

location and magnitude of subsidence. McNabb (1977) has suggested that pressure changes in the aquifer allowing cool surface water to penetrate fissures in the "caprock" and cool thick sections of underlying formations could account for the observed subsidence.

It is probable that the observed subsidence is the result of falling hydrostatic pressure in the deeper part of the aquifer system, falling steam pressure in the upper part of the aquifer and possibly the intrusion of cold water from the surface water table into the aquifer system. There has been no evidence of subsidence causing casing protrusion. Most casing strings are cemented into the Huka Falls Formation, thus the formation causing subsidence must lie below the Huka "caprock."

Hatton and Stilwell drew a correlation between the thickness of the producing aquifer (Waiora Formation) and the amount of subsidence.

9.9.11 ECONOMIC IMPACT OF SUBSIDENCE

The major structures affected by subsidence are the steam pipelines from the production field to the power house and the channels carrying separated geothermal water to waste. Differential subsidence has had no observed effect either on the power house or on ancillary buildings around the production field.

Both horizontal and vertical movement have occurred along the steam mains route. Maximum movement is near bench mark A97 (Figure 9.9.5), where horizontal movement is about 75 mm/year and vertical movement 130 mm/year. As the steam mains cross the edge of the subsiding basin, different sections are put in tension and compression (Hatton, 1970). Movement is accommodated to some extent by expansion loops which were built into the pipelines to allow for thermal

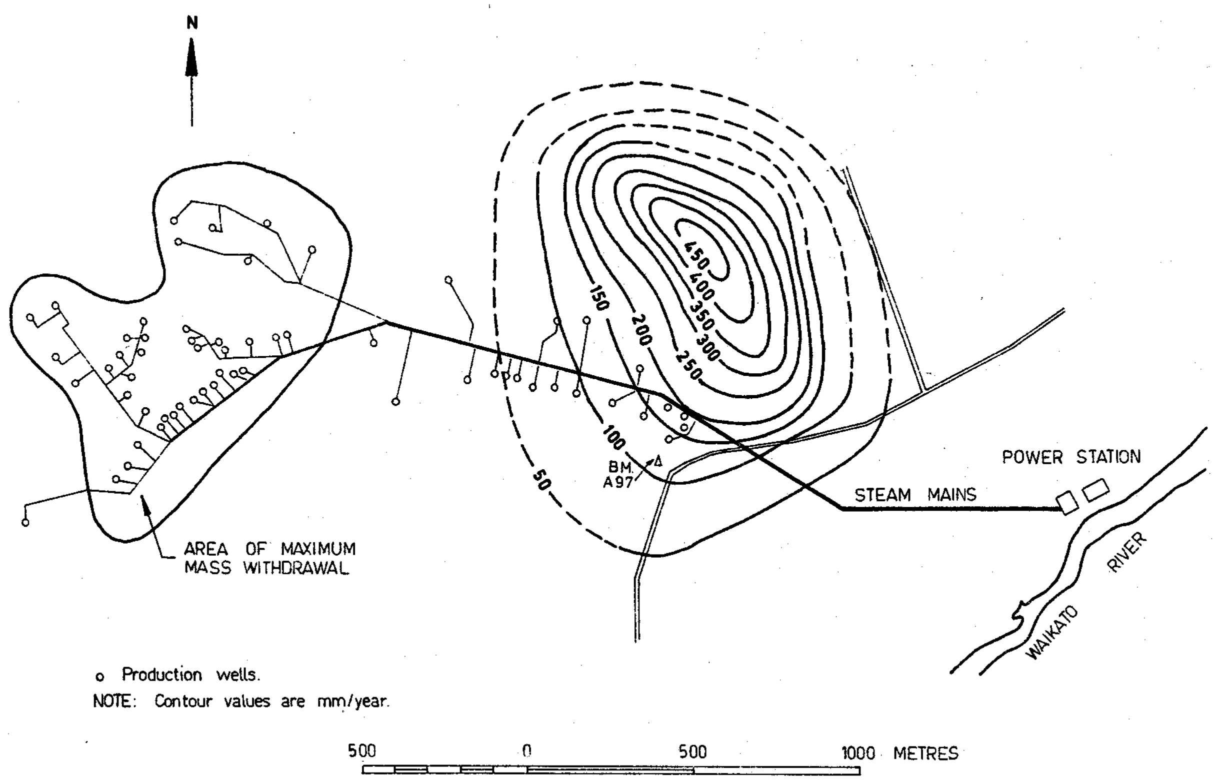


Figure 9.9.5 Subsidence rate in Wairakei production field, 1964-74. Redrawn from Figure 1, in Stilwell, Hall, and Tawhai, 1975, "Ground Movement in New Zealand Geothermal Fields," in Proceedings of Second U.N. Symposium on the Development and Use of Geothermal Resources: San Francisco, May 1975.

expansion. When these loops reach the limit of the travel, sections are either added or removed from the pipeline to restore the loops to their proper operating positions.

The waste water channels have a special sliding joint to allow movement between different sections of the reinforced concrete lining.

No measures are taken to control the rate or location of subsidence. Instead, the amount of subsidence and its effects are closely monitored in areas of economic interest and remedial action taken when installations are endangered.

9.9.12 ACKNOWLEDGMENT

The permission of Mr. N. C. McLeod, Commissioner of Works, to publish this paper is acknowledged.

9.9.13 REFERENCES

BOLTON, R. S. 1970. The behaviour of the Wairakei geothermal field during exploitation. Geothermics, Special Issue 2, pp. 1426-1439.

BROWNE, P. R. L. 1973. Geology, mineralogy and geothermometry of the Broadlands geothermal field, Taupo volcanic zone, New Zealand. Submitted for Ph.D. thesis at Victoria University of Wellington.

GRINDLEY, G. W. 1965. The geology, structure and exploitation of the Wairakei geothermal field, Taupo, New Zealand. Bulletin, N.Z. Geological Survey, 75.

HATTON, J. W. 1970. Ground subsidence of a geothermal field during exploitation. Geothermics, Special Issue 2, pp. 1294-1296.

MCNABB, A. 1977. Ground subsidence and reinjection. Internal report, Applied Maths Division, Department of Scientific and Industrial Research.

PRITCHETT, J. W. 1977. Geohydrological environmental effects of geothermal power production, phase IIA. Systems, Science and Software Report SSS-R-77-2998.

STILWELL, W. B., HALL, W. K., and J. TAWAHAI. 1975. Ground movement in New Zealand geothermal fields. Proceedings of second United Nations symposium on the development and use of geothermal resources.

Table 9.10.1 Aquifer characteristics

Aquifer	Location of tested well	Coefficient of transmissibility [m^2/hr]	Permeability [m/hr	Storage coefficient
Bangkok	Bang Pun	160	3.40	1.00×10^{-4}
Phra Pradaeng	Pom Phra	110	3.74	-
	Chun Navy Base Phra Pradaeng	70	2.38	-
	Wat Phai Ngoen	120	2.38	1.00×10^{-4}
Nakhon Luang	Wat Phai Ngoen	65	2.21	1.00×10^{-4}
	Lum Phini Park	100	3.40	2.00×10^{-4}
	Pak Kret	110	2.55	-
	Bang Bua	125	3.45	2.2×10^{-4}
	Dept. of Mineral Resources	50	2.65	2.60×10^{-4}

Table 9.10.2.Ground water pumpage for public water supply in Bangkok[1]

Year	Pumping rate (m^3/day)	Year	Pumping rate (m^3/day)
1965	84,314	1971	331,966
1966	199,170	1972	318,276
1967	316,963	1973	362,738
1968	342,963	1974	370,032
1969	310,027	1975	350,000
1970	307,540	1976	345,000

[1]Source of data: Bangkok Metropolitan Water Works Authority.

a

Case History No. 9. 10. Bangkok, Thailand, compiled by Soki Yamamoto, Rissho University, Tokyo, Japan

9.10.1 GEOLOGIC FORMATIONS AND GROUND WATER

The Lower Central Plain, approximately 120 kilometres in width and 200 kilometres in length, was originally formed by the accumulation of clastic sediments more than 2,000 metres thick in the fault/flexure depression since Tertiary time (Figure 9.10.1).

The ground surface of Bangkok is entirely underlain by blue to gray marine clay up to 30 metres thick, known as the Bangkok Clay. The upper 15 metres of the Bangkok Clay, generally called the Bangkok Soft Clay, is very soft and highly compressible. The lower part, referred to as the Bangkok Stiff Clay, which is rather stiff and less compressible, extends to an average depth of 25-30 metres. The water in these clays is very saline and salty.

The water-bearing formations of Bangkok consist mainly of sands and gravels with minor clay lenses. They are similar in occurrence and composition but can be zoned according to the geoelectrical properties (Figure 9.10.2) into 8 principal artesian aquifers, separated by thick confining clay or sandy clay layers; namely:

Bangkok Aquifer (50 m zone),
Phra Pradaeng Aquifer (100 m zone),
Nakhon Luang Aquifer (150 m zone),
Nonthaburi Aquifer (200 m zone),
Sam Khok Aquifer (300 m zone),
Phaya Thai Aquifer (350 m zone),
Thon Buri Aquifer (450 m zone),
Pak Nam Aquifer (550 m zone).

Aquifer characteristics of three aquifers are listed in Table 9.10.1. These aquifers generally extend the full width and length of the Plain. Most wells in Bangkok penetrate the second, third and fourth aquifers because they are highly productive, with a Coefficient of Transmissibility of 40-130 m^2hr (150,000-250,000 gallons per day per foot), and yield water of relatively excellent quality. The first aquifer, immediately beneath the Bangkok Clay, gives saline water whereas the fifth and sixth aquifers are not popular due to their greater depths and water of inferior quality. The seventh and the eighth have been proved to yield fresh water but have been tapped by only few wells. The sediments at depths from 650 metres to the metamorphic basement rocks at about 2,000-3,000 metres have been indicated by electric well logging to yield brackish to saline water. In the northern part of the Lower Central Plain, however, fresh ground water could be obtained from the first aquifer.

Ground water has been exploited for domestic supply in Bangkok for the past six or seven decades, but heavy utilization began in 1957 when the surface water for domestic and industrial use could not meet demand. For many years, about one third of the total public water supply in Bangkok has come from the aquifers (Table 9.10.2.).

It is estimated that the present total pumpage for domestic and industrial use is as high as 700,000 m^3/day. This pumping rate exceeds the safe yield and brings about an acute problem of water level decline. At an early stage of development, the water level was about at the ground surface but it gradually fell until cones of depression developed in many areas. In 1958-1959 the water level in the center of Bangkok was about 8-9 metres from the ground surface while that in the suburbs was 4.5-6 metres. Since 1967 a remarkable change of water level could be observed. During 1968-1969 the depth to water level in heavily pumped area was 22-25 metres, and 10-12 metres in the suburbs. At present the general depth to water level is 30 metres while that at the center of the cone of depression is in excess of 33 metres (Figure 9.10.3). The annual rate of decline in the water level is now as high as 3-4 metres for the 100-metre, 150-metre and 200-metre aquifers. In the 50-metre aquifer the water level is also falling about 1 metre a year due to the interception of recharging water at the northern part of the Plain. The consequences of heavy pumpage are not only the over-draft of aquifers but also the salt water encroachment into the southern part of the Bangkok Metropolis and the possibility of land subsidence.

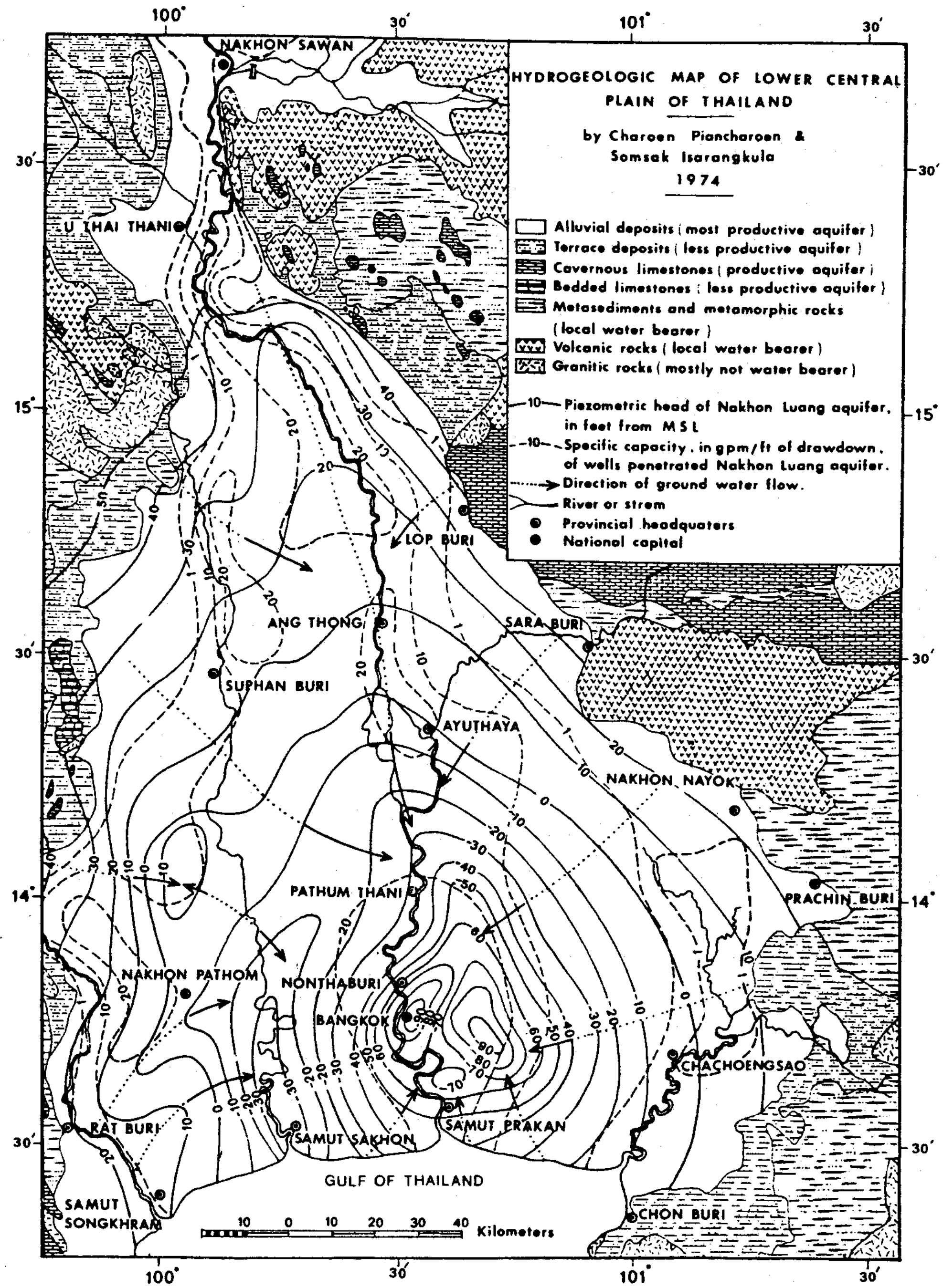


Figure 9.10.1 Hydrogeologic map of the Lower Central Plain of Thailand (after Piancharoen, 1977, Figure 1).

9.10.2 PROBLEMS OF LAND SUBSIDENCE

There is no serious damage due to land subsidence in Bangkok at the present time although flooding in localized areas is believed to be a result of a reduction in the altitude of the ground surface. However, the possibility of land subsidence due to the effect of deep well pumping has been spoken of by soil scientists for many years but no definite scientific proofs could be issued. No systematic investigation or observation leading to a reliable quantitative expression of subsidence behavior has been made to date.

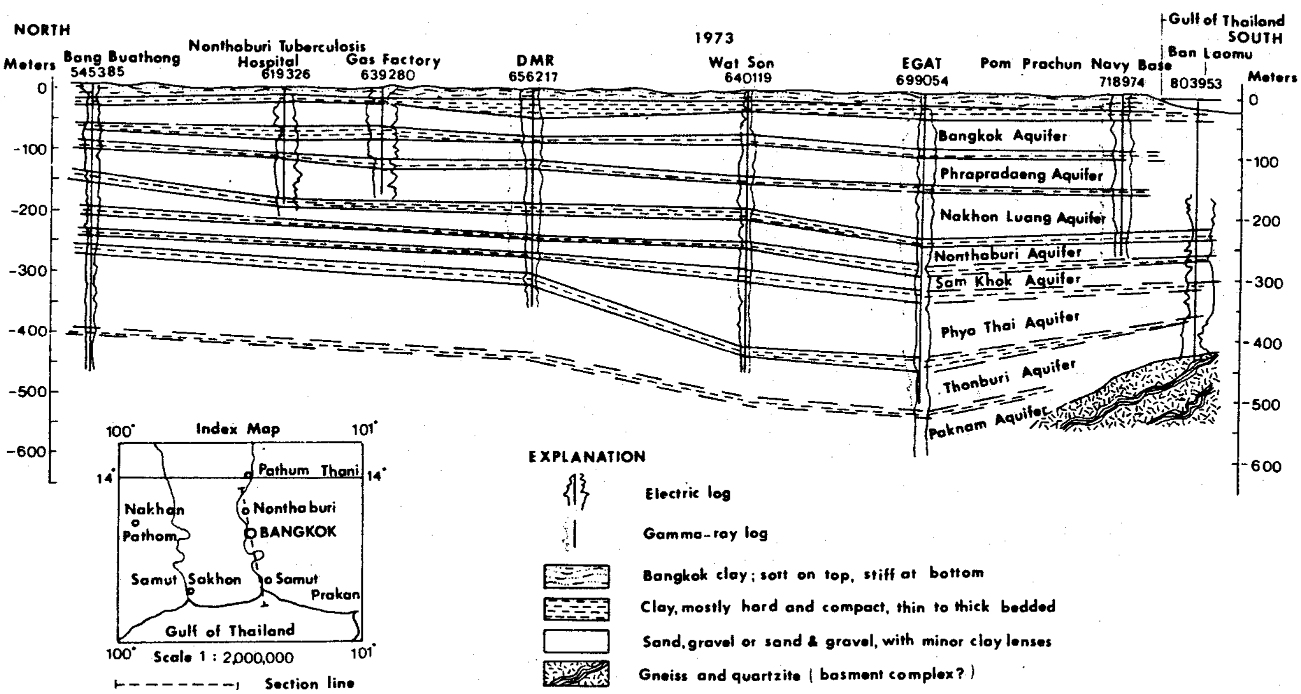


Figure 9.10.2 Hydrogeologic north-south section of the Lower Chao Phraya Delta showing principal aquifers of Bangkok Metropolis (correlated by electric and gamma-ray logs) (after Piancharoen, 1977, Figure 2).

9.10.3 INVESTIGATION PROGRAMS

Since the tests and accompanying evidence are far from conclusive, the problem of land subsidence and whether Bangkok is sinking is still debatable. Many geologists and hydrologists believe that the present deep well pumpage, mostly below 150 metres, has no effect on land subsidence, and if there is any subsidence external loads are to blame. Local flooding is also believed to be due to poor drainage in Bangkok. Three projects are now being submitted for consideration; namely, the leveling in the Bangkok Metropolitan Area for the investigation of land subsidence, the investigation of land subsidence caused by deep well pumping, and the development and management studies of ground water resources in the Bangkok area. These programs will be interrelated and aimed for completion within four years with a total budget of about 1.5 million U.S. dollars.*

9.10.4 SELECTED REFERENCES

PIANCHAROEN, C. 1977. Ground water and land subsidence in Bangkok, Thailand. IAHS. Pub. No. 121, pp. 355-364.

PIANCHAROEN, C., and C. CHUAMTHATSONG. 1978. Ground water of Bangkok Metropolis, Thailand. IAH Memoire, Vol. XI (Budapest), pp. 510-528.

---

* According to Dr. Prinya Nutalaya of the Asian Institute of Technology, progressive protrusion of water-well casings has been noted in Bangkok (oral communication to Joseph F. Poland, September 1978). This would suggest the beginnings of sediment compaction and land subsidence.

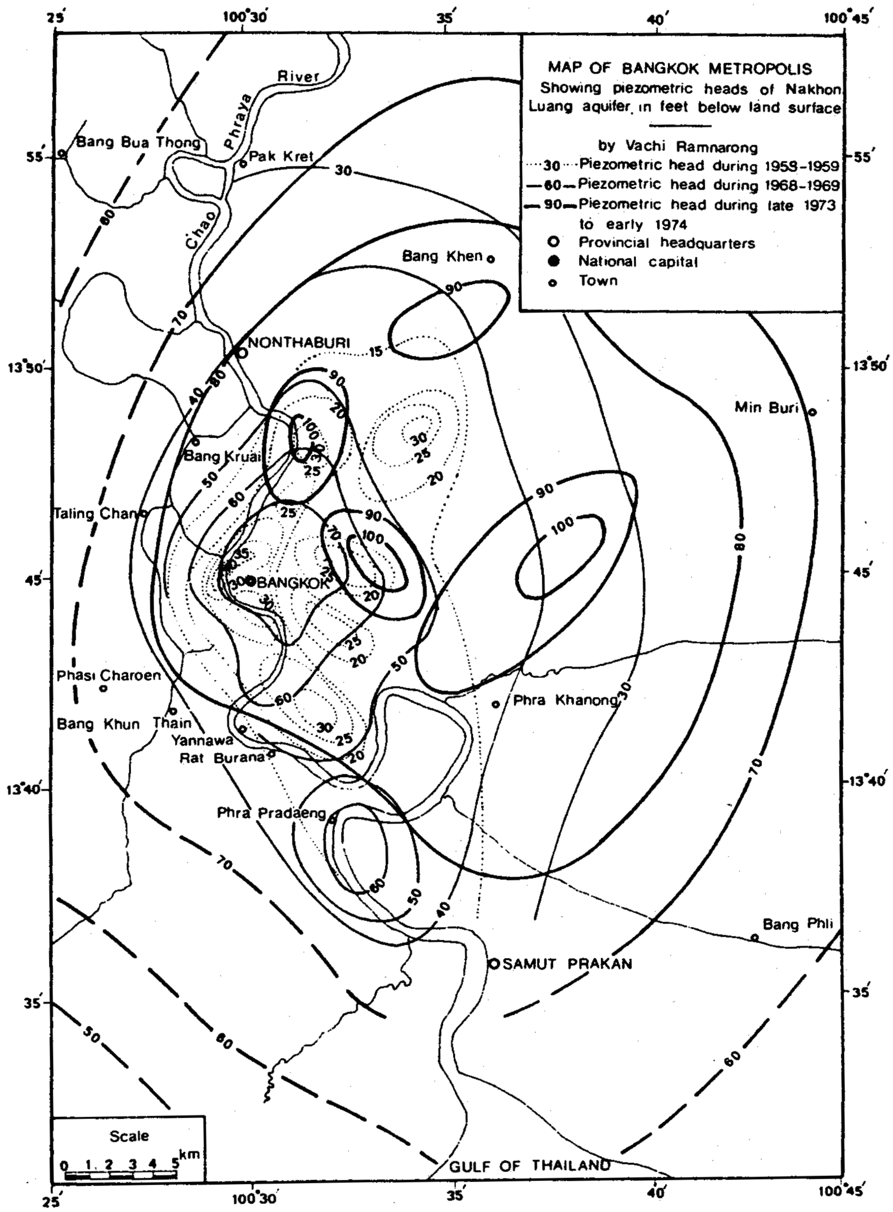


Figure 9.10.3 Water level map of the Nakhon Luang Aquifer (after Piancharoen, 1977, Figure 3).

Case History No. 9.11. Alabama, U.S.A., by J. G. Newton, U.S. Geological Survey, Tuscaloosa, Alabama

9.11.1 INTRODUCTION

Sinkholes in Alabama are divided into two categories defined as "induced" and "natural." Induced sinkholes are those related to man’s activities whereas natural sinkholes are not. Induced sinkholes are further divided into two types: those resulting from a decline in the water table due to ground-water withdrawals and those resulting from construction. Those resulting from a decline in the water table, the subject of this case history, far outnumber those resulting from all other causes. Information presented here consists of excerpts taken from five reports by the author. These reports, approved for publication by the Director, U.S. Geological Survey, are listed with the references cited in this case history. They resulted from investigations by the U.S. Geological Survey made in cooperation with the Geological Survey of Alabama and/or the Alabama Highway Department.

9.11.2 GEOLOGIC AND HYDROLOGIC SETTING

The terrane used to illustrate sinkhole development is a youthful basin underlain by carbonate rocks such as limestone and dolomite (Figure 9.11.1). The basin contains a perennial or near-perennial stream. This particular terrane is used because it is very similar to that of 10 active areas of sinkhole development in Alabama that have been examined by the author. Factors related to the development of sinkholes that have been observed in these areas are generally applicable to other carbonate terranes. The terrane illustrated differs from those examined only in the inclination of beds, which is shown as horizontal for ease of illustration.

The development of sinkholes is primarily dependent on past and present relationships between carbonate rocks and water, climatic conditions, vegetation, and topography, and on the presence or absence of residual or other unconsolidated deposits overlying bedrock. The source of water associated with the development of sinkholes is precipitation which, in Alabama, generally exceeds 1,270 mm annually. Part of the water runs off directly into streams, part replenishes soil moisture but is returned to the atmosphere by evaporation and transpiration, and the remainder percolates downward below the soil zone to ground-water reservoirs.

Water is stored in and moves through interconnected openings in carbonate rocks. Most of the openings were created, or existing openings along bedding planes, joints, fractures, and faults were enlarged by the solvent action of slighly acidic water coming in contact with the rocks. Water in the interconnected openings moves in response to gravity from higher to lower altitudes, generally toward a stream channel where it discharges and becomes a part of the streamflow.

Water in openings in carbonate rocks occurs under both water-table and artesian conditions; however, this study is concerned primarily with that occurring under water-table conditions. The water table is the unconfined upper surface of a zone in which all openings are filled with water. The configuration of the water table conforms somewhat to that of the overlying topography but is influenced by geologic structure, withdrawal of water, and variations in rainfall. The lowest altitude of the water level in a drainage basin containing a perennial stream occurs where the water level intersects the stream channel (Figure 9.11-1). Openings in bedrock underlying lower parts of the basin are water filled. This condition is maintained by recharge from precipitation in the basin. The water table underlying adjacent highland areas within the basin occurs at higher altitudes than the water table near the perennial stream. Openings in bedrock between the land surface and the underlying water table in highland areas are air filled (Figure 9.11.1).

The general movement of water through openings in bedrock underlying the basin, even though the route may be circuitous, is toward the stream channel and downstream under a gentle gradient approximating that of the stream. Some water moving from higher to lower altitudes is discharged through springs along flanks of the basin because of the intersection of the land surface and the water table. The velocity of movement of water in openings underlying most of the lowland

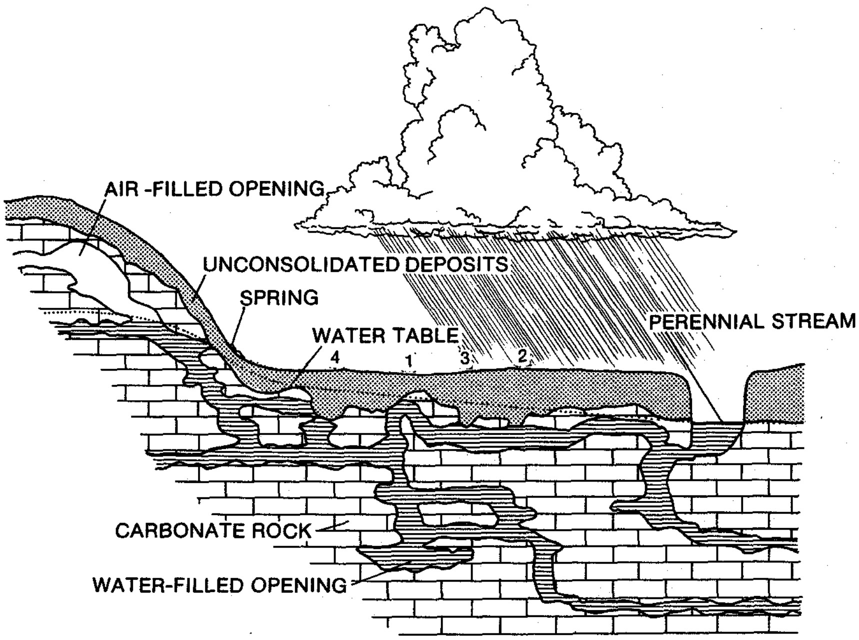


Figure 9.11.1 Schematic cross-sectional diagram of basin showing geologic and hydrologic conditions. (Numbers apply to sites described in report.)

area is probably sluggish when compared to that in openings at higher altitudes.

A mantle of unconsolidated deposits consisting chiefly of residual clay (residuum), that has resulted from the solution of the underlying carbonate rocks, generally covers most of the bedrock in the typical basin described. Alluvial or other unconsolidated deposits often overlie the residual clay. The residuum commonly contains varying amounts of chert debris that are insoluble remnants of the underlying bedrock. Some unconsolidated deposits are carried by water into openings in bedrock. These deposits commonly fill joints, fractures, or other openings enlarged by solution that underlie the lowland areas. The buried contact between the residuum and the underlying bedrock, because of differential solution, can be highly irregular (Figure 9.11.1).

9.11.3 CAUSE

A relationship between the formation of sinkholes and high pumpage of water from new wells was recognized in Alabama as early as 1933 (Johnston, 1933). Subsequent studies in Alabama (Robinson and others, 1953; Powell and LaMoreaux, 1969; Newton and Hyde, 1971; Newton and others, 1973; and Newton, 1976) have verified this relationship. Dewatering or the continuous withdrawal of large quantities of water from carbonate rocks by wells, quarries, and mines in numerous areas in Alabama is associated with extremely active sinkhole development. Numerous collapses in these areas contrast sharply with their lack of occurrence elsewhere.

Two areas in Alabama in which intensive sinkhole development has occurred and is occurring have been studied in detail. Both areas were made prone to the development of sinkholes by major declines of the water table due to the withdrawal of ground water. The formation of sinkholes in both areas resulted from the creation and collapse of cavities in unconsolidated deposits caused by the declines (Newton and Hyde, 1971; Newton and others, 1973). The growth of one such cavity in Birmingham has been photographed through a small adjoining opening (Newton, 1976).

Previous reports have described only indirectly or in part the hydrologic forces resulting from a decline in the water table that create or accelerate the growth of activities that collapse and form sinkholes. These forces, based on studies in Alabama (Newton and Hyde, 1971; Newton and others, 1973), are (a) a loss of support to roofs of cavities in bedrock previously filled with water and to residual clay or other unconsolidated deposits overlying openings in bedrock, (b) an increase in the velocity of movement of ground water, (c) an increase in the amplitude of water-table fluctuations, and (d) the movement of water from the land surface to openings in underlying bedrock where recharge had previously been rejected because the openings were water filled. The same forces creating cavities and subsequent collapses also result in subsidence. The movement of unconsolidated deposits into bedrock where the strength of the overlying material is not sufficient to maintain a cavity roof, will result in subsidence at the surface (Donaldson, 1963).

To demonstrate forces that result in the development of cavities and their eventual collapse, a schematic diagram is shown in Figure 9.11.2 that illustrates changes in natural geologic and hydrologic conditions previously described and shown in Figure 9.11.1. A description of the forces triggered by a lowering of the water table follows.

The loss of buoyant support following a decline in the water table can result in an immediate collapse of the roofs of openings in bedrock or can cause a downward migration of unconsolidated deposits overlying openings in bedrock. The buoyant support exerted by water on a solid (and hypothetically) unsaturated clay overlying an opening in bedrock, for instance, would be equal to about 40 per cent of its weight. This determination is based on the specific gravities of the constituents involved. Site 1 on Figure 9.11.1 shows the unconsolidated deposit overlying a water-filled opening in bedrock. Site 1 on Figure 9.11.2 shows the decline in the water table and the resulting cavity in the deposit formed by the downward migration, of the unconsolidated deposit caused by the loss of support.

The creation of a cone of depression in an area of water withdrawal results in an increased hydraulic gradient toward the point of discharge (Figure 9.11.2) and a corresponding increase in the velocity of movement of water. This force can result in the flushing out of the finer grained unconsolidated sediments that have accumulated in the interconnected openings enlarged by solution. This movement also transports unconsolidated deposits migrating downward into bedrock openings to the point of discharge or to a point of storage in openings at lower altitudes.

The increase in the velocity of ground-water movement also plays an important role in the development of cavities in unconsolidated deposits. Erosion caused by the movement of water through unobstructed openings and against joints, fractures, faults, or other openings filled with clay or other unconsolidated sediments results in the creation of cavities that enlarge and eventually collapse (Johnston, 1933; Robinson and others, 1953).

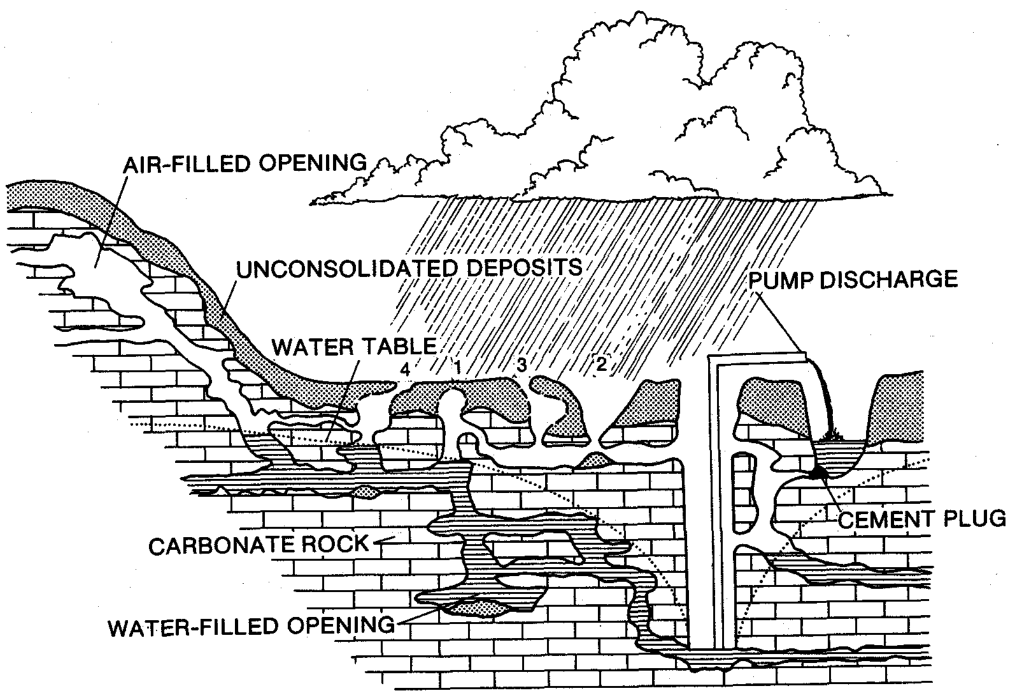


Figure 9.11.2 Schematic cross-sectional diagram of basin showing changes in geologic and hydrologic conditions resulting from water withdrawal. (Numbers apply to sites described in report.)

Pumpage results in fluctuations in ground-water levels that are of greater magnitude than those occurring under natural conditions. The magnitude of these fluctuations depends principally on variations in water withdrawal and on fluctuations in natural recharge. The repeated movement of water through openings in bedrock against overlying residuum or other unconsolidated sediments causes a repeated addition and subtraction of support to the sediments and repeated saturation and drying. This process might be best termed "erosion from below" because it results in the creation of cavities in unconsolidated deposits, their enlargement, and eventual collapse. Fluctuations of the water table against the roof of a cavity in unconsolidated deposits near Greenwood, Alabama, have been observed and photographed through a small collapse in the center of the roof. These fluctuations, in conjunction with the movement of surface water into openings in the ground, resulted in the formation of the cavity and its collapse (Newton and others, 1973).

A drastic decline of the water table in a lowland area (Figure 9.11.2) in which all openings in the underlying carbonate rock were previously water filled (Figure 9.11.1) commonly results in induced recharge of surface water. This recharge was partly rejected prior to the decline because the underlying openings were water filled. The quantity of surface water available as recharge to such an area is generally large because of the runoff moving to and through it from areas at higher altitudes.

The inducement of surface-water infiltration through openings in unconsolidated deposits interconnected with openings in underlying bedrock results in the creation of cavities where the material overlying the openings in bedrock is eroded to lower altitudes. Repeated rains result in the progressive enlargement of this type cavity. A corresponding thinning of the cavity roof due to this enlargement eventually results in a collapse. The position of the water table below unconsolidated deposits and openings in bedrock that is favorable to induced recharge is illustrated in Figure 9.11.2. Sites 2, 3, and 4 on Figure 9.11.2 illustrate a collapse and cavities in unconsolidated deposits that were formed primarily or in part by induced recharge. The creation and eventual collapse of cavities in unconsolidated deposits by induced recharge is the same process described by many authors as "piping" or "subsurface mechanical erosion" where it has been applied mainly to collapses occurring on noncarbonate rocks (Allen, 1969).

In an area of sinkhole development where a cone of depression is maintained by constant pumpage (Figure 9.11.2), all of the forces described are in operation even though only one may be principally responsible for the creation of a cavity and its collapse. For instance, the inducement of recharge from the surface (site 2 on Figure 9.11.2) where the water table is maintained at depths well below the base of unconsolidated deposits, can be solely responsible for the development of cavities and their collapse. In contrast, a cavity resulting from a loss of support (site 1 on Figure 9.11.2) can be enlarged and collapsed by induced recharge if it has intersected openings interconnected with the surface. In an area near the outer margin of the cone (site 4 on Figure 9.11.2), the creation of a cavity and its collapse can result from all forces. The cavity can originate from a loss of support; can be enlarged by the continual addition and subtraction of support and the alternate wetting and drying resulting from waterlevel fluctuations; can be enlarged by the increased velocity of movement of water; and can be enlarged and collapsed by water induced from the surface.

9.11.4 MAGNITUDE AND AREAL EXTENT

It is estimated that more than 4,000 induced sinkholes, areas of subsidence, or other related features have occurred in Alabama since 1900. Most of them have occurred since 1950. Almost all have resulted from a decline in the water table due to ground-water withdrawals.

Dewatering or the continuous withdrawal of large quantities of water from carbonate rocks by wells, quarries, and mines in numerous other areas in Alabama is associated with extremely active sinkhole development. Numerous collapses in these areas contrast sharply with their lack of occurrence in adjacent geologically and hydrologically similar areas where withdrawals of water are minimal. For example, in five areas examined by the author in north-central Alabama in Jefferson and Shelby Counties, an estimated 1,700 collapses, areas of subsidence, or other associated features have formed in a total combined area of about 36 km^2.

In Alabama, most induced sinkholes related to water withdrawals from wells, except those drilled specifically for dewatering purposes, were found within 150 m of the site of withdrawal. The yield of these wells commonly exceeds 22 l/s. Most sinkholes related to quarry operations were found within 600 m of the point of withdrawal; those related to mining operations can occur several kilometres from the point of withdrawal.

Recent collapses forming sinkholes in Alabama in areas in which large quantities of ground water are being withdrawn generally range from 1 to 90 m in diameter and from 0.3 to 30 m in depth. The largest, located in a wooded area in Shelby County, apparently occurred in a matter of seconds in December 1972. The collapse was about 90 m in diameter and 30 m deep (Figure 9.11.3).

9.11.5 ECONOMIC IMPACT

Costly damage and numerous accidents have occurred or nearly occurred in Alabama as a result of collapses beneath highways, streets, railroads, buildings, sewers, gas pipelines, vehicles, animals, and people. Unfortunately, no inventory of costs or loss in property values has been made. The maintenance and protection of highways in sinkhole prone areas indicate costs resulting from their development. The cost of filling collapses, leveling pavement and monitoring subsidence along less than a kilometre of Interstate Highway 59 in Birmingham, Alabama, during the period 1972-77 is estimated to have exceeded $250,000 (L. Lockell, oral commun.). The estimated cost of bridging a, part of this area, and planned safety measures for highways crossing two similar areas near Birmingham exceeds $4,660,000 (C. Kelly, oral commun.). The need for these protective measures is well illustrated by the damage to a warehouse in 1973 (Figure 9.11.4) that resulted from a collapse adjacent to Interstate Highway 59 in Birmingham.

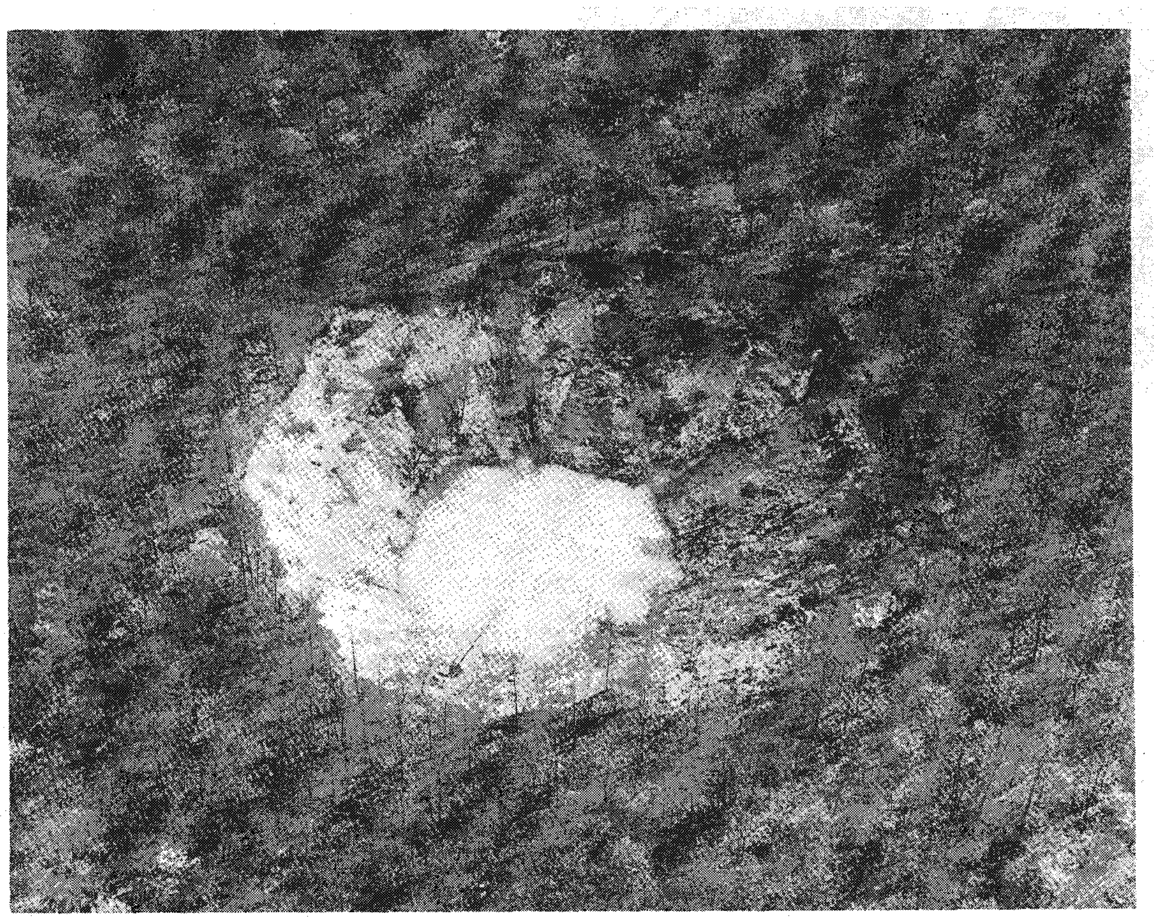


Figure 9.11.3 Sinkhole resulting from collapse near Calera in Shelby County, Alabama (photograph by Curtis Frizzell).

Figure 9.11.4 Collapse in warehouse near Interstate Highway 59 in Birmingham, Alabama (photograph by T. V. Stone).

9.11.6 CORRECTIVE MEASURES

Ideally, the development of sinkholes can be eliminated or minimized by ceasing the pumpage that causes the decline of the water table. The cessation of or drastic decrease in sinkhole activity following a recovery of the water table has been recognized previously (Foose, 1953; Newton and Hyde, 1971; Newton, 1976). Most efforts in Alabama have been directed toward measures minimizing sinkhole development and eliminating potential hazards and damage to structures rather than dealing with the cause. The measures that have been or will be utilized include bridging, adding additional support, the removal of unconsolidated deposits overlying bedrock, grouting, minimizing the diversion of natural drainage, and the construction of flumes and other impermeable drainage systems.

9.11.7 REFERENCES CITED

ALLEN, A. S. 1969. Geologic settings of subsidence, in Reviews in engineering geology: Geol. Soc. America, v. 2, p. 305-342.

DONALDSON, G. W. 1963. Sinkholes and subsidence caused by subsurface erosion: Regional Conf. for Africa on Soil mechanics and Foundation Eng., 3rd, Salisbury, Southern Rhodesia 1963 Proc., p. 123-125.

FOOSE, R. M. 1953. Ground-water behavior in the Hershey Valley, Pennslyvania: Geol. Soc. America Bull. 64, p. 623-645.

JOHNSTON, W. D., Jr. 1933. Ground water in the Paleozoic rocks of northern Alabama: Alabama Geol. Survey Spec. Rept. 16, 441 p.

NEWTON, J. G. 1976. Early detection and correction of sinkhole problems in Alabama, with a preliminary evaluation of remote sensing applications: Alabama Highway Dept., Bur. Research and Devel., Research Rept. No. HPR-76, 83 p.

___________., 1976. Induced and natural sinkholes in Alabama--a continuing problem along highway corridors, in Subsidence over mines and caverns, moisture and frost action, and classification: Natl. Acad. Sci. Transp. Research Rec. 612, p. 9-16.

___________., 1977. Induced sinkholes--a continuing problem along Alabama highways, in Proceedings of second international symposium on land subsidence: Internat. Assoc. Hydrol. Sci. Pub. No. 121, p-. 453-463.

NEWTON, J. G., and HYDE, L. W. 1971. Sinkhole problem in and near Roberts Industrial Subdivision, Birmingham, Alabama--a reconnaissance: Alabama Geol. Survey Circ. 68, 42 p.

NEWTON, J. G., COPELAND, C. W., and SCARBROUGH, L. W. 1973. Sinkhole problem along proposed route of Interstate Highway 459 near Greenwood, Alabama: Alabama Geol. Survey Circ. 83, 53 p.

POWELL, W. J., and LAMOREAUX, P. E. 1969. A problem of subsidence in a limestone, terrane at Columbiana, Alabama: Alabama Geol. Survey Circ. 56, 30 p.

ROBINSON, W. H., IVEY, J. B., and BILLINGSLEY, G. A. 1953. Water supply of the Birmingham area, Alabama: U.S. Geol. Survey Circ. 254, 53 p.

Case History No. 9.12. The Houston-Galveston Region, Texas, U.S.A., by R. K. Gabrysch, U.S. Geological Survey, Houston, Texas

9.12.1 INTRODUCTION

The Houston-Galveston region of Texas, as described in this report, includes all of Harris and Galveston Counties and parts of Brazoria, Fort Bend, Waller, Montgomery, Liberty, and Chambers Counties (Figure 9.12.1). Land-surface subsidence has become critical in parts of the region because some low-lying areas along Galveston Bay are subject to inundation by normal tides, and an even larger part of the region may be subject to catastrophic flooding by hurricane tides. Hurricanes resulting in tides of 3.0-4.6 metres above sea level strike the Texas coast on the average of once every 10 years.

Land-surface subsidence due to fluid withdrawals was first documented in the Goose Creek oil field in Harris County (Pratt and Johnson, 1926). Since then, numerous reports on subsidence in the Houston-Galveston region have attributed subsidence to the compaction of fine-grained material associated with the oil- and water-bearing sands. The more recent reports (Winslow and Doyel, 1954; Winslow and Wood, 1959; Gabrysch, 1969; and Gabrysch and Bonnet, 1975a) present data and interpretations of regional subsidence and its relation to the withdrawals of ground water for municipal supply, industrial use, and irrigation. The authors of these reports recognized that subsidence due to the removal of oil and gas has occurred in the region, but the data are not sufficient to describe in detail the localized areas of occurrence.

9.12.2 GEOLOGY AND HYDROLOGY OF THE HOUSTON-GALVESTON REGION

The aquifers in the Houston-Galveston region are composed of sand and clay beds that are not persistent in either lithology or thickness. The beds grade into each other both laterally and vertically within short distances; consequently, differentiation of geological formations on drillers' logs and electrical logs is almost impossible. However, by use of both the logs and the hydraulic properties of the aquifers, the subsurface units have been divided into three major aquifer systems and one confining system (Jorgensen, 1975).

The age of the geological formations composing the aquifers and the confining layer ranges from Miocene to Holocene. The deepest aquifer containing freshwater is the Jasper aquifer of Miocene age, which is separated from the overlying Evangeline and Chicot aquifers by the Burkeville confining layer. The two principal aquifer systems of the region are the Chicot aquifer of Pleistocene age and the Evangeline aquifer of Pliocene age. The Burkeville confining layer is probably part of the Fleming Formation of Miocene age.

The aquifers are under artesian conditions throughout most of the region, but little information on the hydraulic properties of the Jasper aquifer is available because it is undeveloped. Reports of test holes in the Jasper (W. F. Guyton, oral commun., 1977) indicate that the hydraulic head in the Jasper is above land surface, which probably approximates the original conditions. It is assumed that with no change in head, compaction of the deposits in the Jasper system has not occurred; therefore, the discussion of subsidence in this report will be restricted to a discussion of the Evangeline and Chicot aquifer systems.

The Evangeline aquifer system is composed of the Goliad Sand and possibly the upper part of the Fleming Formation. The system contains sands that yield freshwater of good quality in about the inland two-thirds of the region. The transmissivity of the aquifer system ranges from less than 460m^2/d to about 1,400 m^2d. The horizontal hydraulic conductivity is about 4.57 metres per day, and the storage coefficient ranges from about 0.00005 to more than 0.001.

The Chicot aquifer system is composed of the Willis Sand, Bentley Formation, Montgomery Formation, Beaumont Clay, and the Quaternary alluvium and includes the deposits from the land surface to the top of the Evangeline aquifer. The transmissivity of the Chicot aquifer ranges from 0 to about 1,858 m^2/d. The horizontal hydraulic conductivity is about twice that of the Evangeline aquifer, and the storage coefficient ranges from 0.00004 to 0.20. The larger values

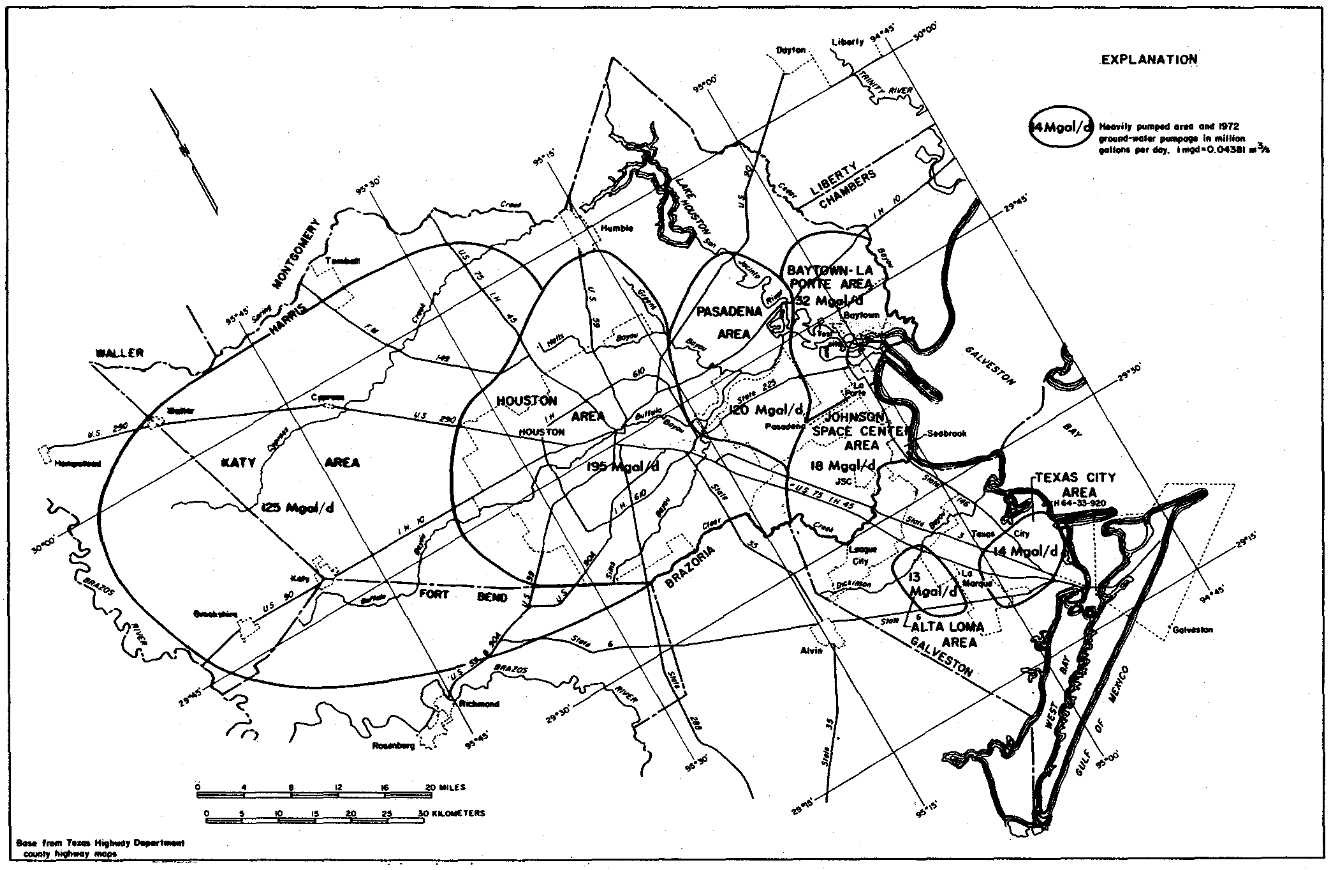


Figure 9.12.1 Locations of principal areas of ground-water withdrawals and average rates of pumping in 1972.

of the storage coefficient occur in the northern part of the region where the aquifer crops out and is partly under water-table conditions.

Both the Evangeline and the Chicot aquifer systems contain many layers of clay interbedded with the water-bearing sands. The clay beds are generally less than 6 metres thick, but locally they retard the vertical movement of water. Every sand bed, therefore, has a different hydraulic head. Data from cores of the clay beds were obtained at six sites for evaluation of subsidence in the Houston-Galveston region. The mineral composition of 27 samples from 5 sites were also determined (Gabrysch and Bonnet, 1975b and unpublished data). Montmorillonite is the principal mineral constituent of the clay beds, which also contain smaller amounts of illite, chlorite, and kaolinite.

9.12.3 DEVELOPMENT OF GROUND WATER

Development of ground water in the Houston-Galveston region for municipal supply and irrigation began in the 1890's. Ground-water withdrawals increased gradually to about 4.4 M^3/s (cubic metres per second) with population growth, increased irrigation, and industrial use until the late 1930's. Construction of the large industrial complex in the region began in 1937, and by 1954 ground-water pumping had increased to about 18 m^3/s.

Ground-water pumping decreased to about 14 m^3/s by 1959 because of the introduction of a supply of surface water in 1954 from nearby Lake Houston on the San Jacinto River. By 1962, ground-water pumping was again at a rate of about 18 m^3/s.

Pumping of ground water for municipal supply, industrial use, and irrigation was approximately 46 per cent, 33 per cent, and 21 per cent, respectively, of the total of 23 m^3/s pumped in 1972. The principal areas of pumping and the average daily rates of pumping in 1972 in each area are shown on Figure 9.12.1. Pumping in 1975 for all uses was 22 m^3/s.

The pumping of larger amounts of ground water has resulted in water-level declines during 1943-73 of as much as 61 metres in wells completed in the Chicot aquifer and as much as 99 metres in wells completed in the Evangeline aquifer (Figures 9.12.2 and 9.12.3). The maximum average annual rate of water-level decline for 1943-73 was 2.0 metres in the Chicot aquifer and 3.3 metres in the Evangeline aquifer. During 1964-73, the maximum rate of decline was 3.0 metres in the Chicot and 5.4 metres in the Evangeline.

9.12.4 SUBSIDENCE OF THE LAND SURFACE

The area of the greatest amount of subsidence coincides with the area of the greatest amount of artesian-pressure decline, which is east-southeast of Houston at Pasadena. Figure 9.12.4 shows that as much as 2.3 metres of subsidence occurred at Pasadena between 1943 and 1973. It should be noted, however, that within the entire region of subsidence, more than one center occurs. These areas are indicated by the closed contours on Figure 9.12.4.

Some of the centers of subsidence may be associated with the pumping of oil and gas and some may be associated with the pumping of ground water. Additional complications in analyzing the causes and areal distribution of subsidence result from the varying thicknesses of individual beds of fine-grained material, the varying total thickness of fine-grained material, the vertical distribution of changes in artesian head, and the relation of compressibility to depth of burial. An example of the effects of compressibility and depth of burial occurs in the southern part of Harris County where about 55 per cent of the subsidence is due to compaction in the Chicot aquifer, which composes only the upper one-fourth of the estimated compacting interval.

Figure 9.12.5 shows subsidence for 1964-73. The maximum amount of subsidence during this period was about 1.1 metres. The indicated maximum average rate for the 9-year period is about 0.12 metre per year as compared to the maximum average rate of 0.08 metre per year for the 30 year period 1943-73. During the last part of the 1943-73 period, the rate of subsidence accelerated, and the area of subsidence increased. The area in which subsidence is 0.3 metre or more increased from about 906 square kilometres in 1954 to about 6,475 square kilometres in 1973.

The maps showing the amounts of subsidence (Figures 9.12.4 through 9.12-6) were constructed from data obtained from the leveling program of the National Geodetic Survey (formerly the U.S. Coast and Geodetic Survey) supplemented by data obtained from local industries. Some subsidence occurred before 1943, but the amount is difficult to determine. However, an approximation of the amount and extent of the subsidence that occurred between 1906 and 1943 is shown on Figure 9.12.6. By 1943, four centers of subsidence were apparent. The centers at Pasadena, Baytown, and Texas City were the result of ground-water pumping; and the center in the Goose Creek oil field resulted from the production of oil, gas, and saltwater.

Because of the nature of deposition of the aquifer systems, each sand bed has a different hydraulic head, and each clay layer is under a different amount of stress. The water-level declines shown by Figures 9.12.2 and 9.12.3 are the maximum declines that have occurred in each of the aquifers. Water-level measurements indicate that the water table is approximately at its original position (about 2 to 6 metres below land surface). Piezometers installed at different depths at each of eight sites are used to define the potentiometric profiles. The differences between the measurements in the piezometers and the original potentiometric surfaces define the stress profile. As an example, at a site in the Pasadena area, the depths to water below land surface in January 1978 were as follows:

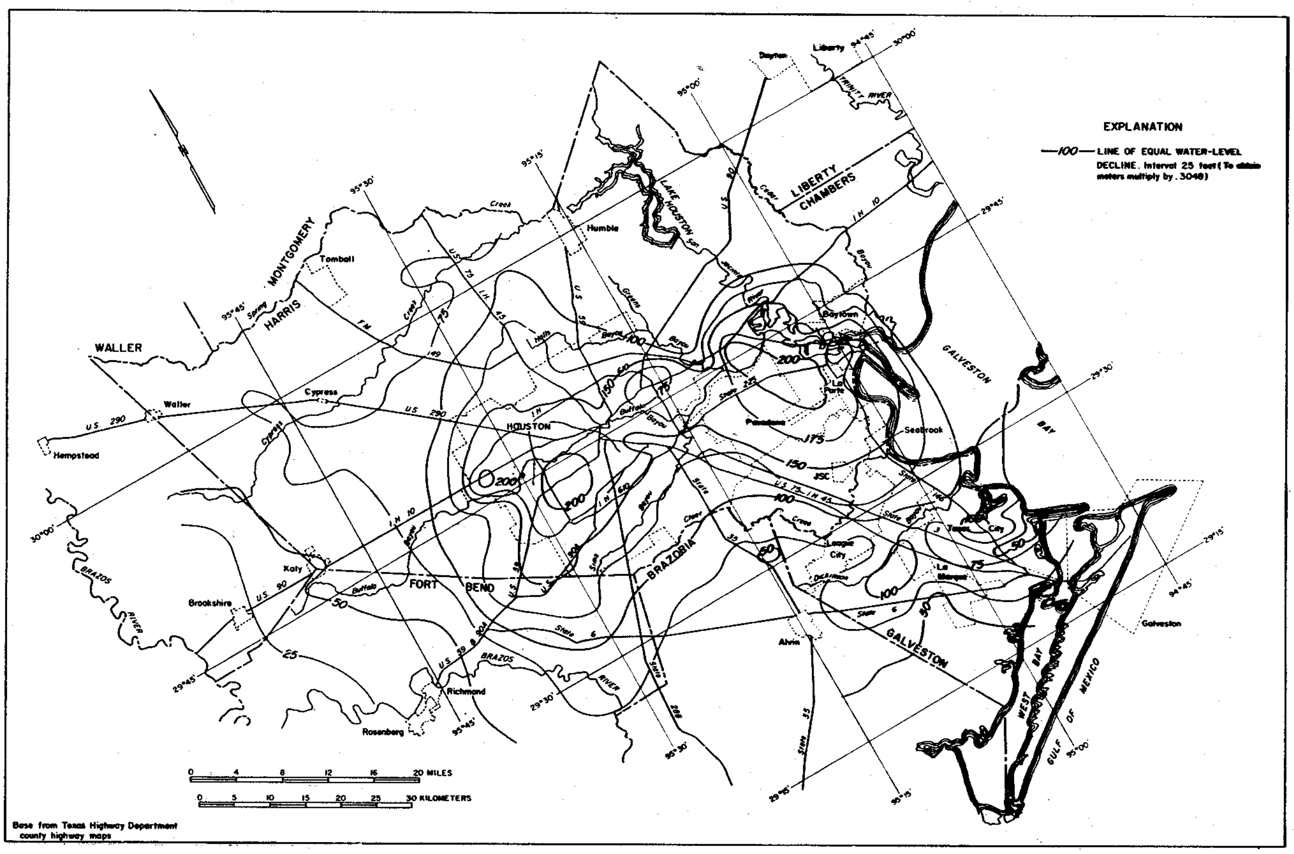


Figure 9.12.2 Approximate declines of water levels in wells completed in the Chicot aquifer, 1943-73.

Piezometer depth (metres)	Depth to water (metres)
10	1.85
30	4.31
119	45.21
221	100.31
284	102.74
403	100.44
552	93.20
828	47.28

The potentiometric surface in each of the two aquifer systems was 15 to 30 metres above land surface before large withdrawals began.

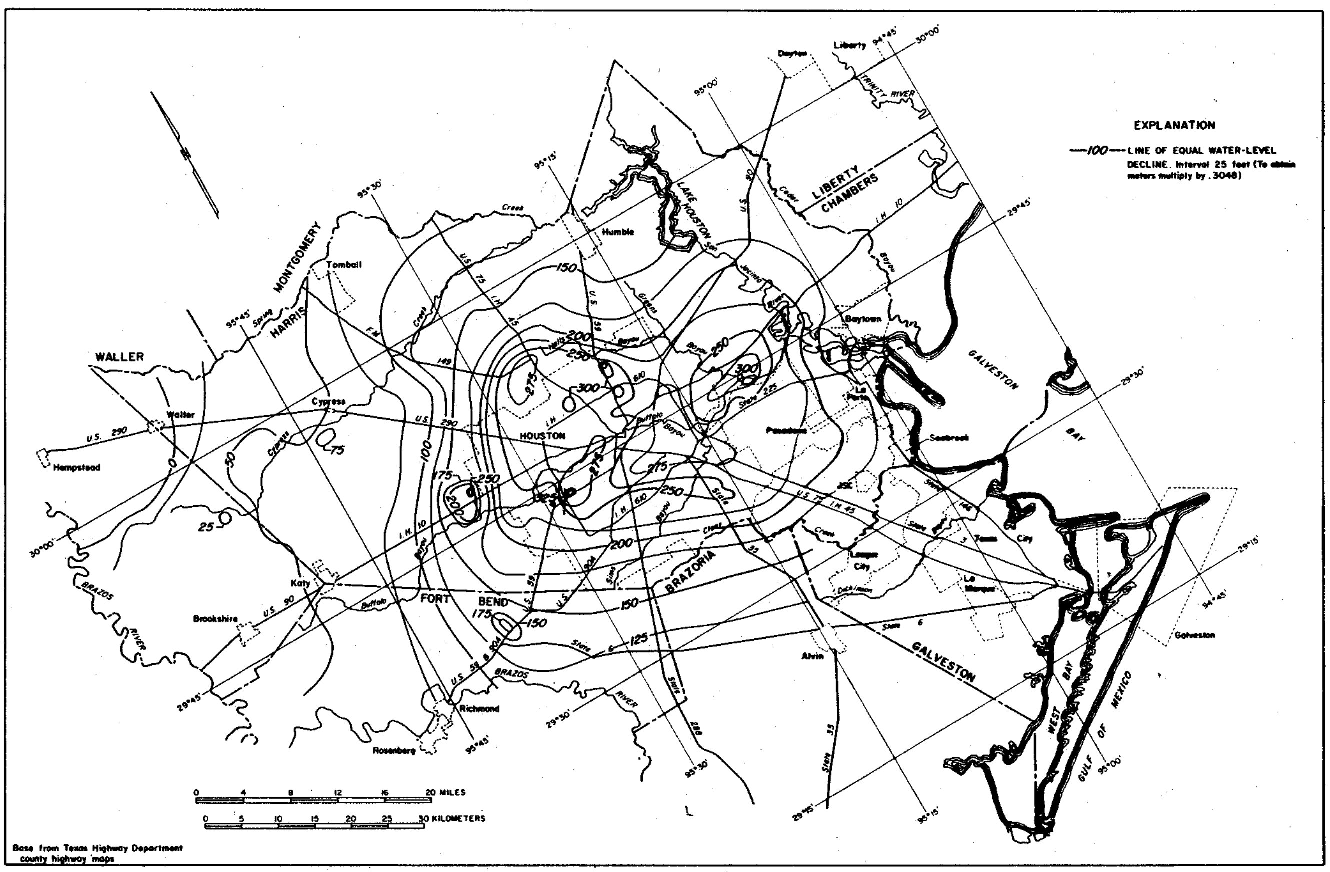


Figure 9.12.3 Approximate declines of water levels in wells completed in the Evangeline aquifer, 1943-73.

The compressibility of the aquifer system has been estimated at two locations. At Seabrook, it is assumed that no compaction due to ground-water pumping occurred below a depth of about 610 metres. Above 610 metres, the sediments include about 243.5 metres of fine-grained material, and the average stress applied to the system during 1943-73 was estimated to be a change in head of 38.6 metres of water. Subsidence during 1943-73 was 0.91 metre; therefore, the compressibility of the fine-grained materials was determined to be

$$0.91 \text{ m}/(243.5 \text{ m})\ (38.6) = 9.7 \times 10^{-5} \text{m}^{-1}.$$

At Texas City, it is assumed that no compaction due to ground-water pumping occurred below a depth of 506 metres. Above 506 metres, the sediments include about 151.5 metres of fine-

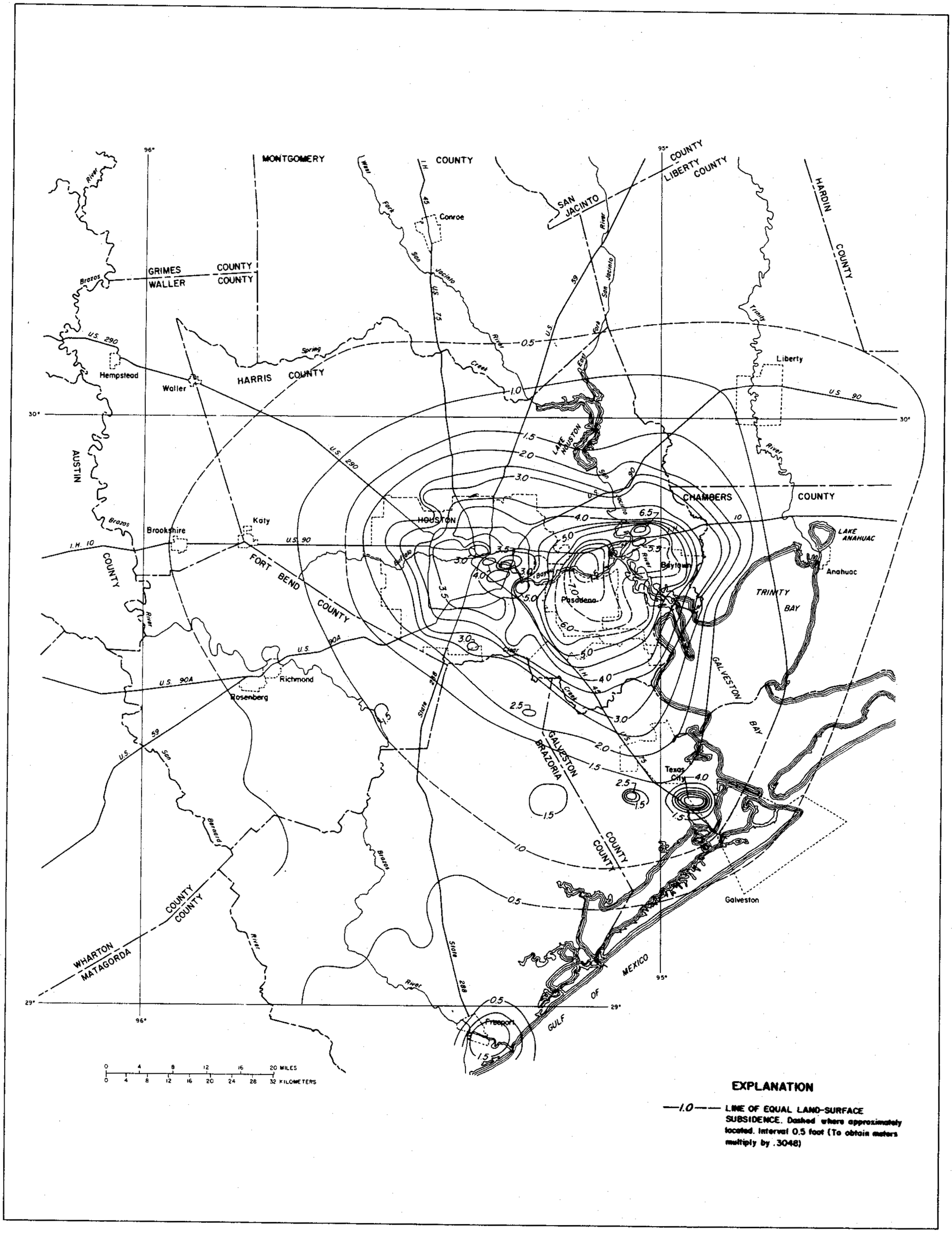


Figure 9.12.4 Subsidence of the land surface, 1943-73.

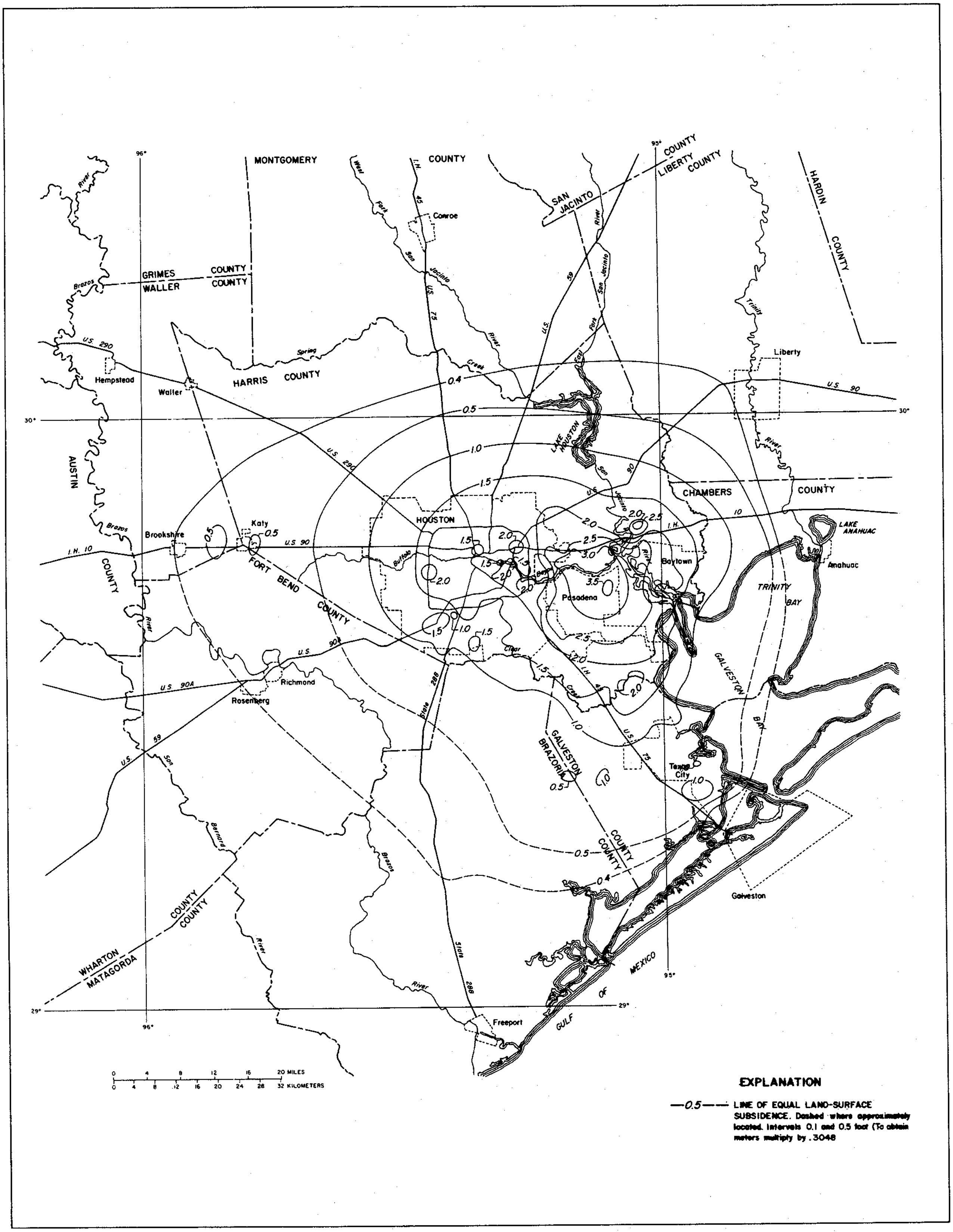


Figure 9.12.5 Subsidence of the land surface, 1964-73.

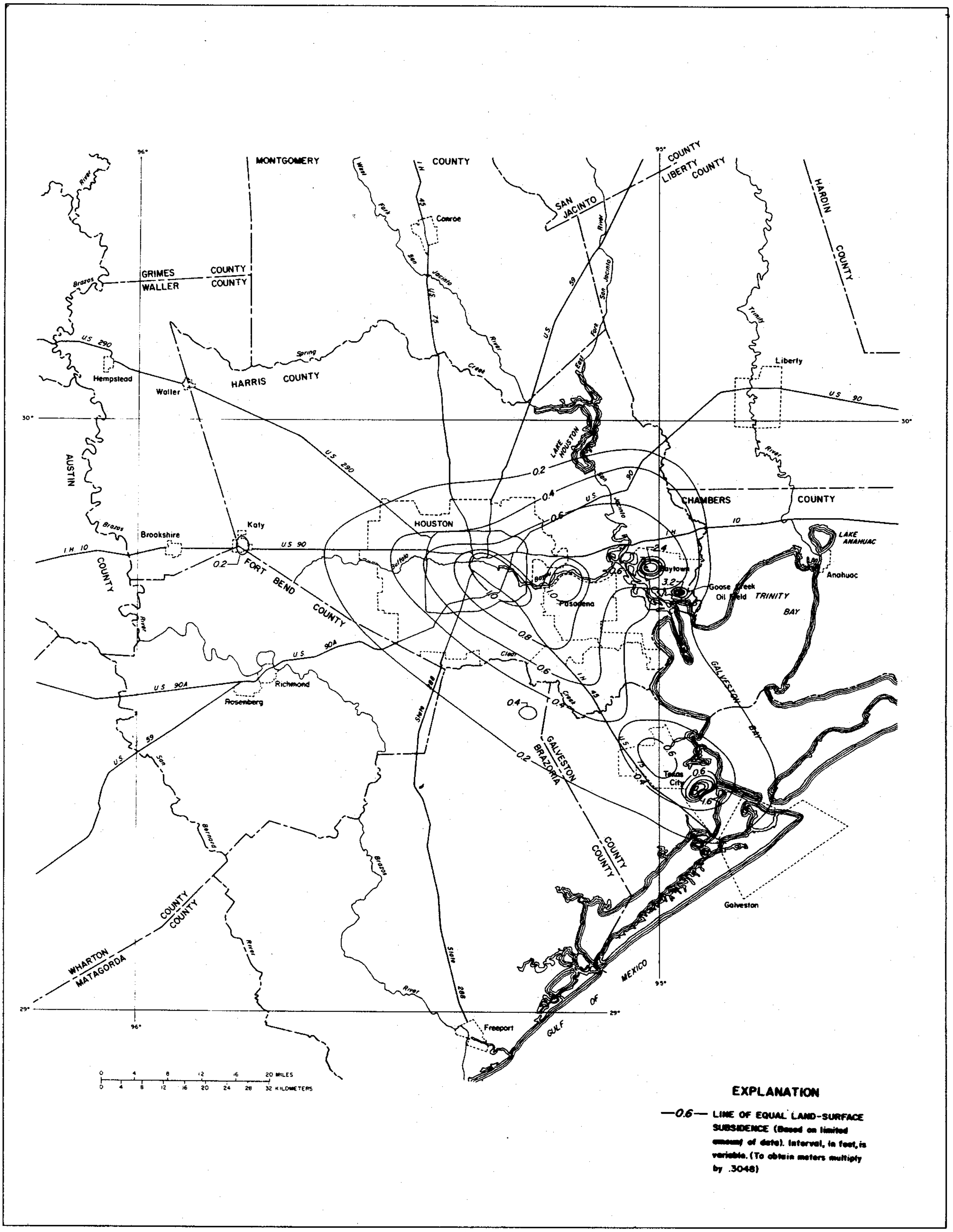


Figure 9.12.6 Approximate subsidence of the land surface, 1906-43.

grained material, and the average stress applied to the system during 1964-73 was estimated to be a change in head of 5.7 metres of water. Subsidence during 1964-73 was 0.18 metre; therefore, the compressibility of the fine-grained material was determined to be:

$$0.18/(151.5\ \text{m})\ (5.7\ \text{m}) = 2.1 \times 10^{-4}\text{m}^{-1}.$$

The weighted average compressibility as determined by laboratory consolidation tests of 15 cores from three sites was $3.2 \times 10^{-4}\text{m}^{-1}$. Because the sediments were still undergoing compression, the compressibilities determined at the Seabrook and Texas City sites are minimum estimates of specific storage.

It has been suggested by some investigators that, in addition to inundation of land by tidal waters, some if not all of the numerous existing faults in the Houston-Galveston region are reactivated by man-caused land-surface subsidence. Attempts have been made to relate the fault activity to subsidence, but because of a lack of data the relationships are not clear.

In 1977, a network of measurement stations, about 0.6 kilometre apart, were established along a line about 70 kilometres long from the approximate center of subsidence westward along U.S. Highway 90 to the Harris County boundary. In addition, closely spaced marks for horizontal and vertical control will be established at three active faults. The purpose of this network is to measure horizontal strain associated with subsidence and to relate this strain to movement along the fault planes.

It has also been hypothesized (Kreitler, 1977) that the numerous faults act as partial barriers to ground-water flow and therefore control or "compartmentalize" subsidence; however, the data on artesian-pressure fluctuations in the area do not support this hypothesis.

Most of the damage resulting from subsidence is related to the lowering of land-surface elevations in the vicinity of Galveston Bay and the subsequent inundation by tidal waters. Several roadways have been rebuilt at higher elevations; ferry landings have been rebuilt; and levees have been constructed to reclaim or protect some areas. The cost of the damages resulting from subsidence have been estimated in some areas, but comprehensive studies for the entire region have not been made. Jones and Larson (1975, table 5) estimated the annual cost of subsidence during 1969-74 to be $31,705,040 in 2,448 square kilometres of the area most severely affected by subsidence. In their estimate of costs, Jones and Larson attributed fault-caused structural damage to man-caused subsidence.

One outstanding example of both the social and economic impacts of subsidence is in the Brownwood subdivision on the west side of Baytown. The area of the subdivision has subsided more than 2.4 metres since 1915, and some homes in the subdivision are permanently flooded by water from the bay. The U.S. Army Corps of Engineers has recommended that the entire subdivision, consisting of 448 homes occupied by 1,550 residents, be purchased by the Federal Government and the inhabitants be relocated at a cost of about $40 million.

9.12.5 FUTURE SUBSIDENCE IN THE REGION

Ground-water pumping in the Houston-Galveston region increased at a rate of about 6 per cent per year before about 1967. Since then, ground-water pumping has been at an almost stable rate, possibly because of recirculation of cooling water by industry and increased use of surface water from Lake Houston. As a result, the rate of decline in water levels has decreased significantly in many parts of the region since the early 1970’s. Records from borehole extensometers (compaction monitors) indicate a decreased rate of subsidence at seven sites scattered throughout the region. The decrease in the rate of subsidence, which began about September 1976, strongly suggests a reflection of the decreased rate of water-level decline.

Water from a new source, Lake Livingston on the Trinity River, about 97 kilometres east of Houston has become available recently; and voluntary commitments to purchase this water have been made by all major industries using ground water in the southern half of Harris County. As a result, ground-water pumping will decrease by about 3.1 m^3/s in the area of maximum artesian-pressure decline and subsidence. An analog-model study of the effects of the decreased pumping suggests a maximum water-level recovery of about 30 metres in the center of the bowl of subsidence. Data are not sufficient to determine the head recovery necessary to stop subsidence, but the rate of subsidence is expected to decrease substantially. By June 1977, the increased use of surface water had caused a decrease in ground-water pumping of about 0.8 m^3/s. Locally, the recovery in artesian head has been as much as 18 metres.

The Harris-Galveston Coastal Subsidence District was created by the Texas Legislature in 1975 to "provide for the regulation of the withdrawal of ground water within the boundaries of the District for the purpose of ending subsidence which contributes to or precipitates flooding, inundation, or overflow of any area within the District, including without limitation rising waters resulting from storms or hurricanes." The District plans to monitor the stress-strain relationships with additional compaction monitors and piezometers designed for installation prior to the expected voluntary decrease in ground-water pumping. The data collected will be the basis for controlling pumping by the issuance of well permits.

The constitutionality of the subsidence district has been tested in a Texas District Court in a suit titled Sammy Beckendorf, et al., versus the Harris-Galveston Coastal Subsidence District. The District prevailed, but Beckendorf, et al., have appealed the ruling of the court. Other lawsuits against the District have been filed but have not come to trial. Two other lawsuits (Smith-Southwest Industries, et al., versus Friendswood Development Company, et al.; and E. R. Brown, et al., versus Exxon Company, U.S.A., et al.), whereby the plaintiffs seek to establish blame and recover damages from subsidence, have not come to trial.

9.12.6 SELECTED REFERENCES

AMERICAN OIL COMPANY. 1958. Refinery ground subsidence: Plant Engineering Dept., Texas City, Texas, 58p.

GABRYSCH, R. K. 1969. Land-surface subsidence in the Houston-Galveston region, Texas: Internat. Symp. on Land Subsidence, Tokyo, Japan, proc., IASH Pub. no. 88, v. 1, p. 43-54.

GABRYSCH, R. K., and BONNET, C. W. 1975a. Land-surface subsidence in the Houston-Galveston region, Texas: Texas Water Devel. Board Rept. 188, 19 p.

___________. 1975b. Land-surface subsidence at Seabrook, Texas: U.S. Geol. Survey Water Resources Inv. 76-31, 53 p.

JONES, L. L., and LARSON, J. 1975. Economic effects of land subsidence due to excessive ground water withdrawal in the Texas Gulf Coast area: Texas Water Resources Inst., Texas A&M Univ., TR-67, 33 p.

JORGENSEN, D. G. 1975. Analog-model studies of ground-water hydrology in the Houston district, Texas: Texas Water Devel. Board Rept. 190, 84 p.

KREITLER, C. W. 1977. Fault control of subsidence, Houston, Texas: Ground Water, v. 15, no. 3, p. 203-214.

PRATT, W. E., and JOHNSON, D. W. 1926. Local subsidence of the Goose Creek Oil Field: Jour. Geology, v. XXXIV, no. 7, pt. 1, p. 578-590.

WINSLOW, A. G., and DOYEL, W. W. 1954. Land-surface subsidence and its relation to the withdrawal of ground water in the Houston-Galveston region, Texas: Econ. Geology, v. 49, no. 4, p. 413-422.

WINSLOW, A. G., and WOOD, L. A. 1959. Relation of land subsidence to ground-water withdrawals in the upper Gulf coast region, Texas: Mining Eng., Oct., p. 1030-1034; Am. Inst. Mining Metall. Petroleum Engineers Trans., v. 214.

Case History No. 9.13. San Joaquin Valley, California, U.S.A., by Joseph F. Poland, U.S. Geological Survey, Sacramento, California, and Ben E. Lofgren, Woodward-Clyde Consultants, San Francisco, California

9.13.1 INTRODUCTION

The principal areas of land subsidence due to ground-water withdrawal in California are in the San Joaquin Valley and the Santa Clara Valley (Figure 9.13.1). A case history for the Santa Clara Valley is included elsewhere in this publication. In the San Joaquin Valley, subsidence due to ground-water withdrawal occurs in three areas--the Los Banos-Kettleman City area on the central west side, the Tulare-Wasco area on the southeast border, and the Arvin-Maricopa area at the south end (Figure 9.13.1).

Since 1956, the U.S. Geological Survey has carried on two investigative programmes in the San Joaquin Valley. One, a study of land subsidence, was carried on in cooperation with the California Department of Water Resources. The other, a federally financed research project on the mechanics of aquifer systems, had two major goals: to determine the principles controlling the deformation of aquifer systems in response to change in grain-to-grain load, and to appraise the change in storage characteristics as the systems compact under increased effective stress. During the 20 years of research under these two projects, many of the causes and effects of land subsidence have been documented. Sixteen of the principal reports have been pub-

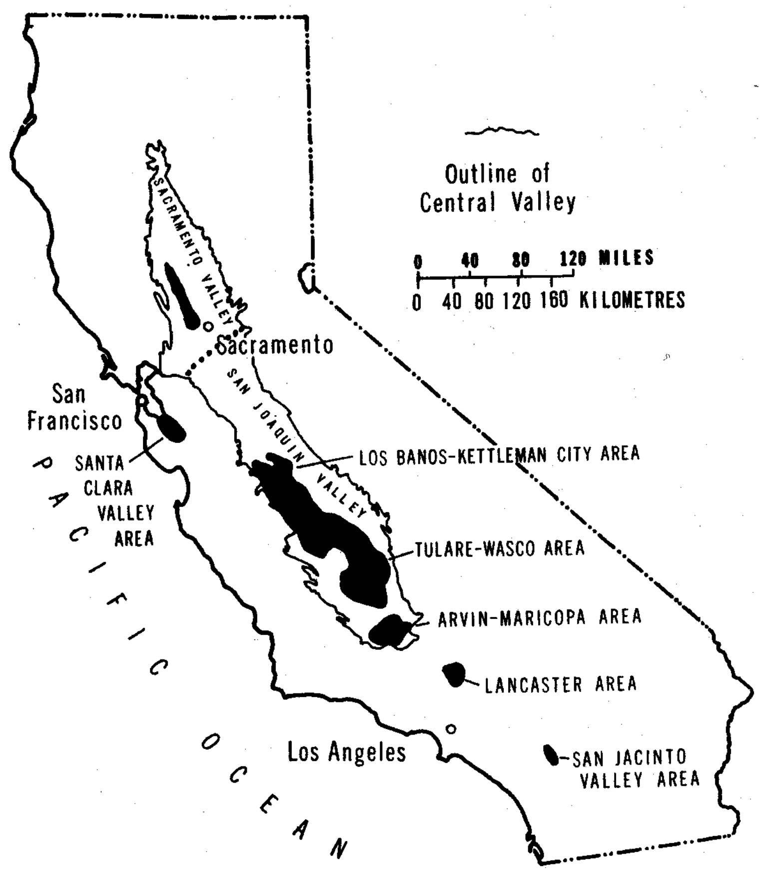


Figure 9.13.1 Areas of land subsidence in California due to ground-water withdrawal.

lished as professional papers of the Geological Survey, the subsidence reports in the Professional Paper 437 series, and the mechanics of aquifer systems papers in the Professional Paper 497 Series. The following case history concerning subsidence in the San Joaquin Valley is taken chiefly from the summary report by Poland, Lofgren, Ireland, and Pugh (U.S. Geol. Survey Prof. Paper 437-H, 1975). More detailed information is available in published reports on the three areas.

9.13.2 GEOLOGY

The San Joaquin Valley includes the southern two-thirds of the Central Valley, an area of 26,000 km^2. It is a broad structural downwarp bordered on the east by the granitic complex of the Sierra Nevada and on the west by the complexly folded and faulted Coast Ranges. The top of the basement complex of the Sierra Nevada block dips gently westward beneath the valley. Late Cenozoic continental deposits form the floor of the valley and attain maximum thickness of 5,000 m near the south edge.

The continental deposits are chiefly of fluvial origin but contain several extensive interbeds of lacustrine origin. The fluvial deposits consist of lenticular bodies of sand and gravel, sand, and silt deposited in stream channels, and sheetlike bodies of silt and clay laid down on flood plains by slow-moving overflow waters.

Along the east side of the valley the sediments deposited by the major streams issuing from the Sierra Nevada--from the Merced River south to the Kings River--have formed a series of coalescing alluvial fans, characterized by a mass of coarse permeable deposits, largely tongues and lenses of sand and gravel, that extend to and beyond the topographic trough of the valley.

In more than half of the San Joaquin Valley area that lies south of Los Banos, the deposits containing freshwater can be divided into: (1) an upper unit of clay, silt, sand, and gravel chiefly alluvial-fan and flood-plain deposits of heterogeneous character, (2) a middle unit consisting of a relatively impermeable diatomaceous lacustrine clay; and (3) a lower unit of clay, silt, sand, and some gravel, in part lacustrine deposits, that extends down to the beds containing saline water. The upper and middle units are Pleistocene age; the lower unit is of Pleistocene and Pliocene age. Together, these three units approximately constitute the Tulare Formation. The middle unit is the Corcoran Clay Member of the Tulare Formation (Miller, Green, and Davis, 1971).

9.13.3 HYDROLOGY

The continental freshwater-bearing deposits can be subdivided into two principal hydrologic units. The upper unit, a semiconfined aquifer system with a water table, also termed the "upper water-bearing zone," extends from the land surface to the top of the Corcoran Clay Member at a depth ranging from 0 to 275 m below the land surface. The lower unit, a confined aquifer system, also termed the "lower water-bearing zone," extends from the base of the Corcoran Clay Member down to the saline water body. The thickness of this confined system ranges from 60 to more than 600 m. The Corcoran Clay Member, which ranges in thickness from 0 to 40 m, is the principal confining bed beneath at least 13,000 km^2 of the San Joaquin Valley. The dotted line in Figure 9.13.2 defines the general extent of this principal confining bed in the valley. South of Bakersfield the confining bed has been designated the E clay by Croft (1972).

Yearly extraction of ground water for irrigation in the San Joaquin Valley increased slowly from 2,500 hm^3 in the middle 1920’s to 3,700 hm^3 in 1940. Then, during World War II and the following two decades, the rate of extraction increased more than threefold to furnish irrigation water to rapidly expanding agricultural demands. By 1966, pumpage of ground water was 12,000 hm^3 per year.

This very large withdrawal caused substantial overdraft on the central west side and in much of the southern part of the valley, mostly within the shaded area of Figure 9.13.2. The withdrawal in these overdraft areas in the 1950’s and early sixties was at least 5,000 hm^3 per year. During the period of long-continued excessive withdrawal, the head (potentiometric surface) in the confined aquifer system between Los Banos and Wasco was drawn down 60 to 180 M. South of Bakersfield the head decline was more than 100 m.

Importation of surface water to these areas of serious overdraft began in 1950 when water from the San Joaquin River was brought south through the Friant-Kern Canal, which extends to the Kern River (Figure 9.13.2).About 80 per cent of the average annual deliveries of 1,250

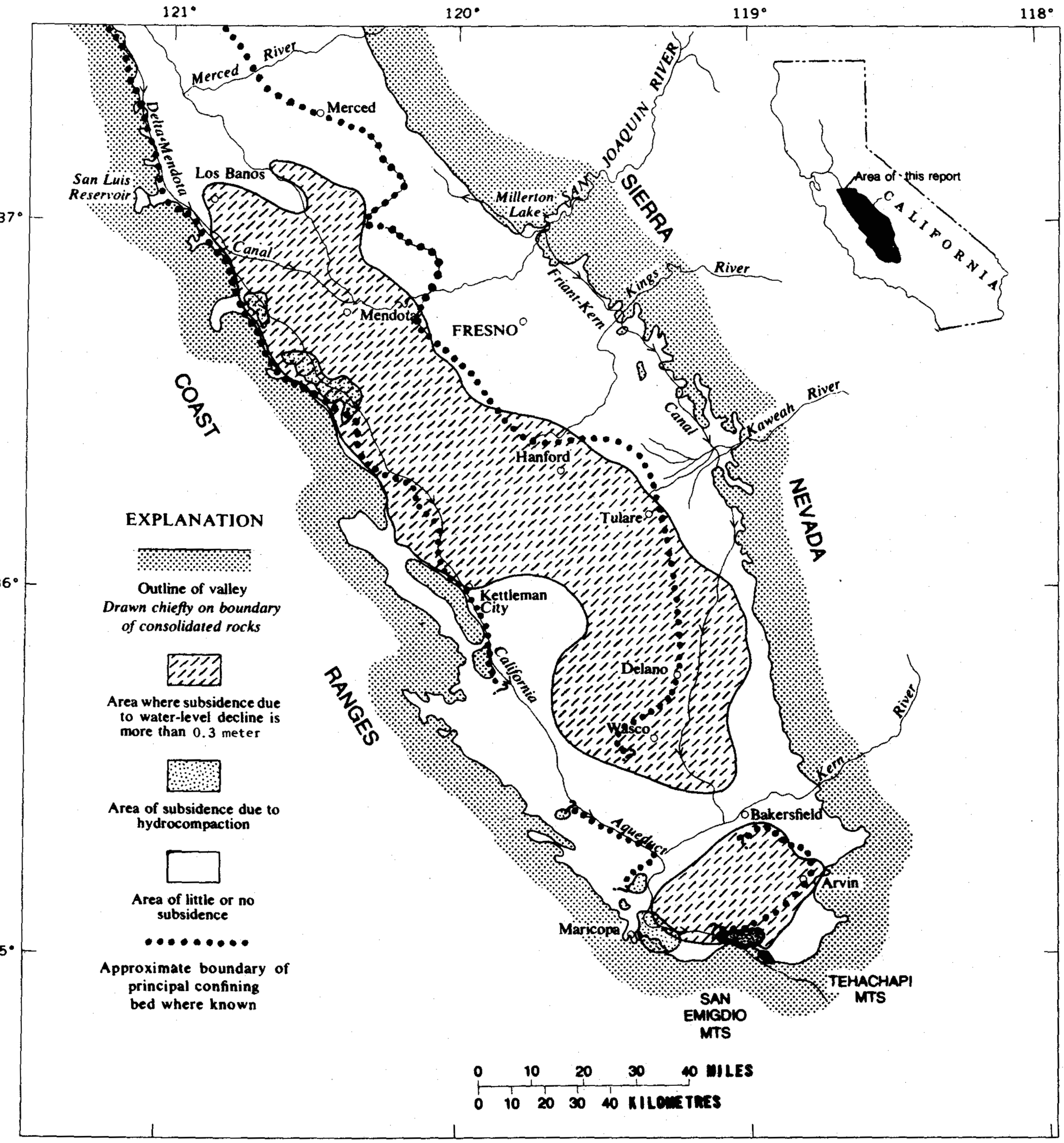


Figure 9.13.2 Pertinent geographic features of central and southern San Joaquin Valley and areas affected by subsidence.

hm^3 of water from this canal is sold to irrigation districts south of the Kaweah River, mostly in the Tulare-Wasco subsidence area.

Large surface-water imports from the northern part of the state to overdrawn areas on the west side and south end of the valley are being supplied through the California Aqueduct (Figure 9.13.2). The joint-use segment of the aqueduct between Los Banos and Kettleman City serves the San Luis project area of the U.S. Bureau of Reclamation and transports State-owned water south of Kettleman City. Surface-water deliveries to the San Luis project area increased from 250 hm^3 in 1968, the first year, to about 1,360 hm^3 in 1974. Also, by 1973 the California Aqueduct delivered 860 hm^3 to the southern part of the San Joaquin Valley (south of Kettleman City), and is scheduled eventually to supply 1,670 hm^3 under long-term contracts.

As a result of these large surface-water imports, the rate of ground-water withdrawal decreased sharply and the decline of artesian head was reversed in most of the areas of overdraft. By the early 1970’s many hundreds of irrigation wells were unused, artesian heads were recovering at a rapid rate, and rates of subsidence were sharply reduced.

9.13.4 LAND SUBSIDENCE

Subsidence in the San Joaquin Valley is of three types. In descending order of importance these are (1) subsidence due to the compaction of aquifer systems caused by the excessive withdrawal of ground water; (2) subsidence due to the compaction of moisture-deficient deposits when water is first applied--a process known as hydrocompaction; and (3) local subsidence caused by the extraction of fluids from several oil fields.

Oil-field subsidence is due to the same process as subsidence caused by excessive pumping of ground water--a lowering of fluid level and consequent increase of effective stress on the sediments within and adjacent to the producing beds. However, measured oil-field subsidence in the San Joaquin Valley, which has been discussed briefly by Lofgren (1975), is less than 0.6 m at the few oil fields where periodic releveling has defined its magnitude. This type of subsidence has not created any problems in the valley.

Hydrocompactible deposits occur locally on the west and south flanks of the valley (see Figure 9.13.2). These are near-surface alluvial-fan deposits, largely mud flows, still above the water table. They have been moisture deficient ever since deposition, chiefly because of the low rainfall in the area. When water is first applied, the clay bond is weakened and the deposits compact. Subsidence of 1.5 to 3 m is common and locally it exceeds 4.5 m (Lofgren, 1960; Bull, 1964). The California Aqueduct (Figure 9.13-2) passes through at least 65 km of deposits susceptible to hydrocompaction, and precompaction by prolonged wetting of the aqueduct alinement was carried on for about one year prior to the placing of the concrete lining.

Subsidence due to the compaction of aquifer systems in response to excessive decline of water levels had affected about 13,500 km^2 of the San Joaquin Valley by 1970. Figure 9.13.3 depicts the distribution and magnitude of subsidence exceeding 1 foot (0.3 m) that had occurred by 1970--affecting an area of 11,100 km^2. Three centers of subsidence are conspicuous on this map. The most conspicuous is the long narrow trough west of Fresno that extends 140 km from Los Banos to Kettleman City (referred to subsequently as the west-side area). Maximum subsidence in this area to 1977 was 29.5 feet (9.0 m), 16 km west of Mendota. The second center, between Tulare and Wasco, is defined by two closed 12-foot (3.7-m) lines of equal subsidence, 32 and 48 km south of Tulare, respectively. Maximum subsidence to 1970 was 4.3 m, at a benchmark 32 km south of Tulare. The third center, 32 km south of Bakersfield, has subsided a maximum of 9.2 feet (2-8 m), mostly since World War II. Note that the California Aqueduct was constructed through the full 140 km of the subsidence trough extending from Los Banos to Kettleman City, as well as through the southwestern edge of the subsidence bowl south of Bakersfield.

The cumulative volume of subsidence in the San Joaquin Valley (Figure 9.13.4) grew slowly until the end of World War II. With the great increase in ground-water extraction in the 1940’s and 1950’s, however, the cumulative volume of subsidence soared to 12,350 hm^3 by 1960, and reached 19,250 hm^3 by 1970. This very large volume is equal to one-half the initial storage capacity of Lake Mead or to the total discharge from all water wells in the San Joaquin Valley for 1.5 years at the 1966 rate. This volume of subsidence represents water of compaction derived almost wholly from compaction of the fine-grained highly compressible clayey interbeds (aquitards), in response to the increase in effective stress as artesian head in the confined system declined. The volume of subsidence for any interval of leveling control was obtained by planimetry of the subsidence map for that period. All leveling data used in the preparation of subsidence maps and graphs were by the National Geodetic Survey (formerly the Coast and Geodetic Survey).

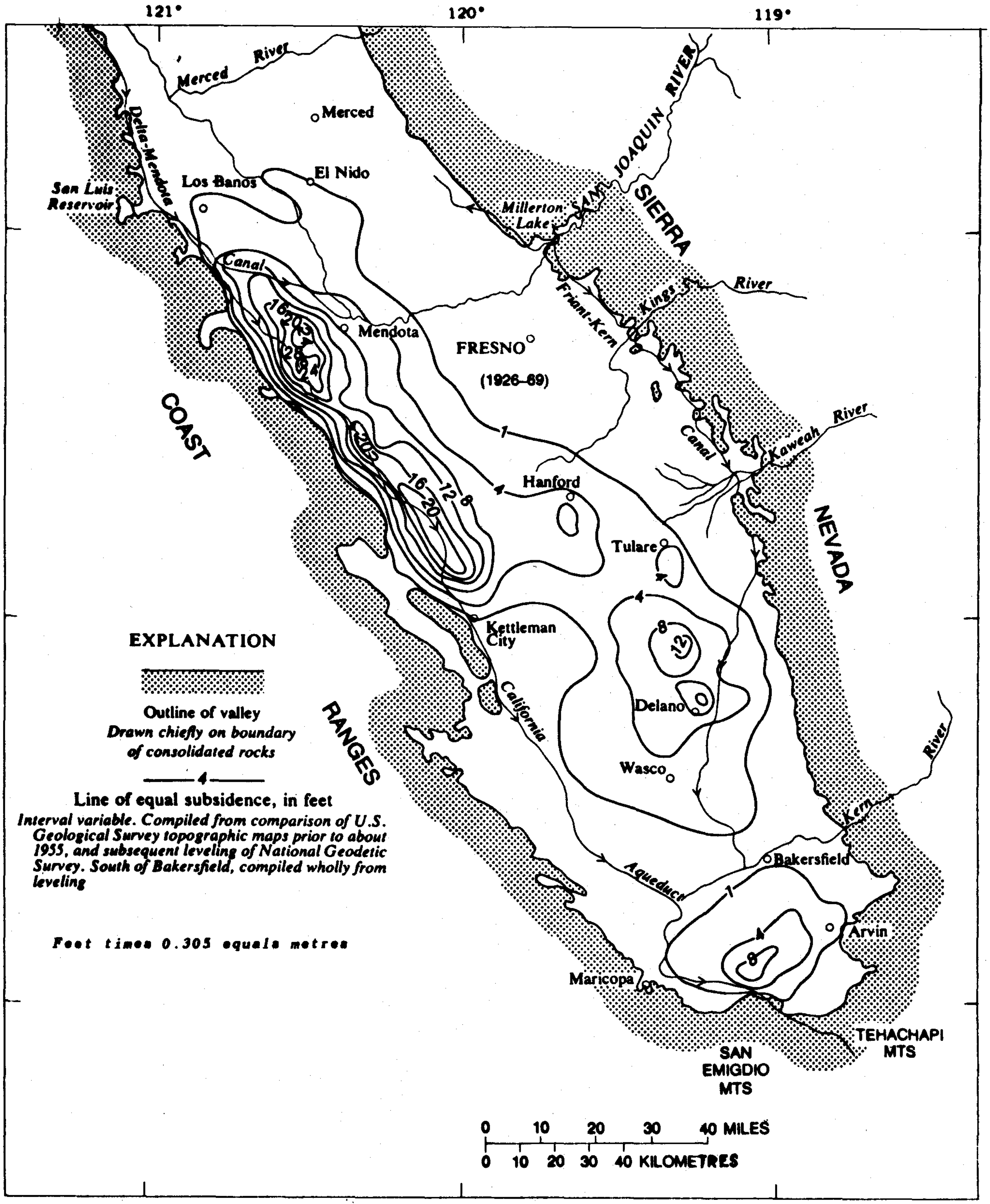


Figure 9.13.3 Land subsidence in the San Joaquin Valley, California, 1926-1970.

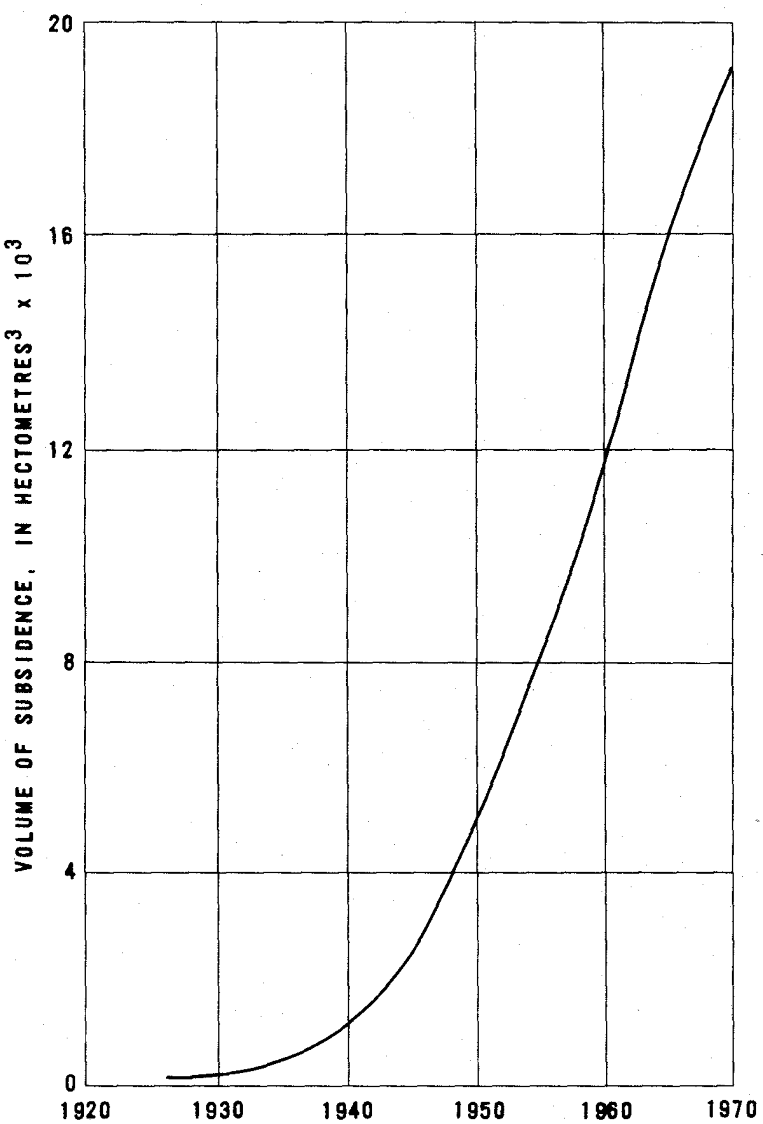


Figure 9.13.4 Cumulative volume of subsidence, San Joaquin Valley, California, 1926-70.

The west-side area has experienced the most severe subsidence (Figure 9.13.5); therefore several illustrations will be presented to show the relation between water-level change (stress change) and compaction or subsidence in that area. Subsidence has affected about 6,200 km^2 and the volume of subsidence, 1926-69, was about 11,850 hm^3, about two-thirds of the valley total. The cumulative volume of ground-water pumpage in the west-side area through March 1969 is estimated as 35,200 hm^3 (Figure 9.13.6). This cumulative pumpage has been plotted with cumulative subsidence at a scale of 3 to 1. The correlation is remarkably consistent, indicating that throughout the 43 years since subsidence began (1926 into 1969), about one-third of the water pumped has been water of compaction derived from the permanent reduction of pore space in the fine-grained compressible aquitards.

Figure 9.13.7 illustrates the relation of subsidence to artesian-head change since 1943 at a site 16 km southwest of Mendota. Bench mark S661, located within the 28-foot (8.5-m) line of equal subsidence in Figure 9.13.5, subsided 8 m from 1943 to 1969, in response to a water-level decline of nearly 125 m as measured in nearby wells. The rate of subsidence at this site reached a maximum of 0.54 m per year between 1953 and 1955 but decreased to 0.04 m per year between 1972 and 1975, due chiefly to substantial recovery of artesian head. Static water level began to recover in 1969 and by 1977 had risen 73 m above the 1968 summer low level because of

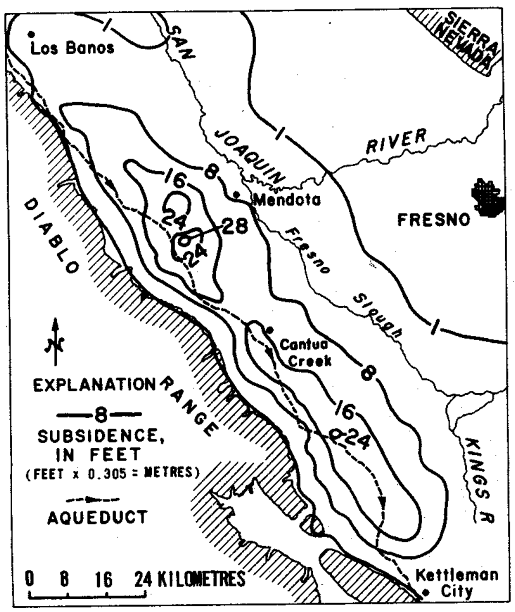


Figure 9.13.5 Land subsidence in the Los Banos-Kettleman City area, California, 1926-69.

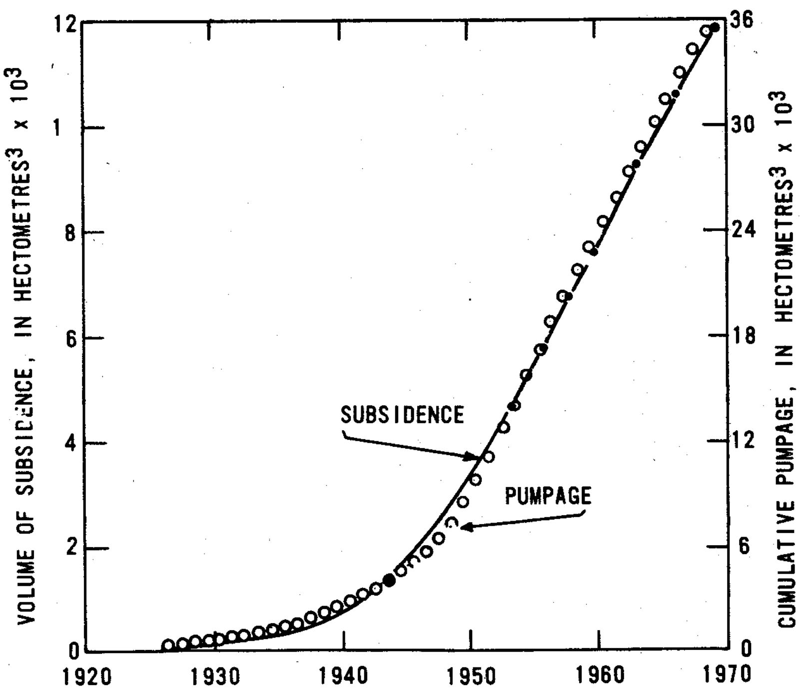


Figure 9.13.6 Cumulative volume of subsidence and pumpage, Los Banos-Kettleman City area, California. Points on subsidence curve indicate times of leveling control.

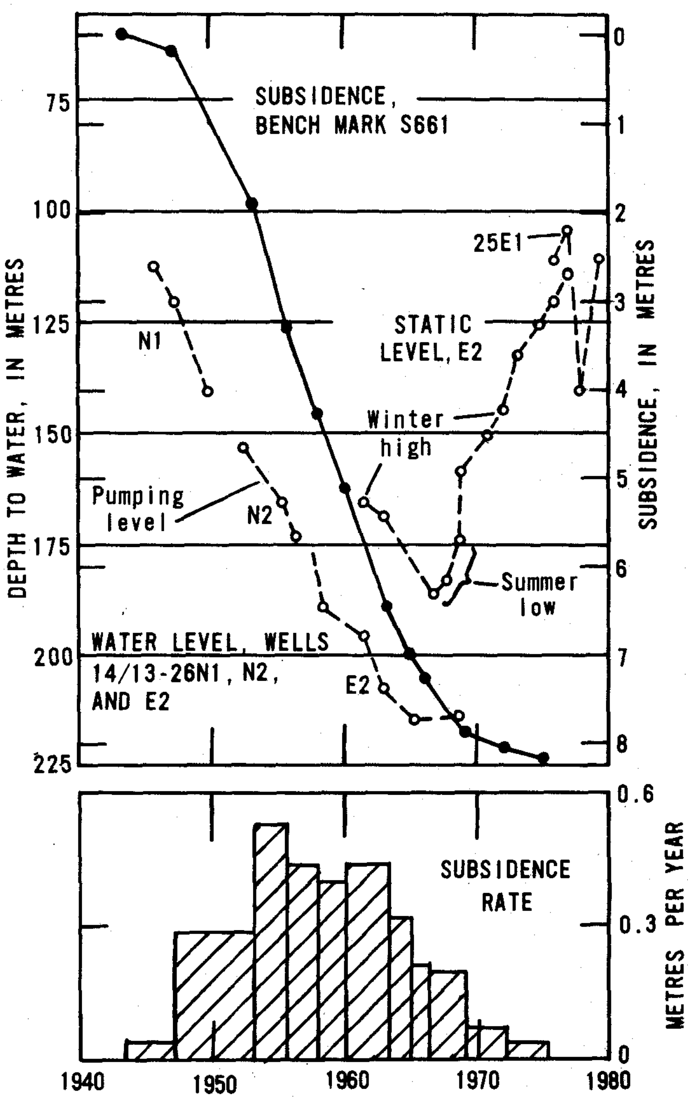


Figure 9.13.7 Subsidence and artesian-head change 16 kilometres southwest of Mendota.

the large imports of surface water through the California Aqueduct and the consequent reduction in pumpage.

If two or more extensometers (compaction recorders) are installed in adjacent wells of different depths, the records from the multiple-depth installation will indicate the magnitude and rate of compaction (or expansion), not only within the total depths of individual wells but also for the depth intervals between well bottoms. Figure 9.13.8 shows the record of compaction from 1958 through 1971 in three adjacent extensometer wells in the west-side area. The site is adjacent to the California Aqueduct at the north end of the southern 16-foot (4.9-m) line of equal subsidence in Figure 9.13.5. The wells are 152, 213, and 610 m deep. Measured compaction in the 13 years was about 0.42 m, 0.97 m, and 3.40 m, respectively. Thus the compaction in the 213-610-m depth interval was 2.43 m. The dashed line represents subsidence of a surface bench mark at this site as determined by repeated leveling from stable bench marks (black dotes on the dashed line show dates of leveling). In the early 1960’s the rate of compaction measured in the 610-m well (Nl) was nearly equal to the rate of subsidence. Subsequently the rate of compaction of deposits below the 610-m depth gradually increased, due to increased pumping and declining pore pressures in deeper wells drilled in the 1960’s. This deeper compaction caused the departure of the subsidence plot from the compaction plot for well Nl.

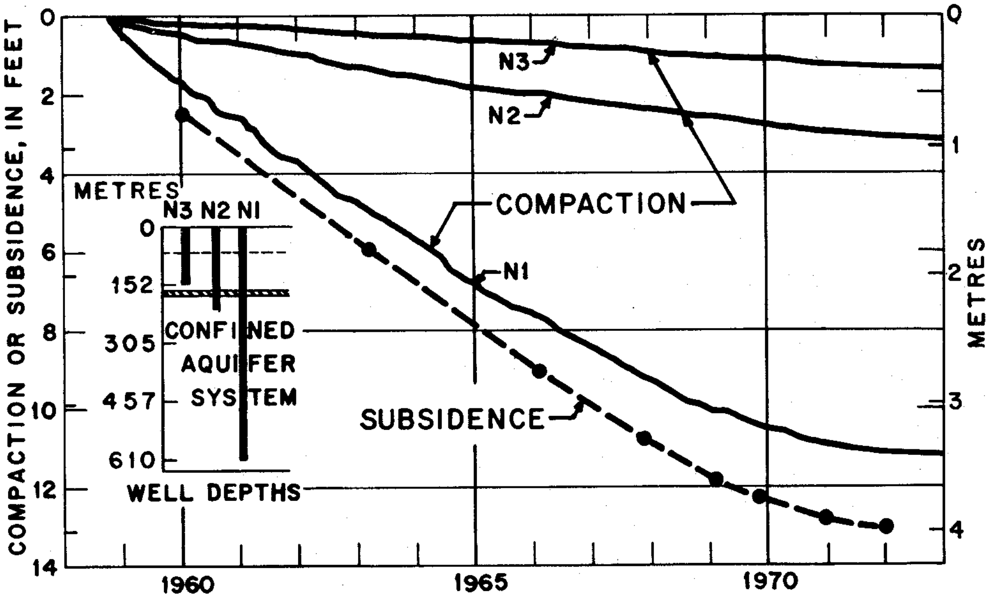


Figure 9.13.8 Compaction and subsidence at Cantua site, 65 kilometres southwest of Fresno, California.

9.13.5 COMPRESSIBILITY AND STORAGE PARAMETERS

In the late 1950's, as one phase of the research on land subsidence and compaction of aquifer sytems, the Geological Survey drilled four core holes in the Los Banos-Kettleman City (west side) area ranging in depth from 305 to 670 m, and two core holes in the Tulare-Wasco area to depths of 232 and 670 m. Cores were tested in the Hydrologic Laboratory for particle-size distribution, specific gravity of solids, dry unit weight, porosity and void ratio, hydraulic conductivity (normal and parallel to bedding) and Atterberg limits. Results have been published (Johnson, Moston, and Morris, 1968). X-ray diffraction studies of 85 samples from the westside cores and 26 samples from the Tulare-Wasco cores indicated that about 70 per cent of the clay-mineral assemblage in these deposits of Pliocene to Holocene age consists of montmorillonite (Meade, 1967, Tables 11-13).

Laboratory consolidation tests were made by the Bureau of Reclamation on 60 fine-grained cores from the four core holes in the west-side area and on 22 fine-grained cores from the two core holes in the Tulare-Wasco area. Parameters tested included the compression index, C_c, a measure of the compressibility of the sample, and the coefficient of consolidation, C_v, a measure of the time-rate of consolidation. Results have been published (Johnson and others, 1968, Tables 8 and 9). The range of the compression index, C_c, was much wider than for samples from the Santa Clara Valley: In the Los Banos-Kettleman City area the range was 0.09 to 1.13; in the Tulare-Wasco area it was 0.25 to 1.53. However, all values greater than 0.47 were either from lacustrine clays or from the fine-grained marine siltstone in the Richgrove core hole 12 km east of Delano.

The subsidence volume represents pore-space reduction occurring chiefly in the fine-grained compressible aquitards. In the west-side area, the volume of subsidence from 1926 to 1969 was about 11,850 hm^3, distributed over 6,200 km^2. If the subsidence had been distributed evenly over this area, it would average about 1.9 m. Roughly half the sediments in the principal aquifer system are fine-grained compressible aquitards. Assuming the average composite thickness of the compacting aquitard is 150 m and the average initial porosity is 40 per cent, a mean subsidence of 1.9 m would represent an average reduction in porosity of roughly 1 per cent in these fine-grained beds (from 40 to 39.2 per cent) - In the small area where the maximum 8.8 m of subsidence has occurred, the local reduction in pore space of aquitards would be roughly 4 per cent (from 40 to 36.3 per cent).

The subsidence/head-decline ratio (specific subsidence) is the ratio between land subsidence and the hydraulic head decline in the coarse-grained permeable beds of the compacting aquifer system, for a common time interval. It can be expressed as the change in thickness per unit change in effective stress (Δb/Δp′). This ratio is useful as a first approximation of compressibility; it is also useful for predicting a lower limit for the magnitude of subsidence in response to a step increase in virgin stress (stress greater than past maximum). If pore pressures in the fine-grained aquitards were eventually to reach equilibrium with those in adjacent aquifers after a step increase beyond preconsolidation stress, compaction would cease and the subsidence/head-decline ratio would indicate the true virgin compressibility of the system.

In the west-side area during the period 1943-60 the decline of artesian head for the lower zone ranged from 30 to 120 m (Bull and Poland, 1975, Figure 25), resulting in subsidence in the 17-year period of 0.3 to 4.9 m (Bull, 1975, Figure 10). The subsidence/head-decline ratio for that same period ranged areally from 0.01 to 0.08 (Bull and Poland, 1975, Figure 32). In other words, the head decline required to produce 1 metre of subsidence ranged from 100 to 12 m. A subsidence/head-decline ratio can be derived from Figure 9.13.7 for the period 1947 to 1965. In the 18 years, bench mark S661 subsided 6.86 m, and the pumping level in nearby wells declined 95 m. Thus, for that time span the ratio at that site equaled 0.07.

In the Tulare-Wasco area, the subsidence/head-decline ratio ranged from 0.01 to 0.06 (Lofgren and Klausing, 1969, Figure 69). In the Arvin-Maricopa area, the subsidence/head-decline ratio for the 8-year period 1957-65 ranged from 0.01 to 0.05 (Lofgren, 1975, Plate 5B).

Areal variation in the subsidence/head-decline ratio can be produced by one or more of several factors. These include variation in the individual, and gross aggregate thickness of the compacting aquitards and variation in compressibility and vertical hydraulic conductivity of the individual aquitards. Such areal variation in compressibility and hydraulic conductivity can be caused by variation in grain size, in depth of compacting beds (in overburden load), in geologic formation tapped, in existing preconsolidation stress, in clay-mineral assemblage, and in other diagenetic effects. Furthermore, because the subsidence values available for computing the ratio seldom represent ultimate subsidence for a designated change in stress within aquifers, time is an important factor. According to soil-consolidation theory, the time required for an aquitard that is draining to adjacent aquifers to reach a specified percentage of ultimate compaction varies directly as (1) the square of the draining thickness and (2) the ratio of compressibility to vertical hydraulic conductivity. Variation in the thicknesses of the many vertical-draining aquitards encountered at any selected site obviously makes that site unique in its rate of compaction, even if all other factors are equal. In the depth interval 214 to 610 m at west-side well 16/15-34N1, for example, interpretation of the microlog defined 60 aquitards ranging in thickness from 0.6 m to 15 m and averaging 4.5 m.

One other factor directly affecting the accuracy of the subsidence/head-decline ratio is the appropriateness or the accuracy of the change-in-stress value used. Even in a ground-water basin containing a single confined aquifer system it is difficult to obtain measurements of head change that truly represent the average stress change on aquitard boundaries within the full well-depth interval experiencing a measured compaction or subsidence. Thus, observation wells used to derive stress-change values, whether for subsidence/head-change ratios or for stress-strain plots, should be selected or constructed very carefully.

Bull (1975, p. 49-82) made a study of geologic factors that caused areal differences in specific unit compaction in the Los Banos-Kettleman City area for the period 1943-60. The factors included total applied stress, lithofacies, and source and mode of deposition.

Field measurements of compaction or expansion of sediments and the correlative change in fluid pressure(s) can be utilized to construct stress-strain curves and to derive storage and compressibility parameters. One example (Figure 9.13.9) is for a well 176 m deep on the west

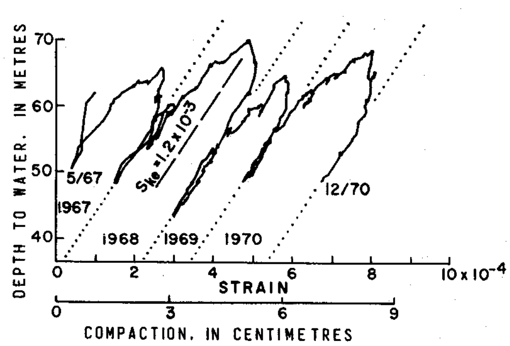


Figure 9.13.9 Stress change, compaction, and strain for a well in western Fresno County, California.

side of the valley. Depth to water is plotted increasing upward (increasing stress). Change in depth to water represents change in effective stress in the aquifers in the confined aquifer system (upper zone) that is 106 m thick. Along the abscissa the lower scale is the measured compaction and the upper scale is the strain (measured compaction/compacting thickness). The yearly fluctuation of water level caused by the seasonal irrigation demand and the permanent compaction that occurs each summer during the heavy pumping season when the depth to water is greatest produce a series of stress-strain loops. The lower parts of the descending segments of the annual loops for the three winters 1967-68 to 1969-70 are approximately parallel straight lines, indicating that the response is essentially elastic in both aquifers and aquitards when the depth to water is less than about 55 m. The heavy dashed line in the 1968 loop represents the average slope of the segments in the elastic range of stress. The reciprocal of the slope of the line is the component of the storage coefficient due to deformation of the aquifer-system skeleton, S_{ke}, and equals 1.2×10^{-3}. The component of average specific storage due to elastic deformation, S_{ske}, equals $S_{ke}/106$ m = $1.1 \times 10^{-5} m^{-1}$. The average elastic compressibility of the aquifer system skeleton, α_{ke}, is S_{ske}/γ_w,; if γ_w, (the unit weight of water) equals 1, the numerical values Of α_{ke} and S_{ske} are identical.

For increase in effective stress in the range of loading exceeding preconsolidation stress, the "virgin" compaction of aquitards is chiefly inelastic--nonrecoverable upon decrease in stress. At Pixley, 27 km south of Tulare (Figure 9.13.3), compaction and change in stress for a confined aquifer system 108-231 m below land surface has been measured since 1958. Riley (1969) showed from a stress-strain plot that the mean virgin compressibility of the aquitards (aggregate thickness 75 m) in this confined aquifer segment 123 metres thick was $7.5 \times l0^{-4} m^{-7}$ and the mean elastic compressibility of the aquifer system was $9.3 \times 10^{-6} m^{-1}$. Thus, for the aquifer system segment 123 metres thick at this site, the mean virgin compressibility of the aquitards is about 80 times as large as the mean elastic compressibility of the confined system.

Figure 9.13.10 shows a generalized plot of water level for the confined aquifer system 32 km south of Mendota (Figure 9.13.5) from 1905 to 1964 and the seasonal high and low in observation well 16/15-34N4 for 1961-77. This well taps the confined system. The regional water level declined about 120 m from 1905 to 1960 and the rate of decline accelerated as the groundwater withdrawal increased. By 1960 the seasonal low had declined below the base of the confining clay, producing a water-table condition. Surface-water imports to the west-side area began in 1968. As the imports increased, ground-water pumpage decreased and water levels recovered sharply. From 1968 to 1976 the water level at well 34N4 rose 82 metres. Then, during 1977, the second of two severe drought years, the imports decreased to 370 hm^3 and pumping draft from both old and newly drilled wells soared to about three times the 1976 rate. As a result the water level in well 34N4 fell 50 m in the 8 months to August 1977.

The changing stress as indicated by the hydrograph of well 34N4 and the resulting strain at this site as measured by an extensometer in well 34Nl since 1959 are clearly displayed in Figure 9.13.11. Well 16/15-34N1, 610 m deep, is equipped with an anchored-cable extensometer.

A time plot of cumulative measured compaction at this site was introduced earlier (Figure 9.13.8). in Figure 9.13.11, the measured compaction is plotted as an annual bar graph for comparison with the fluctuations of the water level in well N4. Note that the water level in well N4 began a sharp rise late in 1968 as surface-water imports began. In response to the sharp recovery of water level, compaction decreased rapidly after 1968 but did not cease until 1975. During this period of rising water levels in the coarse-grained aquifers, nonrecoverable virgin compaction continued through 1974 in the central parts of the thicker aquitards, exceeding the continuing small elastic expansion of the preconsolidated aquifers and the thinner aquitards. The water level in well N4 reached a seasonal high of 107 m below land surface in November 1976. Early in February 1977, when water level was 112 m below land surface (only 5 m below the seasonal high), virgin compaction resumed in well Nl. By March 30, 1977, when water level was 15 m below the seasonal high, the maximum compaction rate of the season was attained. The early February water level 112 m below land surface clearly defined the preconsolidation stress in the central segments of the thickest or least permeable aquitards or both. As the drawdown increased, more and more of the slow draining compressible beds began to contribute water of compaction. By yearend, about 12 cm of renewed nonrecoverable compaction had occurred.

During the first period of water-level decline (1905-68 in Figure 9.13.10), water of compaction represented about one-third of the total water pumped from west-side wells (Figure 9.13.6). By 1968, many of the aquitards were preconsolidated nearly to the 1968 stress level. Early in the second period of water-level decline (in 1977), the response of the preconsolidated sediments was chiefly elastic and the contribution of water of compaction was much less than one-third of the total pumpage. Hence the water level fell very rapidly.

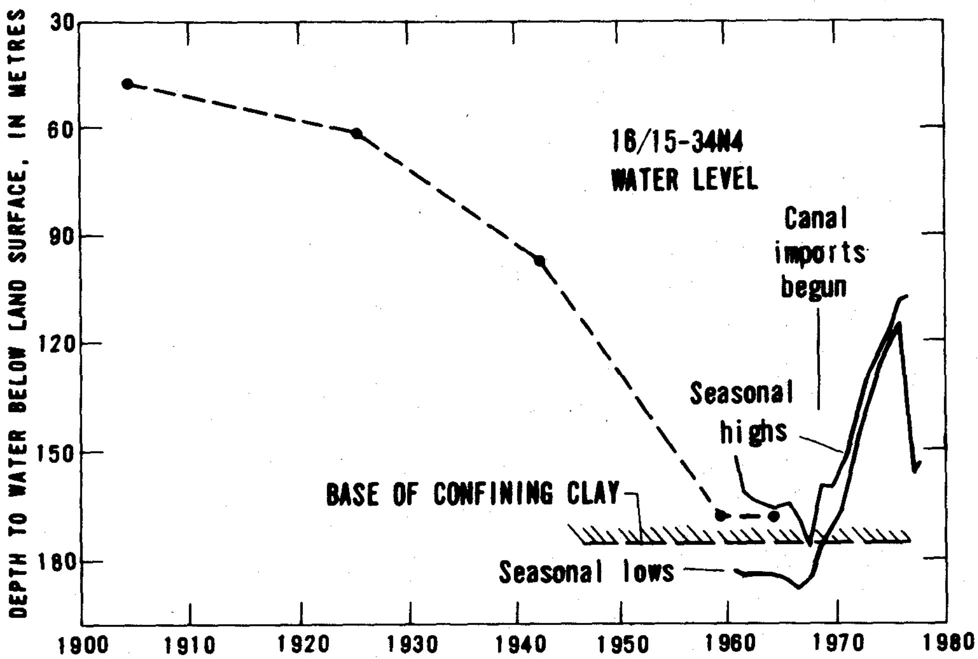


Figure 9.13.10 Long-term trend of water levels near Cantua Creek, and seasonal high and low levels in observation well 16/15-34N4 since 1960. (Modified from Lofgren, 1979,

Figure 9.13.12 displays a similar trend of water-level recovery and reduced compaction, followed by an abrupt head decline and renewed compaction during 1977. Observation well 20/18-6D1 is 25 km north of Kettleman City (Figure 9.13.4) and adjacent to the California Aqueduct. The abrupt head decline of 76 m in 1977 momentarily increased the stress in the aquifers to 1967 levels and stresses in the central parts of the aquitards once again exceeded preconsolidation stresses. In response, virgin compaction of the aquitards exceeded that of 1968. Such stressing and differential compaction in the vicinity of the aqueduct is of concern in sustaining the integrity of such structures. This particular problem appears to be of local extent, however--the intensity of the head decline in well 6Dl is due largely to pumping of a new irrigation well drilled early in 1977 within 60 m of the aqueduct.

9.13.6 ECONOMIC AND SOCIAL IMPACTS

The extensive major subsidence in the San Joaquin Valley has caused several problems. The differential change in elevation of the land surface has created problems in maintenance of water-transport structures, including canals, irrigation and drainage systems, and stream channels. Both the Delta-Mendota Canal and the Friant-Kern Canal (Figure 9.13-3), two major structures of the Central Valley Project of the Bureau of Reclamation, have required remedial work because of subsidence. Also in the period 1926-72, differential subsidence has steepened the channel of the San Joaquin River about 2 m in the 24 km before it reaches the valley trough and has flattened the channel about 2 m in the next 50 km downstream. These changes have affected the transport characteristics of the river and have altered levee requirements.

Another problem common to the subsiding areas in the San Joaquin Valley is the failure of water wells as a result of compressive rupture of casings caused by the compaction of the aquifer systems. In the west-side area, where subsidence has been greatest, many hundreds of deep irrigation wells have required costly repair or replacement. According to Wilson (1968), during 1950-61 approximately 1,200 casing failures were reported in 275 irrigation wells in an area of 1,600 km^2 that spans the region of most intensive subsidence. Well repair and replacement costs attributable to subsidence in the three subsiding areas have been many millions of dollars.

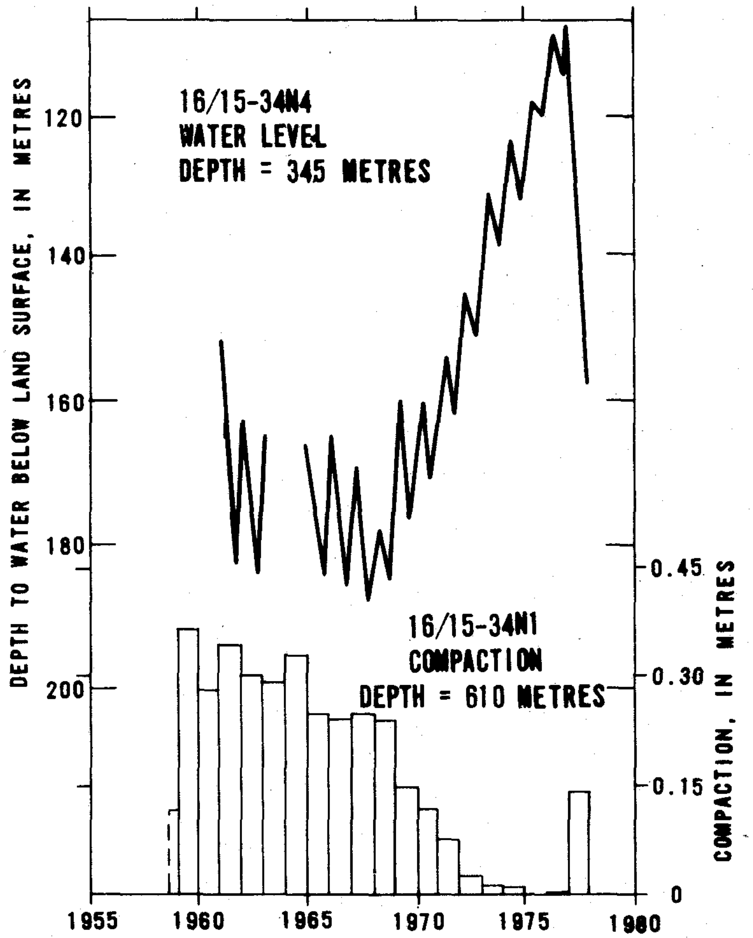


Figure 9.13.11 Seasonal fluctuations of water level in well 16/15-34N4 and measured compaction in observation well 16/15-34N1 near Cantua Creek. (Modified from Lofgren, 1979, Figure 10.)

The need for preconsolidation of deposits susceptible to hydrocompaction substantially increased the construction costs of the California Aqueduct. The aqueduct passes through about 65 km of susceptible deposits. The approximate cost for treatment by prewetting for the reach from Kettleman City to the Tehachapi Mountains has been estimated as $20 million (Lucas and James, 1976, p. 541). Preconsolidation of the susceptible areas between Los Banos and Kettleman City cost an additional estimated $5 million.

The subsidences have increased considerably the number and cost of surveys made by governmental agencies and by private engineering firms to determine the elevations of bench marks or construction sites and to establish grades. In addition, revision of topographic maps has been more frequent and more expensive than in nonsubsiding areas.

9.13.7 LEGAL ASPECTS

So far as known, no legal actions have been taken as a result of the subsidence.

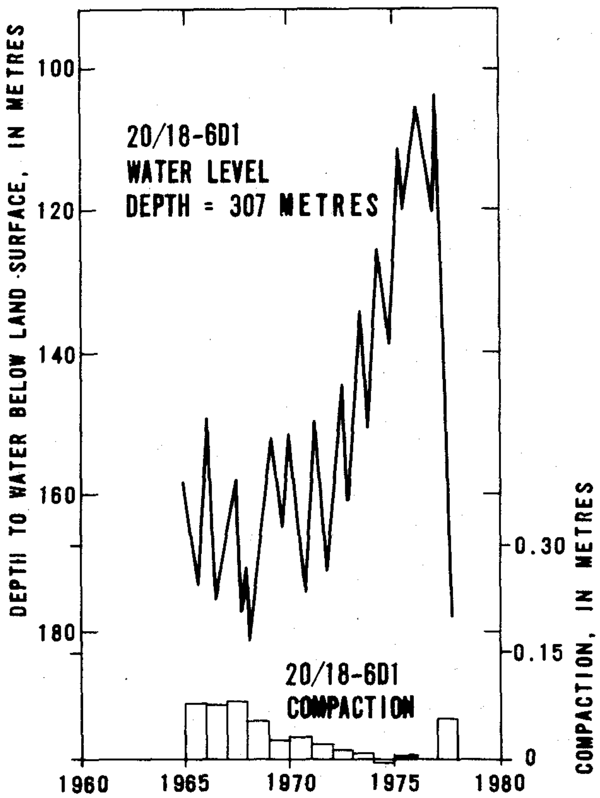


Figure 9.13.12 Seasonal fluctuations of water level and measured compaction in observation well 20/18-6D1 northeast of Huron.

9.13.8 MEASURES TAKEN TO CONTROL OR AMELIORATE SUBSIDENCE

The severe subsidence in all three areas in the San Joaquin Valley has been greatly reduced by the importation of surface water and the consequent decrease in ground-water pumping, as described earlier in this case history.

In the Tulare-Wasco area, the import of surface water from the San Joaquin River through the Friant-Kern Canal began in 1980. In the next 23 years, 1950-1972, the deliveries to this area from the canal averaged about 830 hm^3 per year, roughly 80 per cent of the surface-water supply to the area (Lofgren and Klausing, 1969). In the first 13 years of this period (1950-62), ground-water pumpage averaged about 1,230 hm^3 per year and continued at about this rate into the 1970’s. Thus, the water imported from the San Joaquin River to the area during the 23-year period 1950-72 equaled about one-quarter of the total water supply and two-thirds of the ground-water pumpage.

Hydrographs of wells tapping the semiconfined to confined aquifer system in the eastern part of the Tulare-Wasco area show a water-level recovery of about 60 m from 1950 to 1970. As a result, subsidence decreased to less than 3 cm per year in most of the eastern area as 1962-70. On the other hand, hydrographs for wells tapping the confined aquifer system in the western part of the Tulare-Wasco area show continued decline of water levels since the 1950’s; the supplemental irrigation supply from the Friant-Kern Canal to the western part has been insufficient to achieve a balance with ground-water pumping. As a result, subsidence has continued at rates locally exceeding 9 cm per year.

In the west-side area, the import of surface water through the California Aqueduct, which began in 1968, soon replaced most of the ground-water pumpage. For example, ground-water pumpage in the west-side area averaged 1,300 hm^3 per year from 1960 to 1967, before the imports began. By 1974, surface water imports to the west-side area reached 1,400 hm^3 per year and pumpage had decreased to roughly 250 hm^3 per year. The great decrease in ground-water pumpage and the consequent recovery of the artesian head in the confined aquifer system have nearly eliminated the subsidence problem for the present. However, any deficiency in surface-water imports could trigger renewed pumping, renewed head decline, and renewed subsidence, as in the severe drought year of 1977.

9.13.9 REFERENCES

BULL, W. B. 1964. Alluvial fans and near-surface subsidence in western Fresno County, California: U.S. Geol. Survey Prof. Paper 437-A, 71 p.

_____________. 1975. Land subsidence due to ground-water withdrawal in the Los Banos-Kettleman City area, California, Part 2. Subsidence and compaction of deposits: U.S. Geol. Survey Prof. Paper 437-F, 90 p.

BULL, W. B., and POLAND, J. F. 1975. Land subsidence due to ground water withdrawal in the Los Banos-Kettleman City area, California, Part 3. Interrelations of water-level change, change in aquifer-system thickness and subsidence: U.S. Geol. Survey Prof. paper 437-G, 62 p.

CROFT, M. G. 1972. Subsurface geology of the Late Tertiary and Quaternary water-bearing deposits of the southern part of the San Joaquin Valley, California: U.S. Geol. Survey Water-Supply Paper 1999-H, 29 p.

JOHNSON, A. I., MOSTON, R. P., and MORRIS, D. A. 1968. Physical and hydrologic properties of water-bearing deposits in subsiding areas in central California: U.S. Geol. Survey Prof. Paper 497-A, 71 p.

LOFGREN, B. E. 1975. Land subsidence due to ground-water withdrawal, Arvin-Maricopa area, California: U.S. Geol. Survey Prof. Paper 437-D, 55 p.

_____________. 1979. Changes in Aquifer-System Properties with Ground-Water Depletion, Proceedings, Evaluation and Prediction of Subsidence, American Society of Civil Engineers, p. 26-46.

LOFGREN, B. E., and KLAUSING, R. L. 1969. Land subsidence due to ground-water withdrawal, Tulare-Wasco area, California: U.S. Geol. Survey Prof. Paper 437-B, 103 p.

LUCAS, C. V., and JAMES, L. B. 1976. Land subsidence and the California State Water Project: Internat. Symposium on Land Subsidence, 2d, Anaheim, Calif., Dec. 1976, Proc., p. 533-543.

MEADE, R. H. 1967. Petrology of sediments underlying areas of land subsidence in central California: U.S. Geol. Survey Prof. Paper 497-C, 83 p.

MILLER, R. E., GREEN, J. H., and DAVIS, G. H. 1971. Geology of the compacting deposits in the Los Banos-Kettleman City subsidence area, California: U.S. Geol. Survey Prof. Paper 497-E, 46 p.

POLAND, J. F. 1976. Land subsidence stopped by artesian-head recovery, Santa Clara Valley, California: Internat. Symposium on Land Subsidence, 2d, Anaheim, Calif., Dec. 1976, Proc., p. 124-132.

POLAND, J. F., LOFGREN, B. E., IRELAND, R. L., and PUGH, R. G. 1975. Land subsidence in the San Joaquin Valley as of 1972: U.S. Geol. Survey Prof. Paper 437-H, 78 p.

RILEY, F. S. 1969. Analysis of borehole extensometer data from central California, in Tison, L. J., Ed., Land subsidence, V. 2: Internat. Assoc. Sci. Hydrology, Pub. 89, p. 423-431.

WILSON, W. E. 1968. Casing failures in irrigation wells in an area of land subsidence, California [abs.]: Geol. Soc. America Ann. Mtg., 81st, Mexico City, 1968, Program, p. 324.

Case History No. 9.14. Santa Clara Valley, California, U.S.A., by Joseph F. Poland, U.S. Geological Survey, Sacramento, California

9.14.1 INTRODUCTION

Land subsidence in the central part of the Santa Clara Valley--beneath the southern part of San Francisco Bay and extending to the southern edge of San Jose--was first recognized in 1932-33. Releveling of a line of first-order levels established by the National Geodetic Survey in 1912 showed about 1.2 m of subsidence in downtown San Jose in 1933. The subsiding area extends southward about 40 km from Redwood City and Niles past San Jose, has a maximum width of 22 km, and includes about 750 km^2. As shown by Figure 9.14.1, most of this central area experienced 0.3 to 2.4 m (1 to 8 feet) of subsidence from 1934 to 1967.

9.14.2 GEOLOGY

The Santa Clara Valley is a structural trough extending 110 km southeast from San Francisco. The valley is bounded on the southwest by the Santa Cruz Mountains and the San Andreas fault and on the northeast by the Diablo Range and the Hayward fault. The consolidated bedrock bordering the valley is shown as a single unit in Figure 9.14.1; it ranges in age from Cretaceous to Pliocene and consists largely of sedimentary rocks but includes areas of metamorphic and igneous rocks.

The fresh-water-bearing deposits forming the ground-water reservoir within the valley are chiefly of Quaternary age. They include (1) the semiconsolidated Santa Clara Formation and associated deposits of Pliocene and Pleistocene age and (2) the unconsolidated alluvial and bay deposits of Pleistocene and Holocene age. The Santa Clara Formation, which crops out on the southwest and northeast flanks of the valley, consists of poorly sorted conglomerate, sandstone, siltstone, and clay as much as 600 m thick in outcrop (Dibblee, 1966). Where exposed, this formation has a low transmissivity and yields only small to moderate quantities of water to wells (1 to 6 l/s)--rarely enough for irrigation purposes.

The unconsolidated alluvial and bay deposits of clay, silt, sand, and gravel overlie the Santa Clara Formation and associated deposits their upper surface forms the valley floor. They contain the most productive aquifers of the ground-water reservoir. Wells range in depth from 90 to 360 m. The deeper wells probably tap the upper part of the Santa Clara Formation although the contact with the overlying alluvium has not been distinguished in well logs. Well yields in the valley range from 20 to 160 l/s (Calif. Dept. Water Resources., 1967, pl. 6). The alluvial deposits are at least 460 m thick beneath central San Jose. However, the log of a well drilled to a depth of 468 m revealed a lack of water-bearing material below a depth of 300 M. Coarse-grained deposits predominate on the alluvial fans near the valley margins where the stream gradients are steeper. The proportion of clay and silt layers increases bayward. For example, a well-log section extending 20 km northward from Campbell to Alviso (Tolman and Poland, 1940, Figure 3) shows that to a depth of 150 m, the cumulative thickness of clay layers in the deposits increases from 25 per cent near Campbell to 80 per cent near Alviso.

In 1960, the U.S. Geological Survey drilled core holes to a depth of 305 m at the two centers of subsidence, in San Jose (well 16C6) and in Sunnyvale (well 24C7). (For location, see Figure 9.14.1.) The 305-m depth was chosen because it was the maximum depth of nearby water wells. Cores were tested in the laboratory for particle-size distribution, specific gravity of solids, dry unit weight, porosity and void ratio, hydraulic conductivity (normal and parallel to bedding), Atterberg limits, and one-dimensional consolidation and rebound (Johnson, Moston, and Morris, 1968).

X-ray diffraction studies of 20 samples from the two core holes indicate that montmorillonite composes about 70 per cent of the clay-mineral assemblage in these deposits. Other constituents are chlorite, 20 per cent, and illite, 5-10 per cent (Meade, 1967, p. 44).

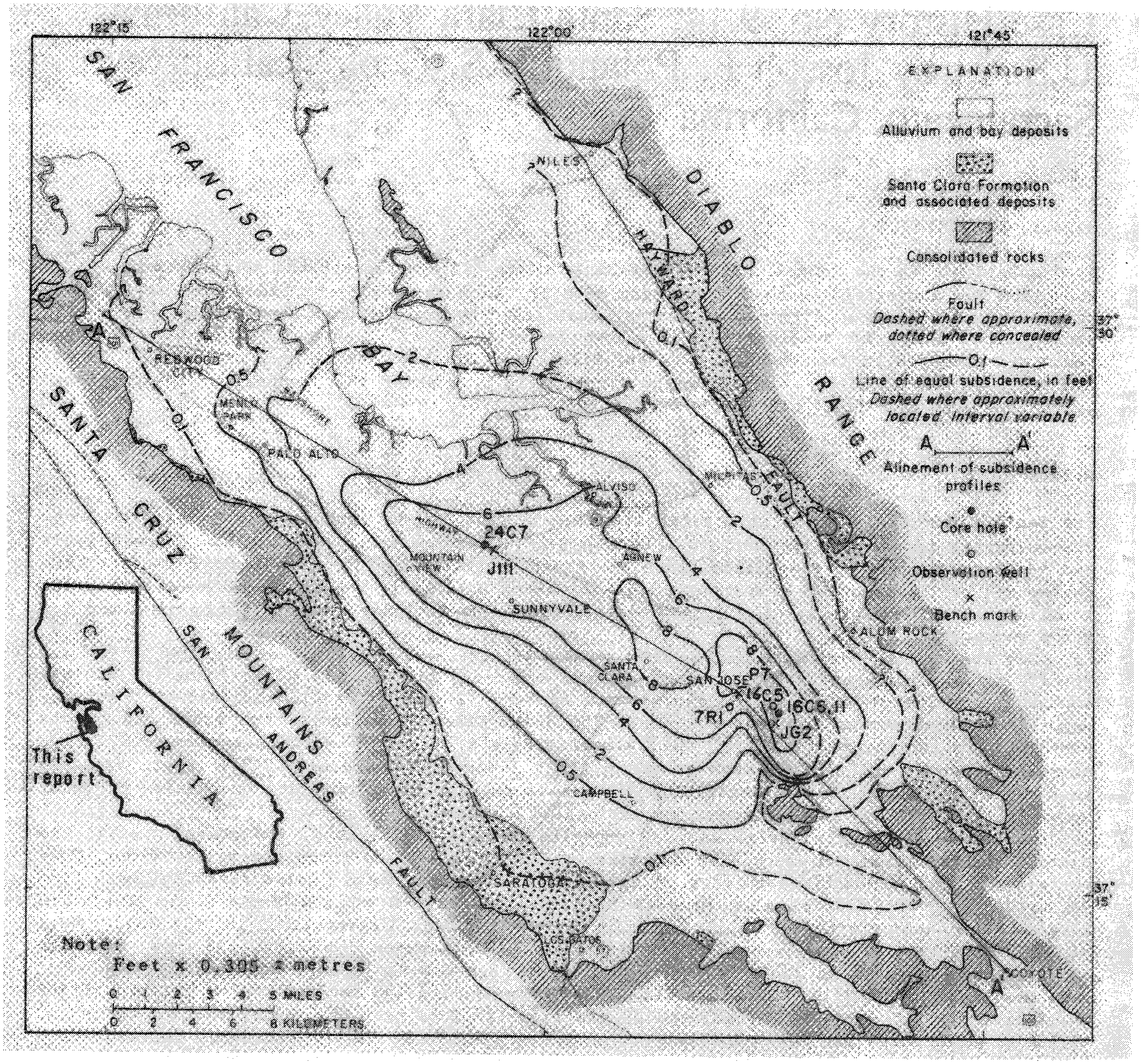


Figure 9.14.1 Land subsidence from 1.934 to 1967, Santa Clara Valley, California. Compiled from leveling of National Geodetic Survey in 1934 and 1967.

9.14.3 HYDROLOGY

In the central part of Figure 9.14.1 and below a depth of 50 to 60 m, ground water is confined. The extent of the confined aquifer system is defined roughly by the 0.6 m (2-ft) 1ine of equal subsidence in Figure 9.14.1. The area of confinement extends southward from beneath San Francisco Bay to San Jose, also west to Palo Alto and east to Milpitas. In the early years of development, wells as far south as San Jose and more than 60 m deep flowed (Clark, 1924, p.1. XV), demonstrating by their areal distribution a minimal extent of the confining sedments. The confined aquifer svstem is as much as 245 m thick. Around the valley margins, ground water is chiefly unconfined and most of the natural recharge to the ground-water reservoir percolates from stream channels in alluvial-fan deposits.

The confining member overlying the confined aquifer system has a thickness ranging from 45 to 60 m. Although predominantly composed of lenses and tongues of clay and silt, it contains some channel fillings and lenses of permeable sand and gravel. This confining member supports a shallow water table distinguished by an irregular surface. As of 1965-70, the shallow water table overlying much of the confined system was less than 10 m below the land surface (Webster, 1973). At least near the Bay, the shallow water table did not fluctuate appreciably during the period of prolonged artesian-head decline terminating in 1966.

The development of irrigated agriculture in the valley began about 1900 and expanded to a maximum about the end of World War II. After 1945, population pressures caused a great transition of land use from agricultural to urban and industrial development. Agricultural pumpage increased from about 50 hm^3 per year in 1915-20 to a maximum of 127 hm^3 per year in 1945-50 (1 cubic hectometre, hm^3, = 1 x $10^6 m^3$ = 810.7 acre-feet). By 1970-75 most of the orchards had been replaced by houses, and agricultural pumpage had decreased to 25 hm^3 per year. Municipal and industrial pumpage, on the other hand, increased from 27 hm^3 per year in 1940-45 to 162 hm^3 per year in 1970-75. Total pumpage (Figure 9.14.2, bottom graph) increased nearly fourfold from 1915-20 to 1960-65--from 60 to 222 hm^3 per year--but then decreased 19 per cent to 185 hm^3 by 1970-75, in response to a rapid increase in surface-water imports, discussed later.

The historical increase in withdrawal of ground water was a principal factor in causing a fairly continuous and severe 50-year decline of artesian head. In the spring of 1916, the artesian head in index well 7Rl in San Jose was 3.7 m above land surface (Figure 9.14.2); by the autumn of 1966 it was 55 m below land surface. The second major factor in this 50-year decline of 59 m was the negative trend of the local water supply. The upper line in Figure 9.14.2 is a plot of the cumulative departure, in per cent, of the seasonal rainfall at San Jose from the 50-year seasonal mean, 1897-98 to 1946-47 (Calif. State Water Resources Board, 1955, p. 26). The 50-year mean is 34.85 cm. Except for the 6-year wet period 1936-42, the departure in the 50

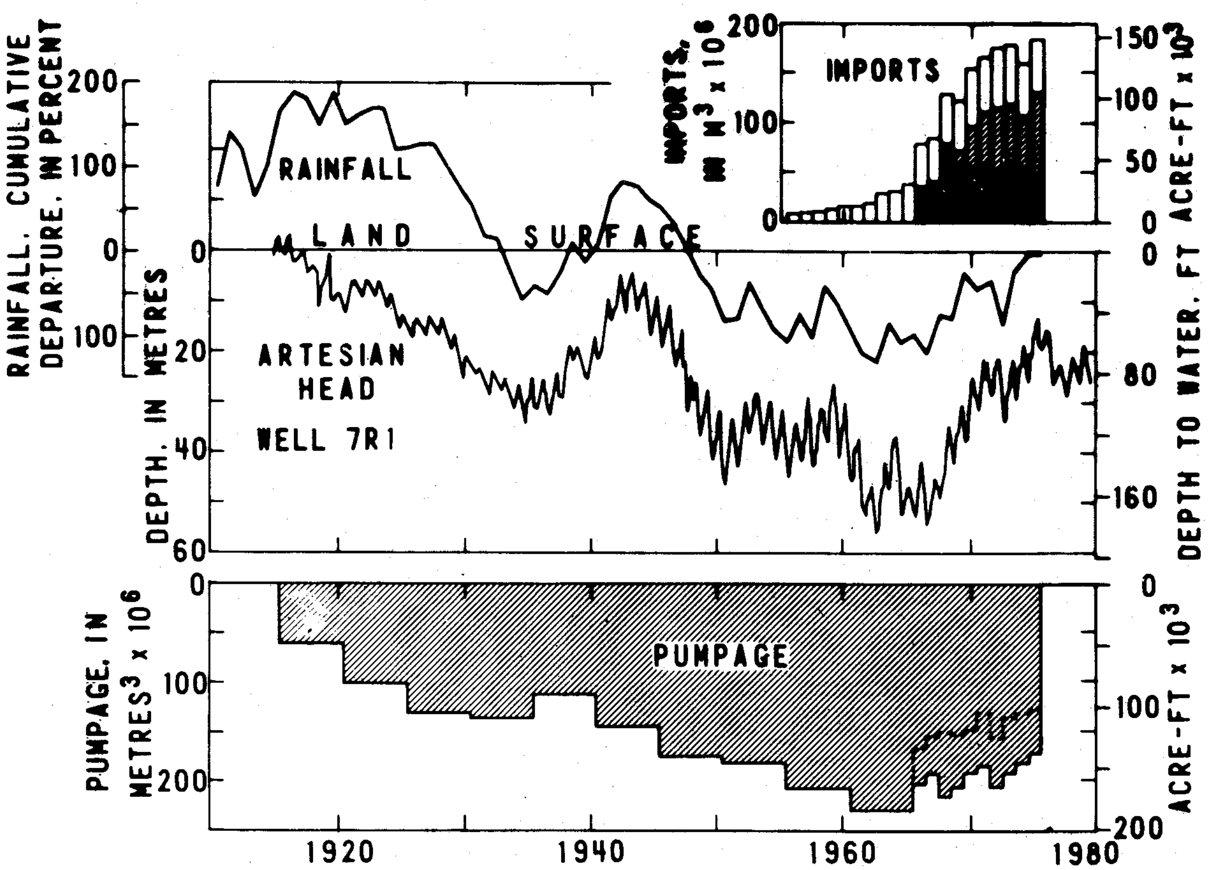


Figure 9.14.2 Artesian-head change in San Jose in response to rainfall, pumpage, and imports.

years 1916--66 was generally negative; the cumulative departure of 310 per cent from 1916 to 1966 represents a cumulative "deficiency" in rainfall of about 108 cm.

The 50-year decline in artesian head from 1916 to 1966 clearly was caused by the cumulative effect of generally deficient rainfall and runoff and a fourfold increase in withdrawals. The plot of artesian-head decline at index well 7R1 conforms in general with the cumulative departure of rainfall at San Jose.

9.14.4 LAND SUBSIDENCE

Land subsidence was first noted in 1932-33 when bench mark P7 in San Jose, established in 1912, was resurveyed and found to have subsided 1.2 m. As a result, a valleywide network of bench marks was established in 1934 (Poland and Green, 1962, Figure 3). The total length of survey lines comprising this bench-mark net was about 400 km. From 1934 to 1967 the National Geodetic Survey (formerly the U.S. Coast and Geodetic, Survey) resurveyed the network from "stable" bedrock ties a dozen times to determine changes in elevation of the bench marks; the latest full survey of the network was in 1967. In the 33 years 1934-67, subsidence along lines of benchmark control ranged from 0.3 to 1.2 m under the Bay to 2.4 m in San Jose (Figure 9.14.1). About 260 km^2 subsided more than 1 m. The subsidence record for bench mark P7 in central San Jose is plotted in Figure 9.14.3, together with the artesian head in nearby index well 7R1, taken from Figure 9.14.2. The black dots on the subsidence curve indicate times of bench-mark surveys. The fluctuations of artesian head represent the change in stress on the confined aquifer system; the subsidence is the resulting strain. Subsidence of bench mark P7 began about 1918 (note dotted inferred segment of subsidence plot representing the period 1912 to 1919) and reached 1.4 m in 1934. From 1938 to 1947 subsidence stopped, during a period of artesian-head recovery, in response to above-normal rainfall and recharge. (The natural recharge was supplemented by controlled percolation releases from newly constructed detention reservoirs on the larger streams.) Subsidence resumed in 1947 as a consequence of a rapidly declining artesian head due to deficient rainfall and increasing demand for ground water (Figure 9.14.2); it attained its fastest average rate in 1960-63 (0.22 m/year), in response to the rapid head decline of 1959-62 during a drought period (see Figure 9.14.2). By 1967 bench mark P7 had subsided 3.86 m.

Figure 9.14.4 shows land-subsidence profiles along line A-A′ from Redwood City to Coyote from 1912 through 1969 (for location, see Figure 9.14.1). The spring 1934 leveling was used as a reference base because this was the first complete leveling of the net. Note that from 1934 to 1967, maximum subsidence of 2.6 m was near bench mark W111, 4.8 km northwest of bench mark P7; also that from 1934 to 1960 the greatest subsidence along line A-A′ was 1.7 m, at bench mark

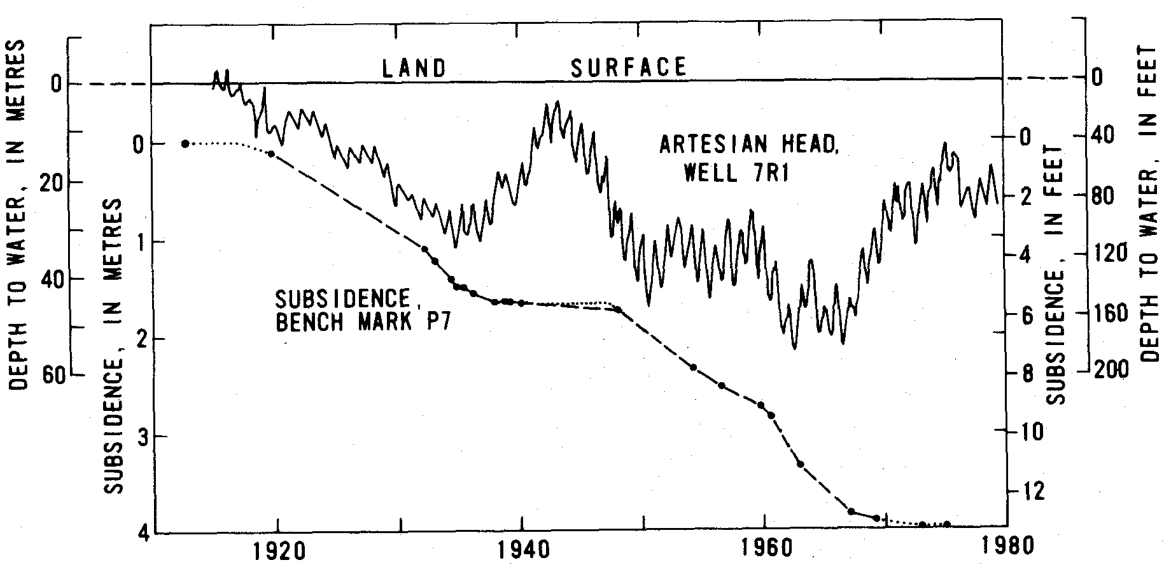


Figure 9.14.3 Artesian-head change and land subsidence, San Jose.

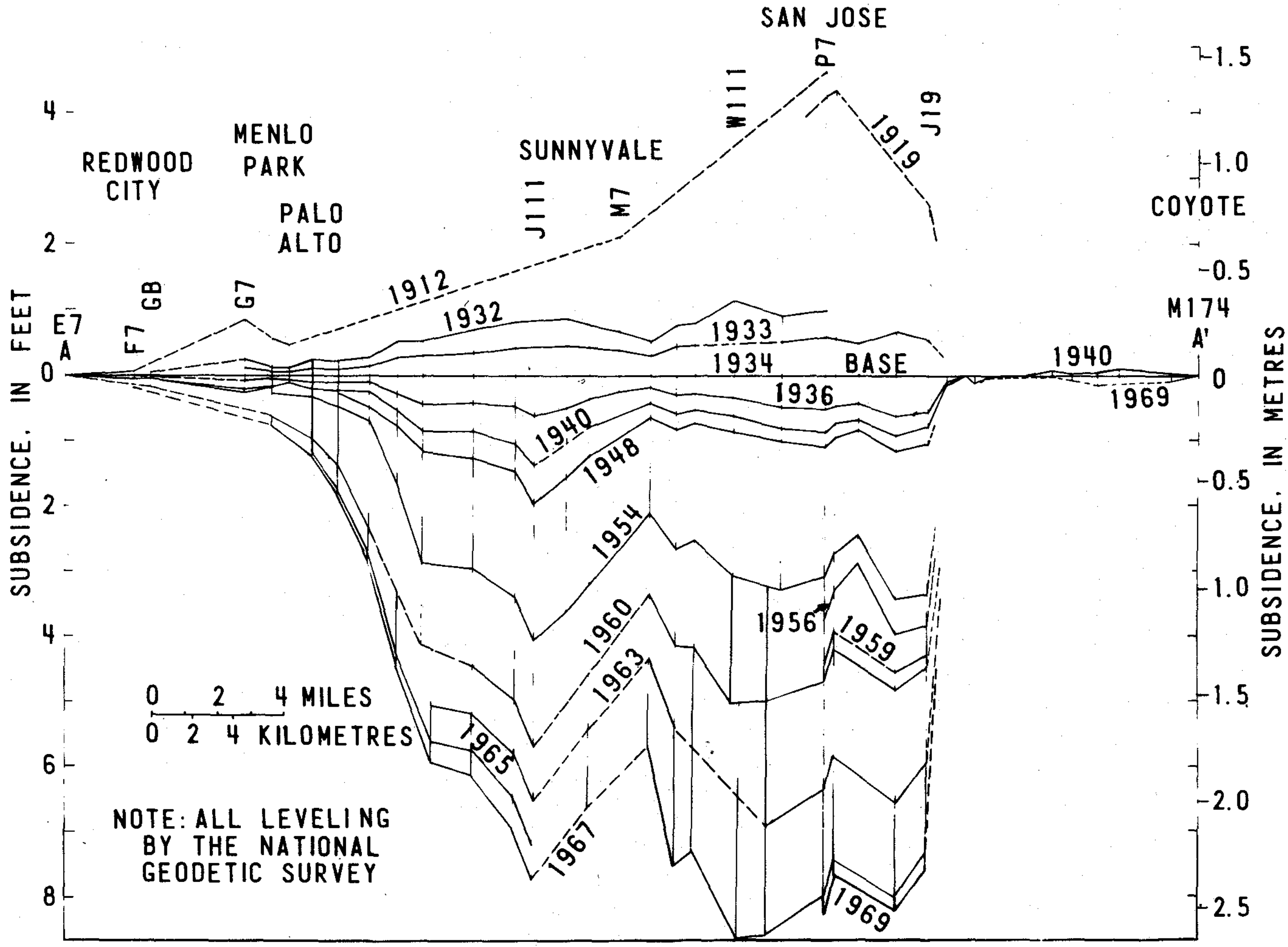


Figure 9.14.4 Profiles of land subsidence, Redwood City to Coyote, California, 1912-69.

J111 in Sunnyvale. Changes in the rate and magnitude of artesian-head decline doubtless have caused such geographic variations in subsidence rate and magnitude with time.

The volume of subsidence (pore-space reduction) planimetered from the 1934-67 subsidence map (Figure 9.14.1) was about 617 hm^3. If the ratio of the pre-1934 subsidence volume to the 1934-67 subsidence volume is assumed to be equal to the ratio of the pre-1934 subsidence of bench mark P7 to the 1934-67 subsidence of that bench mark, then the total subsidence volume from 1912 to 1967 is about 975 hm^3. Protrusion of well casings above the land surface and inundation of lands near the south end of San Francisco Bay also have furnished evidence of subsidence. Protrusion of well casings has been common in the subsiding area (Tolman, 1937, p. 345). Many of the casings gradually protruded 0.6-1 m above ground level but usually were cut off before protruding higher. This protrusion indicates that compaction of the deposits occurred in the depth interval above the bottom of the protruding casing. However, such protrusion often is accompanied by compression and rupture of the casing at depth and thus supplies only a minimal value of subsidence. In general, the deeper the compacting interval, the smaller will be the protrusion in proportion to the subsidence, because the frictional drag of the formation or the gravel-pack on the casing wall should increase proportionately with depth.

Although some horizontal movement doubtless has occurred in the subsidence area in association with the subsidence, no surveys or evidence of horizontal movement are known to the author.

The comparison of artesian-head change and subsidence from 1916 to 1967 (Figure 9.14.3) demonstrates beyond a reasonable doubt that the increase in effective stress resulting from the declining artesian head caused the compaction and the subsidence.

9.14.5 EXTENSOMETERS TO MEASURE COMPACTION

Extensometers (compaction recorders) were installed by the Geological Survey in 1960 in the cased core holes 305 m deep in San Jose (16C6) and in Sunnyvale (24C7) and in several unused water-supply wells. (For location, see Figure 9.14.1.) The purpose of this equipment was to measure the rate and magnitude of compaction occurring between the land surface and the well bottom. When first installed, the extensometer consisted of an anchor placed in the formation below the casing bottom, attached to a cable that passed over sheaves at the land surface and was counterweighted to maintain constant tension (Figure 2.5A). A recorder actuated by cable movement yields a time graph of the movement of land surface with respect to the anchor--the compaction or expansion of the deposits within that depth range. To reduce friction and increase the accuracy of measurement four of the extensometers were modified in 1972 by replacing the cable with a free-standing pipe of 3.8-cm diameter (Figure 2.5B) within the well casing of 10-cm diameter. The records obtained from these instruments show that the measured compaction to the depth of 305 m is nearly equal to the land subsidence as measured periodically by releveling of the bench-mark network. Thus, these instruments function as continuous subsidence monitors.

Figure 9.14.5 shows the measured compaction in the 305-m well in San Jose (well 16C6) and the compaction and artesian-head fluctuation in adjacent unused well 16C5 (depth 277 m) through 1975. The dashed line represents subsidence of adjacent bench mark JG2 as determined by periodic releveling from stable bench marks. Measured compaction of the confined aquifer system to the 305-m depth from July 1, 1960, to December 31, 1976, was 1.4 m.

9.14.6 MEASURES TAKEN TO CONTROL SUBSIDENCE

Local agencies have been working since the 1930's to conserve water and to obtain water supplies adequate to stop the ground-water overdraft and raise the artesian head. Their program has involved (1) salvage of flood waters from local streams that would otherwise waste to the Bay and (2) importation of water from outside the valley. In 1935-36 five storage dams were built on local streams to provide detention reservoirs with combined storage capacity of about 62 hm^3 to retain floodwaters and permit controlled releases to increase streambed percolation (Hunt, 1940). The storage capacity of detention reservoirs was increased to 178 hm^3 in the early 1950's (Calif. State Water Resources Board, 1955, p. 51).

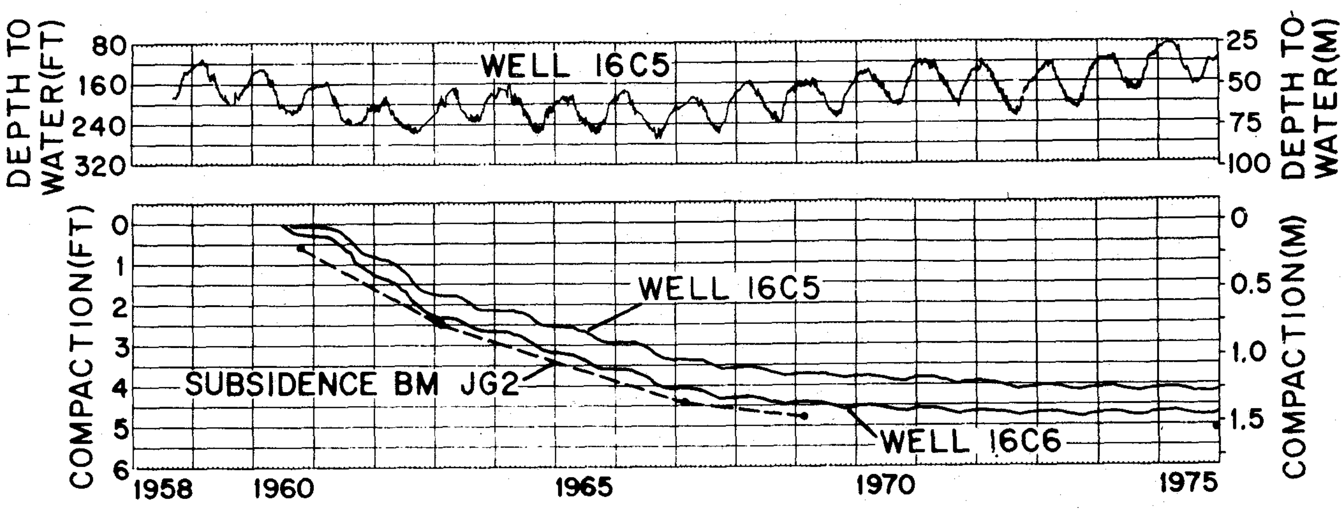


Figure 9.14.5 Measured water-level change, compaction, and subsidence in San Jose.

By 1960, sharply declining water levels furnished evidence that local resources were not adequate to supply present and future water needs. Steps were taken to increase water imports to the County. The import of surface water to Santa Clara County began about 1940 when San Francisco commenced selling water imported from the Sierra Nevada to several municipalities. This import increased to 15 hm^3 in 1960 and to 54 hm^3 by 1975 (see blank segments of yearly bars, upper right graph, Figure 9.14.2). Surface water imported from the Central Valley through the State's South Bay Aqueduct first became available in 1965; by 1974-75, the aqueduct import was 128 hm^3 (see cross-hatched plus diagonally ruled segments of yearly bars, upper right graph, Figure 9.14.2). As a result, total imports to Santa Clara County increased five-fold from 1964-65 to 1974-75--from 37 to 183 hm^3 per year.

The recovery of water level since 1967 has been dramatic. By 1975, the spring high water level at index well 7R1 (Figure 9.14.2) was 32 m above that of 1967, and about equal to the level in this well in 1925. This major recovery of head was due primarily to the fivefold increase in imports from the Central Valley. Two other favorable factors were the above-normal rainfall and the decreased pumpage (Figure 9.14.2).

The average seasonal rainfall at San Jose was 13 per cent above normal in the period 1966-75. The cumulative departure graph (Figure 9.14.2) indicates an increase of 120 per cent or a cumulative excess of about 41 cm above normal in the 9-year period.

The average yearly pumpage of ground water, which had reached its peak of 228 hm^3 in 1960-65, decreased to 185 hm^3 in 1970-75. A principal reason for this 19-per cent decrease was a use tax levied on ground-water pumpage since 1964. In 1977, for example, the ground-water tax was levied at $8.50 per unit (1 acre-ft. or 1234 m^3) for ground water extracted for agricultural purposes and at $34 per unit for ground water extracted for other uses. The energy cost to the consumer for pumping ground water in the Santa Clara Valley at 1977 prices was $10 to $15 per unit. Thus, the average total cost for ground water pumped for agricultural purposes was about $20 per unit and for other uses was about $45 per unit. The price for surface water delivered in lieu of extraction was $14 per unit for water used for agriculture and $39.50 per unit for water used for other purposes. The economic advantage of buying surface water, where available, is obvious.

Recharge to the ground-water reservoir from regulated local runoff released to stream channels and percolation ponds has been augmented since 1965 by water from the South Bay Aqueduct that could not be delivered directly to the user. The quantity diverted to recharge areas (cross-hatched segment of yearly bars, upper right graph, Figure (9.14.2) in the 10 years to 1975 averaged about 50 hm^3 per year and represents 56 per cent of the total import from the South Bay Aqueduct.

The marked decrease in rate of subsidence in response to the dramatic head recovery from 1967 to 1975 is demonstrated graphically by the compaction records from the two deep extensometers in San Jose and Sunnyvale (Figure 9.14.6). The rate of measured compaction in well 16C6 in San Jose decreased from about 30 cm per year in 1961 to 7.3 cm in 1967 and to 0.3 cm in 1973. Net expansion (land-surface rebound) of 0.6 cm occurred in 1974. In Sunnyvale, compaction of the sediments above the 305-m anchor in well 24C7 decreased from about 15 cm per year in 1961 to 1.2 cm in 1973; net expansion of 0.5 cm and 1.1 cm occurred in 1974 and 1975, respectively. Very deficient rainfall in 1975-76 and in 1976-77 virtually eliminated runoff and recharge from local sources, and water levels started to decline once more in 1976. In response, compaction and subsidence resumed once again. In San Jose at well 16C6, compaction in 1976 was 3.5 cm, about equal to that in 1968; in Sunnyvale, compaction was 1.6 cm.

9.14.7 COMPRESSIBILITY AND STORAGE PARAMETERS

Compressibility characteristics of fine-grained compressible layers (aquitards) can be obtained by making one-dimensional consolidation tests of "undisturbed" cores in the laboratory. As one phase of the research on compaction of the aquifer system, laboratory consolidation tests were made on 21 selected fine-grained cores from the two core holes. These tests were made in the Earth Laboratory of the United States Bureau of Reclamation at Denver, Colorado. Parameters tested included the compression index, C_c, a measure of the nonlinear compressibility of the sample, and the coefficient of consolidation, C_v, a measure of the time rate of consolidation. Complete results of these laboratory tests have been published (Johnson and others, 1968, Tables 8 and 9 and Figure 21). The 21 samples tested spanned a depth range from 43 to 292 m below land surface. The range of the compression index, C_c, was small compared to the range in the San Joaquin Valley: the maximum value was 0.33, the minimum 0.13, and the mean was 0.24. Of the 21 samples, 15 had C_c values falling between 0.20 and 0.30. This suggests that the nonlinear

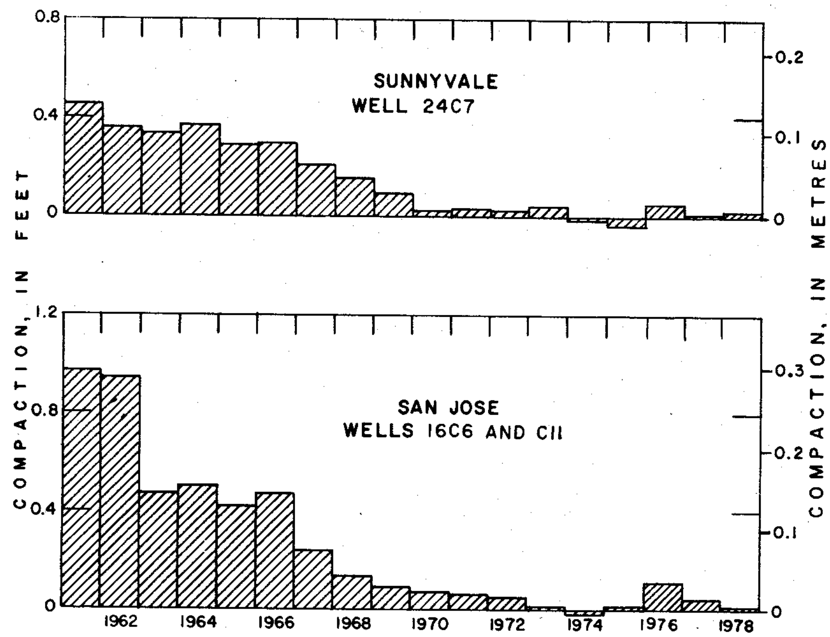


Figure 9.14.6 Measured annual compaction to 305-m (1,000-ft) depth.

compressibility characteristics of the aquitards in the confined aquifer system do not vary widely.

The plot of void ratio against the log of load (effective stress), known as the e-log p plot, can be used to obtain a graphic plot of compressibility versus effective stress. Such a graph can be used to estimate ultimate compaction due to a step increase in effective stress. This procedure applied to the laboratory consolidation tests at the Sunnyvale and San Jose core holes produced estimates of ultimate compaction that were only about one-third to one-half the values obtained by summing field measurements of compaction to date with residual compaction estimated from a one-dimensional simulation of the field observations (Helm, 1976). The reason for this disparity is not known. Apparently the samples tested were not representative of the aquitards that contributed most to the observed compaction.

Subsidence represents pore-space reduction which occurs almost wholly in the fine-grained compressible aquitards. At well 16C6 in San Jose the confined aquifer system is 244 m thick, from 61 to 305 m below land surface. Based on study of the microlog, the confined system contained 38 aquitards with a combined thickness of 145 m. The mean porosity of 27 core samples, determined in the laboratory, was 37 per cent. The total subsidence to date at well 16C6 is about 4 m. A reduction of 4 m in the thickness of the confined system requires about 1.8 per cent reduction in the porosity of the aquitards--for example, from 37 to 35.2 per cent.

The subsidence/head-decline ratio is a useful parameter in subsidence studies. The ratio is a rough approximation of the response of the aquifer system to a given change in stress. At San Jose, referring to the plot of subsidence for bench mark P7 and the artesian-head change in well 7R1 (Figure 9.14.3), the artesian head declined from 6 m below land surface in 1918 (approximate preconsolidation stress) to 55 m below land surface in 1966, for a net change of 49 m. Subsidence at bench mark P7 from 1918-66 was about 3.84 m. This means that as of 1966 the empirical ratio is 3.84 m/49 m = 0.08. The ratio of ultimate subsidence to head decline must therefore be larger than 0.08 at this site. Artesian head as measured in a well casing represents a composite pore pressure of all aquifers in the confined system that are tapped by the observation well. If and when the pore pressures in fine-grained aquitards reach equilibrium with those in the adjacent aquifers, compaction will cease, and the ratio of ultimate subsidence to head decline will be a true measure of virgin compressibility for the entire interval being stressed. Such an ultimate value is analogous to a storage coefficient.

Helm (1977), by means of a one-dimensional simulation of the long-term field observations of subsidence at bench march P7 and artesian head at well 7RI, provided the parameters used for estimating the ultimate compaction (subsidence) resulting from a step change in head of 49 m;

the computed compaction is about 5.3 m. Thus, on the basis of Helm's parameter values, the ultimate subsidence/head-decline ratio would be 5.3 m/49 m = 0.11. If we divide the ratio by the thickness of compacting aquitards, 145 m, we obtain the virgin compressibility (for stress increase beyond preconsolidation stress) of the aquitards:

$$5.3\ \text{m}/(145\ \text{m} \times 49\ \text{m}) = 7.4 \times 10^{-4}\text{m}^{-1}$$

As the water levels in the San Jose area rose rapidly after 1967 (Figure 9.14.2), the stress-strain curves obtained from paired measurements of compaction and artesian head began to show seasonal expansion during the winter months when the water level was highest and the effective stress on the confined system was lowest. These stress-strain loops can be used to obtain the compressibility of the confined system in the recoverable or elastic range of stresses (less than preconsolidation stress). One example (Figure 9.14.7) shows the stress-compaction plot for a pair of wells in San Jose from 1967 through 1974. Compaction was measured in well 16C6,11, 305 m deep, and stress in nearby well 16C5. Depth to water is plotted increasing upward. Change in depth to water represents an average change in stress in all aquifers of the confined aquifer system tapped by well 16C5. The lower parts of the descending segments of the annual loops for the winters of 1967-68, 1969-70, and 1970-71 are approximately parallel, as shown by the dotted lines, indicating that the response is essentially elastic in both aquifers and aquitards when the depth to water is less than about 55 m. The heavy dashed line drawn parallel to the dotted lines represents the average slope of the segments in the range of stresses less than preconsolidation stress. The reciprocal of the slope of this line is the component of the storage coefficient attributable to elastic or recoverable deformation of the aquifer-system skeleton, S_{ke}, and equals 1.5×10^{-3}. The component of average specific storage due to elastic deformation, S_{ske}, equals S_{ke}/244 m = $6.15 \times 10^{-6}\text{m}^{-1}$, if stresses are expressed in metres of water, and if γ_w (the unit weight of water) = 1, the average elastic compressibility of the aquifer system skeleton, α_{ke}, is equal numerically to S_{ske}.

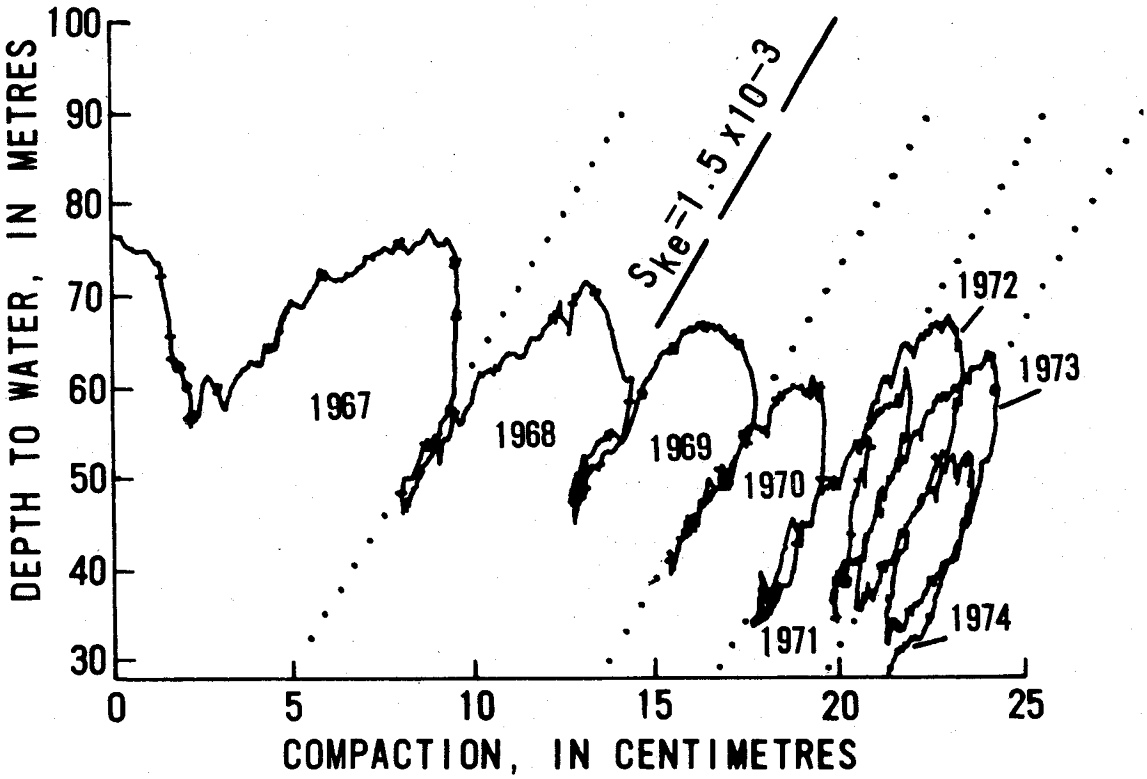


Figure 9.14.7 Stress change and compaction, San Jose site.

In these computations I have assumed that in the range of stresses less than preconsolidation stress, the compressibility of the aquitards and the aquifers is the same. Therefore, the full thickness of the confined aquifer system, 244 m, was used to derive the specific storage component, S_{ske}, in the elastic range of stress.

At these San Jose sites, then, the average compressibility of the aquitards in the virgin range of stress, $7.4 \times 10^{-4}\ m^{-1}$, is 120 times as large as the average compressibility of the confined aquifer system in the elastic range of stress, $6.15 \times 10^{-6}\ m^{-1}$. This great difference in response to stressing should be kept in mind when considering use of aquifer tests to derive hydrologic parameters, as well as in appraisal of subsidence potential.

9.14.8 ECONOMIC AND SOCIAL IMPACTS

Subsidence has created several major problems. Lands adjacent to San Francisco Bay have sunk as much as 2.4 m since 1912, requiring construction and repeated raising of levees to restrain landward movement of the saline bay water onto 44 km2 of land below high-tide level in 1967. Also, flood-control levees have been built and maintained near the bayward ends of the depressed stream channels. About $9 million of public funds had been spent to 1974 on such flood-control levees to correct for subsidence effects, according to Lloyd Fowler, former Chief Engineer of the Santa Clara Valley Water District. In addition, a major salt company has spent an unknown but substantial amount maintaining levees on 78 km2 of salt ponds to counter as much as 2.4 m of subsidence. Several hundred water-well casings have failed in vertical compression, due to compaction of the sediments. The cost of repair or replacement of such damaged wells has been estimated as at least $4 million (Roll, 1967). Including funds spent on maintaining the salt-pond levees, establishing and resurveying the bench-mark net, repairing railroads, roads, and bridges, replacing or increasing the size of storm and sanitary sewers, and making private engineering surveys, the direct costs of subsidence must have been at least 35 million dollars to date.

A major earthquake could cause failure of the bay-margin levees, resulting in the flooding of areas presently below sea level. The levees were constructed of locally derived weak materials and were designed only to retain salt-pond water under static conditions (Rogers and Williams, 1974). The potential for such an earthquake poses a continuing threat to flooding of the estimated 44 km^2 (4400 hectares) of land standing below high tide level as of 1967. Such a threat must have reduced the value of this land very substantially compared to the value if it all still stood above mean sea level as it did in 1912. This decrease in land value should be included in the gross costs of subsidence.

9.14.9 LEGAL ASPECTS

The successful management of a highly variable water supply to achieve a balance with an ever-increasing demand for water in Santa Clara County (not shown on map) has been remarkable for several reasons. First, maximum development of local water supplies and importation of water from two sources have momentarily brought supply and demand into balance. Secondly, by building up the ground-water storage in the recharge area, and thus the artesian head in the confined system, land subsidence was stopped, at least temporarily, by 1973. Thirdly, all this has been accomplished by bond issues, revenue from taxes, and water charges, thus avoiding a drawn-out expensive legal adjudication of the ground-water supply such as occurred in southern California, in the Raymond Basin (Pasadena vs. Alhambra, 1949).

9.14.10 CONCLUSIONS

Both the cause of subsidence and the means of its control are known. The evidence given here proves that the subsidence is caused by decline of the artesian head and the resulting increase in effective overburden load or grain-to-grain stress on the water-bearing beds in the confined system. The sediments compact under the increasing stress and the land surface sinks. Most of the compaction occurs in the fine-grained clayey beds (aquitards) which are the most compressible but have low permeability. Therefore, the escape of water from these slow draining aquitards (decay of excess pore pressure) and the increase in effective stress are slow and time-dependent, but the ultimate compaction is large and chiefly permanent.

The subsidence has been stopped by raising the artesian head in the aquifers until it equaled or exceeded the maximum pore pressures in the aquitards. The compaction and water-level records being obtained by the Geological Survey indicate that if the artesian head can be maintained 3 to 6 m above the levels of 1971-73, subsidence will not recur. On the other hand, subsidence will recommence if artesian head is drawn down as much as 6 to 9 m below the 1971-73 levels.

9.14-11 EPILOGUE

Recently the Santa Clara Valley Water District was given Historical Landmark status by the American Society of Civil Engineers for its major contributions to the development of the region. It was acknowledged that the district’s system is "the first and only instance of a major water supply being developed in a single ground-water basin involving the control of numerous independent tributaries to effectuate almost optimal conservation of practically all of the sources of water flowing into the basin."

9.14.12 REFERENCES

CALIFORNIA DEPARTMENT OF WATER RESOURCES. 1967. Evaluation of ground-water resources, South Bay: Calif. Dept. Water Resources Bull. No. 118-1, Appendix A, Geology, 153 p.

CALIFORNIA STATE WATER RESOURCES BOARD. 1955. Santa Clara Valley Investigation: Calif. State Water Resources Board Bull. No. 7, 154 p.

CLARK, W. 0. 1924. Ground water in Santa Clara Valley, Calif.: U.S. Geol. Survey Water-Supply Paper 519, 207 p.

DIBBLEE, T. W. 1966. Geologic map of the Palo Alto 15-minute quadrangle, California: Calif. Div. Mines and Geology, Map sheet 8.

HUNT, G. W. 1940. Description and results of operation of the Santa Clara Valley Water Conservation Districts project: Am. Geophys. Union Trans., pt. 1, p. 13-22.

HELM, D. C. 1977. Estimating parameters of compacting fine-grained interbeds within a confined aquifer system by a one-dimensional simulation of field observations: Internat. Symposium on Land Subsidence, 2d, Anaheim, Calif., Dec. 1976, Proc., p. 145-156.

JOHNSON, A. I., MOSTON, R. P., and MORRIS, D. A. 1968. Physical and hydrologic properties of water-bearing deposits in subsiding areas in central California: U.S. Geol. Survey Prof. Paper 497-A, 71 p.

MEADE, R. H. 1967. Petrology of sediments underlying areas of land subsidence in central California: U.S. Geol. Survey Prof. Paper 497-C, 83 p.

PASADENA v. ALHAMBRA (33 Cal. 2d 908 207 Pac. 2d 17) 1949; certiorari denied (339 U.S. 937) 1950.

POLAND, J. F. 1969. Land subsidence and aquifer-system compaction, Santa Clara Valley, California, USA, in Tison, L. J., ed., Land Subsidence, Vol. 2: Internat. Assoc. Sci. Hydrology, Pub. 88, p. 285-292.

_________. 1977. Land subsidence stopped by artesian-head recovery, Santa Clara Valley, California.: Internat. Symposium on Land Subsidence, 2d, Anaheim Calif., Dec. 1976, Proc., p. 124-132 (I.A.H.S., Pub. 121).

POLAND, J. F., and Green, J. H. 1962. Subsidence in the Santa Clara Valley, California--A progress report: U.S. Geol. Survey Water-Supply Paper 1619-C, 16 p.

ROGERS, T. H., and Williams, J. W. 1974. Potential seismic hazards in Santa Clara County, Calif.: Calif. Div. Mines and Geology, Special Report 107, 39 p. 6 pl.

ROLL, J. R. 1967. Effect of subsidence on well fields: Am. Water Works Assoc. Jour., v. 59, no. 1, p. 80-88.

TOLMAN, C. P. 1937. Ground Water: New York, McGraw-Hill Book Co., 593 p., lst ed.

TOLMAN, C. P., and Poland, J. F. 1940. Ground-water, salt-water infiltration, and ground-surface recession in Santa Clara Valley, Santa Clara County, California: Am. Geophys. Union Trans., p. 23-35.

WEBSTER, D. A. 1973. Map showing areas bordering the southern part of San Francisco Bay where a high water table may adversely affect land use: U.S. Geol. Survey Misc. Field Studies Map MF 530.

Case History No. 9.15. Ravenna, Italy, by Laura Carbognin, Paolo Gatto, and Giuseppe Mozzi, National Research Council, S. Polo 1364, Venice, Italy

9.15.1 INTRODUCTION

Ravenna is about 60 km south of the Po delta, in a symmetric position with respect to Venice (Figure 9.15.1). Land subsidence in this area has been observed for a long time but only recently did the related consequences become critical. Progressively affecting the entire territory of about 700 km^2 (Figure 9.15.2), the subsidence increasingly threatens not only the industrial area, and the urban zones, but also the surrounding vast marshland reclamations which could be submerged once again. The existence of several buildings and historical monuments is jeopardized as well, since their foundations have to be kept dry by pumping out water continuously.

It became clear from the first analysis, started in 1970 by the National Research Council of Venice at the request of the Municipality of Ravenna, that the causes of land subsidence had to be mainly ascribed to the removal of fluids from the subsurface (Bertoni, et al., 1973).

The investigation began with the inventory of available stratigraphic, hydrological, geotechnical, and geodetic data. Unfortunately no information was available concerning physical and mechanical properties of the formations. Good historical data are available for both piezometry of the aquifers and subsidence. Field measurements such as leveling and hydraulic head records were carried out almost annually, using networks of suitably placed bench marks and piezometers, similarly to what was done for Venice. The preliminary hydrogeological in-

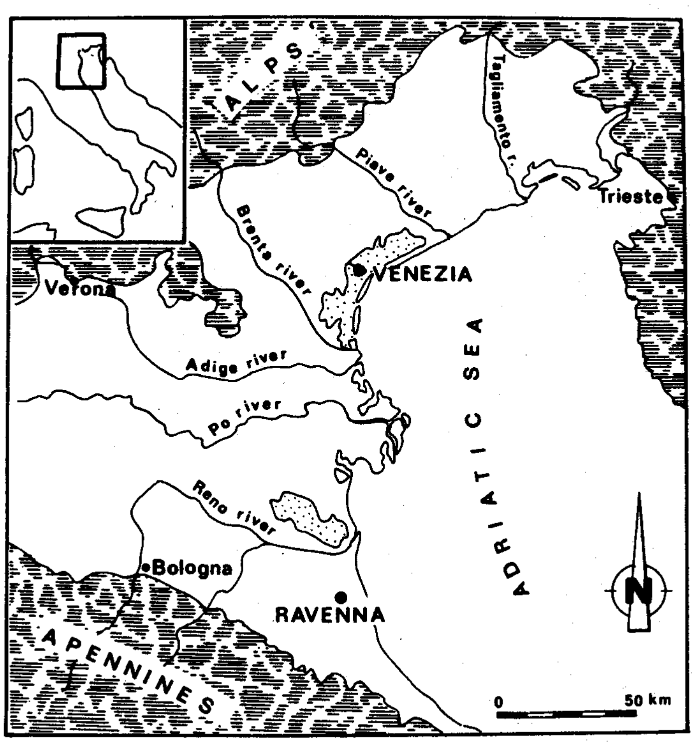


Figure 9.15.1 Areas of Ravenna and Venezia. They are symmetric with respect to the Po delta. (From Carbognin, et al., 1978, Figure 13; published with permission of the American Society of Civil Engineers.)

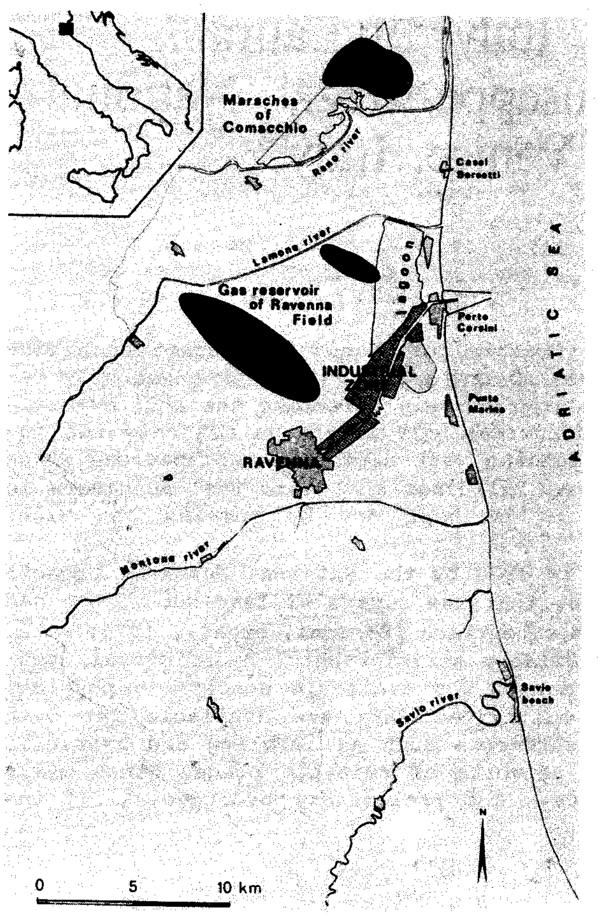


Figure 9.15.2 Map of the area under investigation (District of Ravenna). (From Carbognin, et al., 1978, Figure 1; published with permission of the American Society of Civil Engineers.)

vestigation will be further improved by using the information obtained through specifically programmed test holes. The research already undertaken has, however, provided a good understanding of the overall subsidence occurrence.

After a preliminary description of the geological environment, this paper presents the history of the pressure decline in the aquifer and land settlement and discusses their relationships.

9.15.2 HYDROLOGICAL FEATURES

The total thickness of Quaternary sediments in the Ravenna area ranges between 1500 and 3000 metres and mostly consists of sandy and silty-clay layers of alluvial and marine origin. The bottom of the Quaternary sediments follows the structure of the pre-Quaternary substratum, characterized by folds and faulted overfolds which are parallel to the main tectonic profiles of the Apennines and include several gas-bearing traps at depths on the order of 2000 m (Figure 9.15.3) (Agip Mineraria, 1969a).

The presence of massive Quaternary deposits confirms that in the past the geologic subsidence was quite pronounced in this area and is still rather active (Salvioni, 1957); it is apparent that the tectonic stresses acting along a SW direction tend to increase the Po basin curvature. The deep structure has influenced the thickness of the Neozoic formations and consequently the subsidence rate exhibits a non-uniform space distribution (Dal Piaz, 1969).

The stratigraphy of the upper Quaternary sediments is not defined with accuracy, due to the partial lack of information. However, it has been possible to reconstruct schematically the map

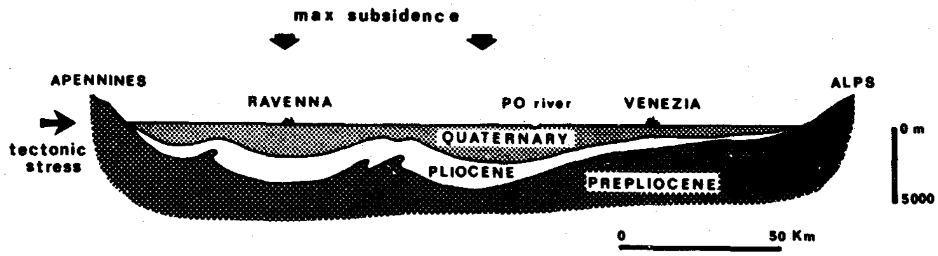


Figure 9.15.3 Very schematic cross-section of the Po Valley between Venezia and Ravenna (Agip Mineraria, 1969a).

of the aquifer system down to 500 m using the relative positions of the intakes of several pumping wells and other sparse lithological information.

Between 90 and 430 m the confined units are well identified and rather continuous (Figure 9.15.4) (Bertoni, et al., 1973). In the upper 90 m the areal continuity of the sands is quite limited and the definition of large important formations is uncertain. This portion of the system is little exploited due both to reduced productivity and possible water pollution from the overlying polluted unconfined aquifer. Below 430 m the salt content becomes very high (Agip Mineraria, 1972) and the water cannot be used any longer for industrial and/or agricultural purposes.

From the information available, the aquitards separating the various sandy formations appear to be rather continuous with very low permeability. The logs suggest that large amounts of silty sediments are present. The aquifers shown in Figure 9.15.4 consist mostly of fine and medium sands with occasional shells. However, clayey or silty sands also may be found which locally reduce the aquifer transmissivity.

The recharge of this confined multi-aquifer system comes mainly from the foothills of the Apennines as well as from the Po River basin (Figure 9.15.5) (Carbognin, et al., 1978). It is clearly impossible on the basis of the available records to quantify the respective contributions.

9.15.3 SUBSOIL RESOURCES EXPLOITATION AND SUBSIDENCE

It was soon quite clear that as in the Venice case the surface settlement was caused by the removal of subsoil fluids. Since the withdrawal rate is hard to assess with accuracy, the behaviour of the subsurface flow field was kept under periodic observation through a network of 120 piezometers (Figure 9.15.6). A 1972 survey of the area revealed that 877 active wells tapped the 9 confined aquifers. These wells were scattered across the area, but the most recent and productive ones were concentrated on the industrial zone (Bertoni, et al., 1973).

Figure 9.15.7 shows the behavior of the piezometric levels of the various aquifers underlying the historical center (Carbognin, et al., 1978). It is evident from this figure that:

- -- there was a lowering of the hydraulic head below the ground level beginning in the 1950’s;
- -- the greatest decline occurred after 1960, simultaneously with the development of the nearby industrial zone;
- -- aquifers 4 and 5 are the most intensively exploited;
- -- among the head gradients found in the aquitards the highest occurs between aquifers 3 and 4, with a difference of head of 22.50 m;
- -- in recent years the piezometric level tends to be constant;
- -- aquifers exhibit a somewhat independent hydraulic behavior (except perhaps aquifers 1 and 2). This is further evidence that the basin underlying Ravenna is a real multi-aquifer system.

Piezometric records permitted periodical plotting of equipotential lines. As an example, Figure 9.15.8 gives the piezometric surface in 1977 averaged over all the aquifers between 100 and 430 m (Carbognin, et al., 1978). It may be observed that the maximum drawdown of about 40 m occurs in the industrial zone (it was the same in 1972). Today, however, a large decline extends even

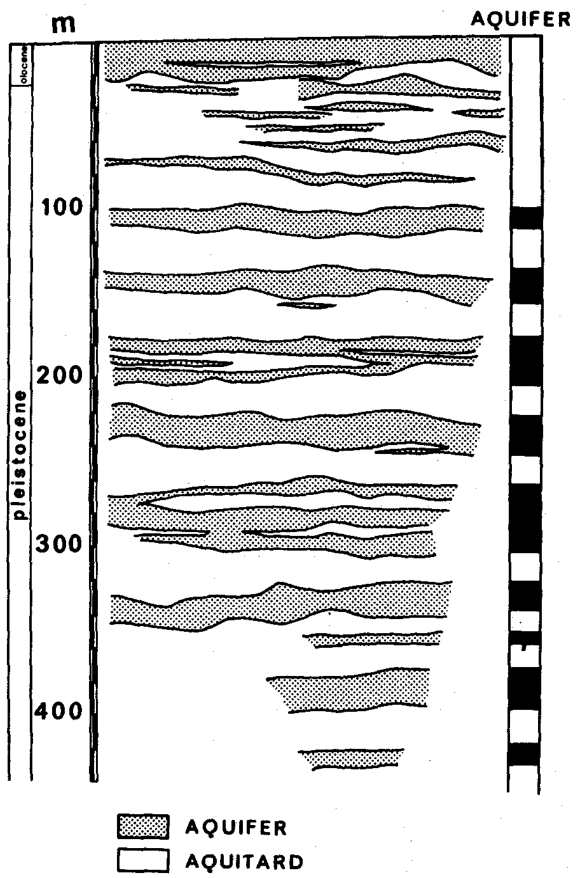


Figure 9.15.4 Schematic cross-section of the Ravenna aquifer system. (From Carbognin, et al., 1978, Figure 3; published with permission of the American Society of Civil Engineers.)

to the western and southern parts of the territory due to the increase of water withdrawn for agricultural uses, seaside resorts, and new industrial parks springing up on the outskirts of Ravenna.

The asymmetric cone of depression develops with its major axis from NW to SE, greatly affecting the coastline. A strong gradient appears in the southern part, corresponding to the direction of the Apennines recharge.

Between 1972 and 1977, the maximum decline of the piezometric head has not changed substantially (see Figure 9.15.7). Nevertheless, even if encouraging, this does not correspond to the arresting of land subsidence, as will be seen later.

So far as the geodetic survey of the area is concerned, it was not homogeneous in time. Although the land subsidence began in the early 1950’s, only since 1970 have land levelings been systematically carried out at the same time as the measurement of the piezometric levels. As an example, Figure 9.15.9 shows the subsidence experienced from 1972 to 1977. The general increase of the subsidence in these years is shown by the two maps of Figure 9.15.10. In the evaluation of the rate of subsidence linear trends are assumed. It may be noted that the area experiencing subsidence exceeding 3 cm/y in the latter period is about 30 times greater than the corresponding area in the former one. Moreover, a settlement rate exceeding 5 cm/y was experienced in the last few years (Figure 9.15.10b). The maximum rate of about 11 cm was recorded in the industrial zone between 1972 and 1973 and in Ravenna’s historical center about 8 cm was observed.

The shape of the subsiding areas is in close correspondence with the cone of depression of the aquifers in both periods. The time and space correlation between ground sinking and water withdrawals is clearly evidenced in Figure 9.15.11, which shows the average piezometric level and subsidence from 1950 to 1977 along a line crossing the city and extending to the country

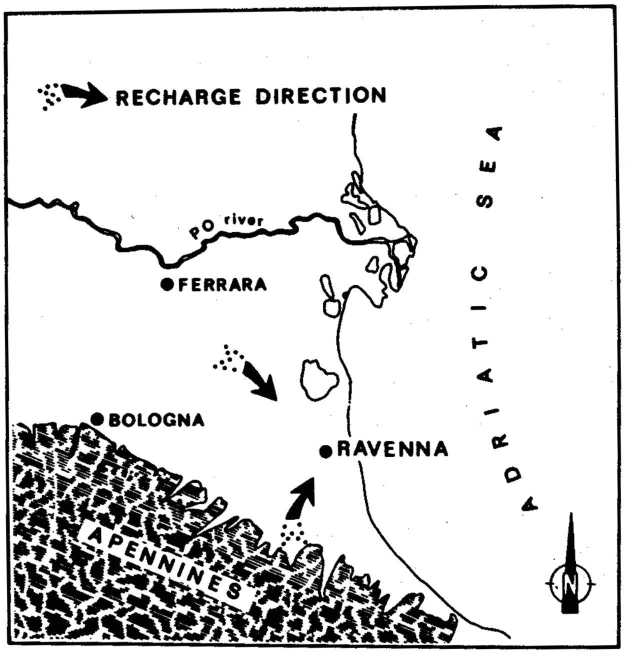


Figure 9.15.5 Map of the recharge areas of the Ravenna aquifer system. (From Carbognin, et al., 1978, Figure 4; published with permission of the American Society of Civil Engineers.)

side. This comparison stresses the nearly absolute behavioral identity of these parameters (Carbognin, et al., 1978).

From 1949 to 1977 maximum subsidence of about 1.20 m was recorded in the industrial zone, but in general and especially in recent years (1972-1977) the entire area has been affected at alarming rates. Bearing in mind that the ground elevation of 90 per cent of the land between the city and the coastline does not exceed 1 m above sea level and that 20 per cent of the latter is below mean sea level, the situation is becoming more and more serious.

In the past, the main cause of subsidence was wrongly ascribed to gas exploitation. The analyses carried out, though not precisely quantified, allowed us to estimate its effective contribution to the subsidence. With no doubt gas extraction from the natural deposits contributes in some zones to increase land settlement, but it has had limited effects. For instance, by superimposing the subsidence contour map of the period 1949-1972 on that of the gas reservoir of Ravenna Field, a good correspondence is observed between the area of the traps and area of the lines of equal subsidence, both being elliptic and with their major axes oriented in a NW-SE direction (Bertoni, et al., 1973) (Figure 9.15.12).

Likewise a comparison of land subsidence and the piezometric level recorded between 1949 and 1972 along a line crossing the Ravenna Field and industrial zone (Figure 9.15.13) shows a secondary local maximum, A, of subsidence corresponding to the location of the gas reservoir, but there is no corresponding piezometric decline [for which a minimum does not exist]. On the other hand, the maximum, B, of subsidence over the industrial zone corresponds to the maximum of drawdown. However, this gas reservoir is practically depleted and in 1972 its development had already achieved 95 per cent of the potential productivity: therefore the present contribution of gas withdrawal is probably negligible.

Unfortunately little is known about the more recent offshore gas exploitations and consequently it is impossible to say how much they influence the sinking of the coastal areas. This matter requires further investigation.

Among the man-induced causes of subsidence it must be remembered that marsh-land reclamation occurred on a large scale in this territory. Since the reclamation works were completed a long time ago (over 50 years), the contribution of the fill should no longer have any influence in the subsidence occurrence.

Natural subsidence gives a nonnegligible contribution in the overall occurrence. The bench mark of Porta Adriana in the historical center provides a useful indication to quantify this

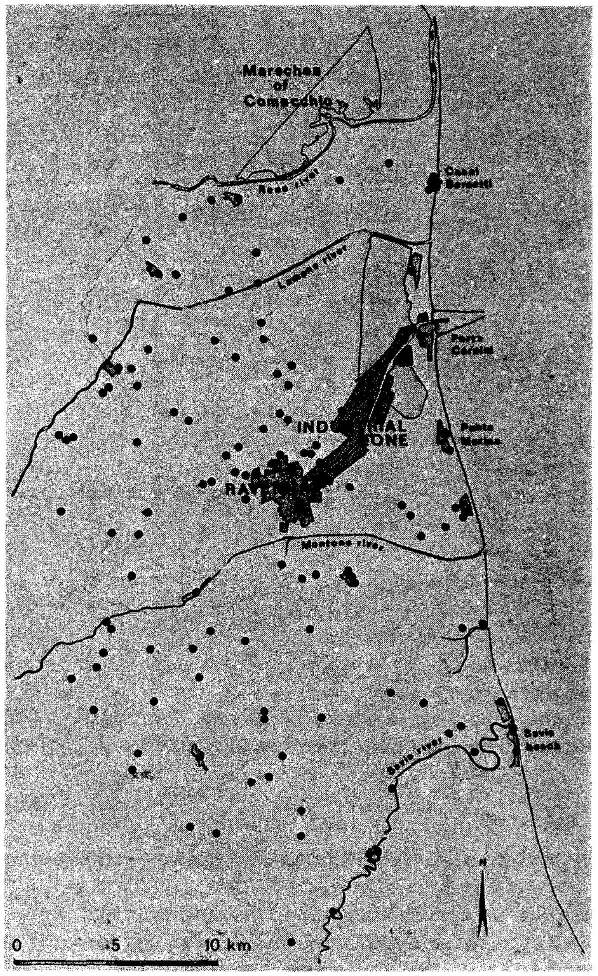


Figure 9.15.6 Map of the network of piezometers in the Ravenna area. (From Carbognin, et al., 1978, Figure 5; published with permission of the American Society of Civil Engineers.)

component since its elevation was recorded for the first time as early as 1902 (Figure 9.15.14). The data points of Figure 9.15.14 show that from 1902 to 1950 the subsidence rate was 5.14 mm/y (assuming as usual a linear trend in this period), while later on the rate has increased greatly due to the intensive exploitation of the subsurface resources. Since before 1950 water consumption was very small, the value of 5.14 mm/y may be considered as indicative of the geologic component of the subsidence in Ravenna.

To the present time the dominant factor of Ravenna subsidence is the intensive withdrawal of artesian water in the industrial zone, where the apex of the cone of depression is always found. The minimal piezometric levels reached in 1972 in the industrial zone have not changed but in spite of this additional subsidence occurred in the following years (Figure 9.15.15). This fact is partly explainable by a delay between the head declines in the aquifers and the resulting subsidence. As a second partial explanation it seems likely also that the maintenance of a very strong depression in the deepest aquifer over the last five years has introduced a secondary phenomenon of an upconing from the salt-water aquifers lying below 430 m, i.e., an irreversible pollution of the fresh-water system and a further compaction of the clayey soil aquitard. It is known in fact that some chemical variations of interstitial water in the clay soils can cause a change in the electrochemical equilibrium and therefore a collapse.

This contamination by salt water has been confirmed by the chemical analyses of the aquifer waters which evidence a progressive pollution in the industrial zone; this intrusion happened from the underlying saline water. In the nearby littoral, salt pollution of the same aquifer

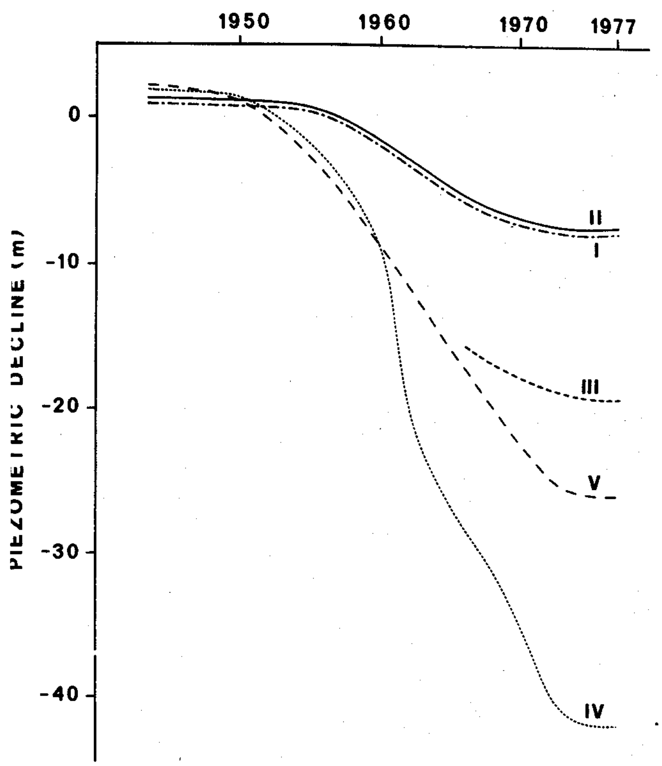


Figure 9.15.7 Piezometric levels from 1944 to 1977 of the various aquifers below the historical center of Ravenna. (From Carbognin, et al., 1978, fig. 6; published with permission of the American Society of Civil Engineers.)

occurred later but never reached the high values recorded in the industrial zone. In the coastal areas, salt water intrusion would also occur laterally.

As already shown in Figure 9.15.10b, the greatest sinking area after 1972 includes the coastline. The consequences are indeed very serious. In fact a striking regression of the shoreline and in some places the vanishing of the famous beaches of Romagna are the most severe effect of the sinking of the littoral. Not only coastal processes are responsible for it, as was believed before.

The following examples confirm the statement:

> Area of Lido Adriano: From 1957 to 1977 the regression of the shoreline has been 126 m. In the same period this zone has experienced a subsidence of about 45 cm. With a 4 per mill mean average beach slope (computed up to the isobath -8), the subsidence prevails on the process of beach regression (Figure 9.15.16).
>
> Area of Punta Marina: Between 1957 and 1977, the reported shoreline regression has been 70 m south of Punta Marina. The subsidence during these years has been 35 cm. Here the mean slope is around 4-5 per mill, and the beach regression is mostly attributable to the subsidence.

9.15.4 CONCLUSIONS

It is now clear that subsidence in the territory of Ravenna is mostly due to the intensive artesian water exploitation for industrial purposes, and, in more recent time, for agricultural uses. In some places the salt water intrusion has caused further compaction.

The exploitation of the gas reservoir of Ravenna Field has provided a minor local contribution to the subsidence; the possibility of a greater influence from the very active offshore gas fields is recognized and should be monitored.

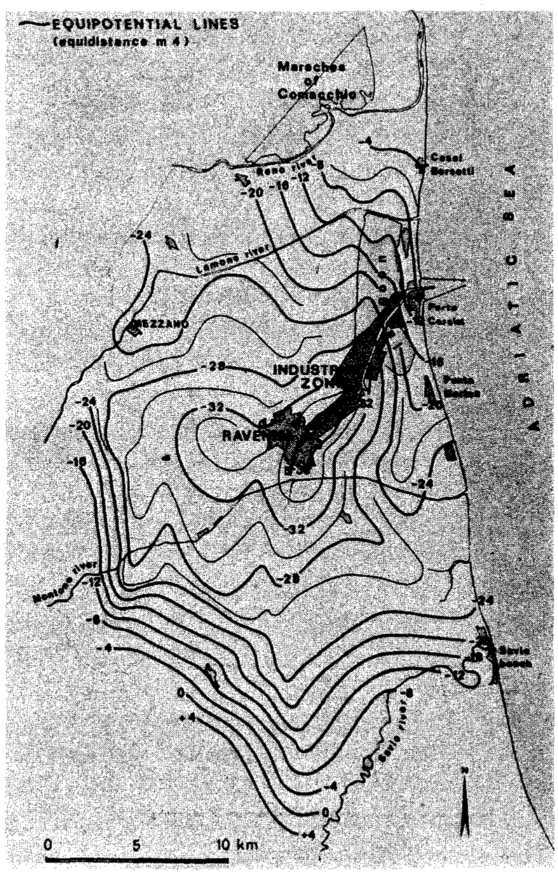


Figure 9.15.8 Average piezometric surface in 1977; datum is mean sea level. (From Carbognin, et al., 1978, Figure 7; published with permission of the American. Society of Civil Engineers.)

Since 1949 the average piezometric decline has nearly reached 45 m in the industrial area; correspondingly the average subsidence has been about 1 m. The close relationship between land settlement and water withdrawals has been clearly proven by the present analysis. Moreover if z indicates the land subsidence induced by man (i.e., the overall sinking minus geological component) and Δh is the piezometric decline expressed in the same units, we obtain a value $z/\Delta h$ approximately equal to 1/52. This result means that every 52 cm of withdrawal has produced 1 cm of subsidence. These values related to the environmental conditions place the Ravenna case among the more alarming in the world.

Apart from the values themselves, it is interesting to examine the trend of the occurrence. It is a matter of concern to find that while the subsidence still seemed quite localized around the industrial zone until 1972, it has assumed a broad increase since 1972. At present, the subsidence is affecting wide areas at a large rate and the related consequences are becoming highly critical for the survival of the whole physical and human environment.

The situation is very precarious along the littoral areas where a regression of the coastline over 150 m has been observed in some points. This threatens the most profitable industry of Romagna, i.e., the tourism.

The lands lying behind the coastal areas are in danger too. Bearing in mind that they lie at a height of less than 1 m above m.s.l., if the present trend is maintained for 10 years and if some sea storm would destroy the remaining dunes, 70 per cent of the territory between Ravenna and the beach (about 200 km^2) would permanently be inundated by the sea. Some urban zones, the industrial area, all harbor structures and several beach resort centers are in this part of the municipality. The damages would be incalculable. It is only a hypothesis, but not altogether unlikely.

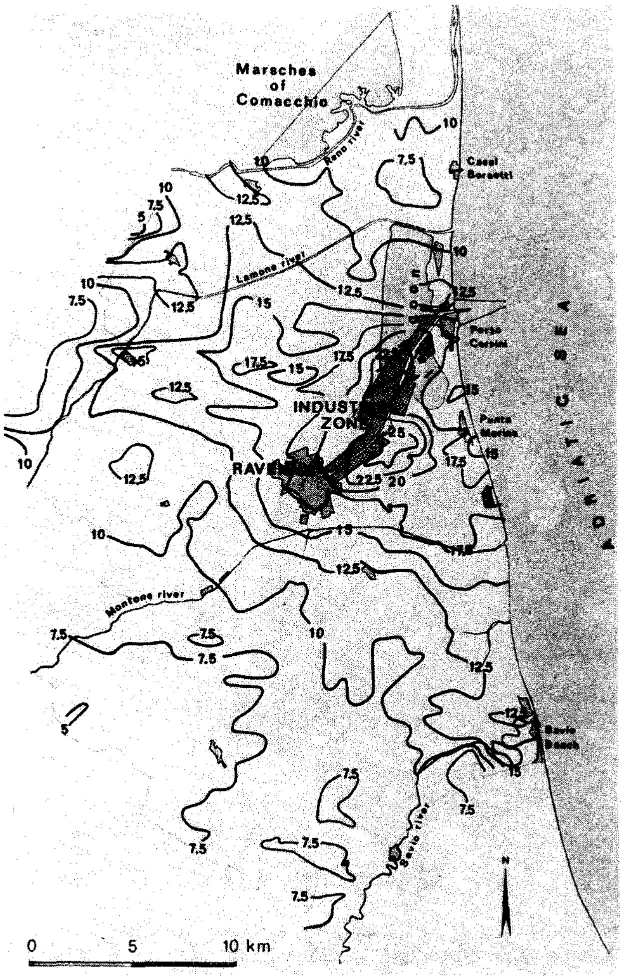


Figure 9.15.9 Land subsidence in the Ravenna area from 1972 to 1977, expressed in cm. (From Carbognin, et al., 1978, Figure 8; published with permission of the American Society of Civil Engineers.)

All this, however, is a simple projection of the present trend: a precise modeling is now in order. With enough information on physical and mechanical characteristics of the soils it would be possible to implement a mathematical simulation of the subsidence which would allow us to make real predictions on a long-term basis and understand the actual behavior of the system. In 1974 the authors (Carbognin, et al., 1974) suggested the necessary operations for investigating the knowledge on subsoil and improving the control of phenomenon evolution. In any case the subsidence control is today no longer achievable by local intervention, but only on a regional scale because of the vastness of the subsidence occurrence.

9.15.5 REFERENCES

AGIP MINERARIA. 1969a. "Relazione sull'incontro della Commissione veneziana con la Direzione Agip." Agip Mineraria, S. Donato Milanese.

__________. 1969b. "La pianura Padana-Veneta," in Italia, Geologia e ricerca petrolifera, Enciclopedia Petrologia e Gas Naturali, ENI, Colombo Ed., Milano.

__________. 1972. "Acque dolci sotterranee," Inventario dei dati raccolti dall'Agip durante la ricerca di idrocarburi in Italia, S. Donato Milanese.

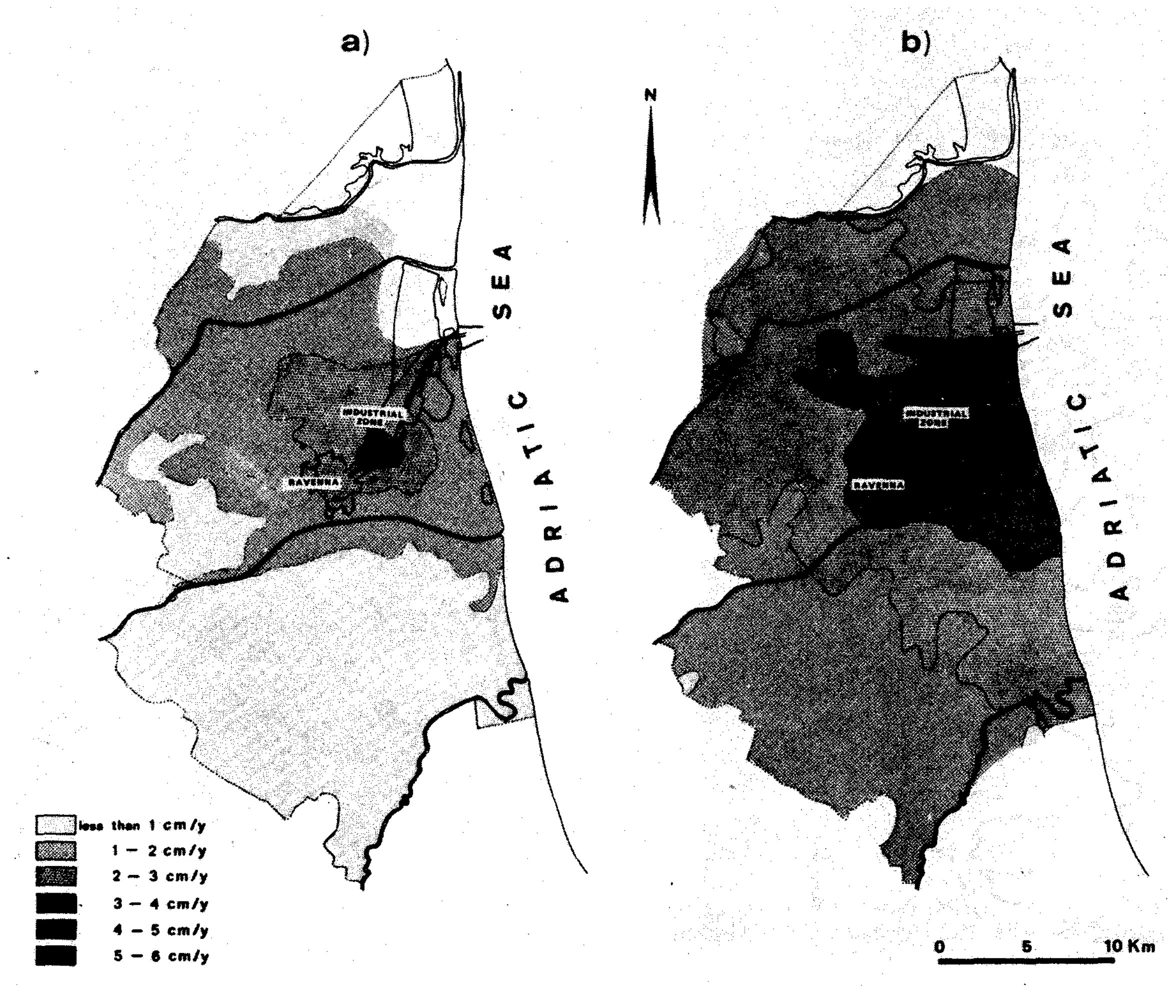


Figure 9.15.10 Space distribution of the subsidence rate between 1949-1972 (a) and 1972-77 (b). (From Carbognin, et al., 1978, Figure 10; published with permission of the American Society of Civil Engineers.)

BERTONI, W., L. CARBOGNIN, P. GATTO, and G. MOZZI. 1973. "Note interpretative preliminari sulle cause della subsidenza in atto a Ravenna," C.N.R., Lab. per lo Studio della Dinamica delle Grandi Masse, Tech. Rep. 65, Venezia.

CARBOGNIN, L., P. GATTO, and G. MOZZI. 1974. "Ricerca sulla subsidenza in atto nel Ravennate. Programma per la realizzazione della III e IV fase di studio e relativo piano finanziario," C.N.R., Lab. per 16 Studio della Dinamica delle Grandi Masse, Tech. Note 56, Venezia.

CARBOGNIN, L., P. GATTO, G. MOZZI, and G. GAMBOLATI. 1978. "Land subsidence of Ravenna and its similarities with the Venice case," Proceedings of the Engineering Foundation Conference on "Evaluation and Prediction of Subsidence," pp. 254-266, ASCE, New York.

DAL PIAZ, G. 1969. "Il bacino quaternario polesano-ferrarese e i suoi giacimenti gasseferi," Atti Convegno Giacimenti Gassiferi Europa Occidentale, Vol. 1, Roma.

SALVIONI, G. 1957. "1 movimenti del suolo nell’Italia centro-settentrionale," Boll. di Geo desia e Scienze Affini, I.G.M., anno XVI, Firenze.

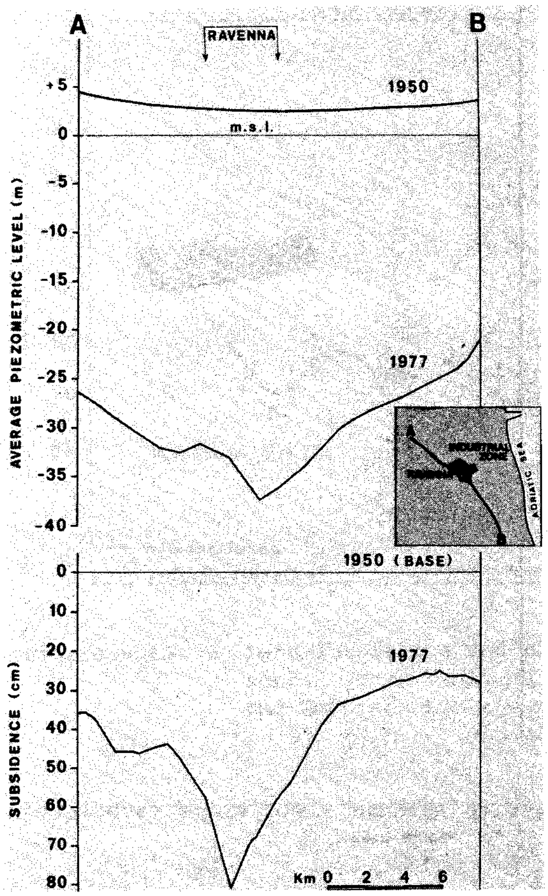


Figure 9.15.11 Comparison between the average piezometric level and the ground level over the city of Ravenna and its rural area. (From Carbognin, et al., 1978, Figure 9; published with permission of the American Society of Civil Engineers.)

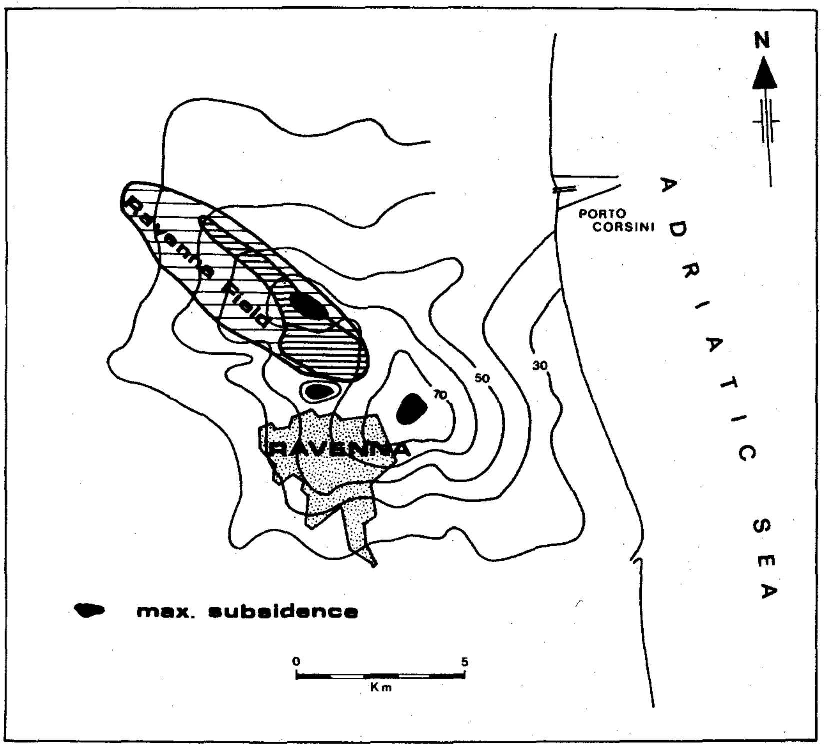


Figure 9.15.12 Comparison of development of sinking area with Ravenna Field traps (subsidence in cm).

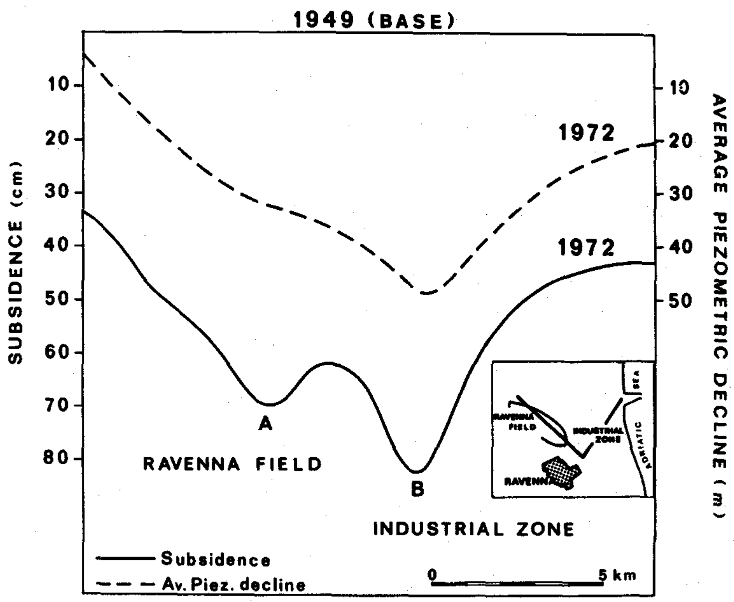


Figure 9.15-13 Land subsidence and piezometric level over the Ravenna Field and the industrial zone. (From Carbognin, et al., 1978, Figure 11; published with permission of the American Society of Civil Engineers.)

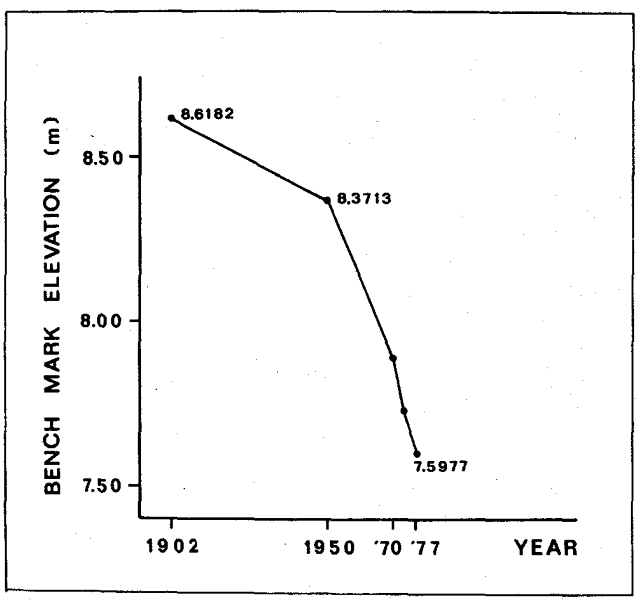


Figure 9.15.14 Elevation of bench mark of Porta Adriana (historical center) from 1902 to 1977.

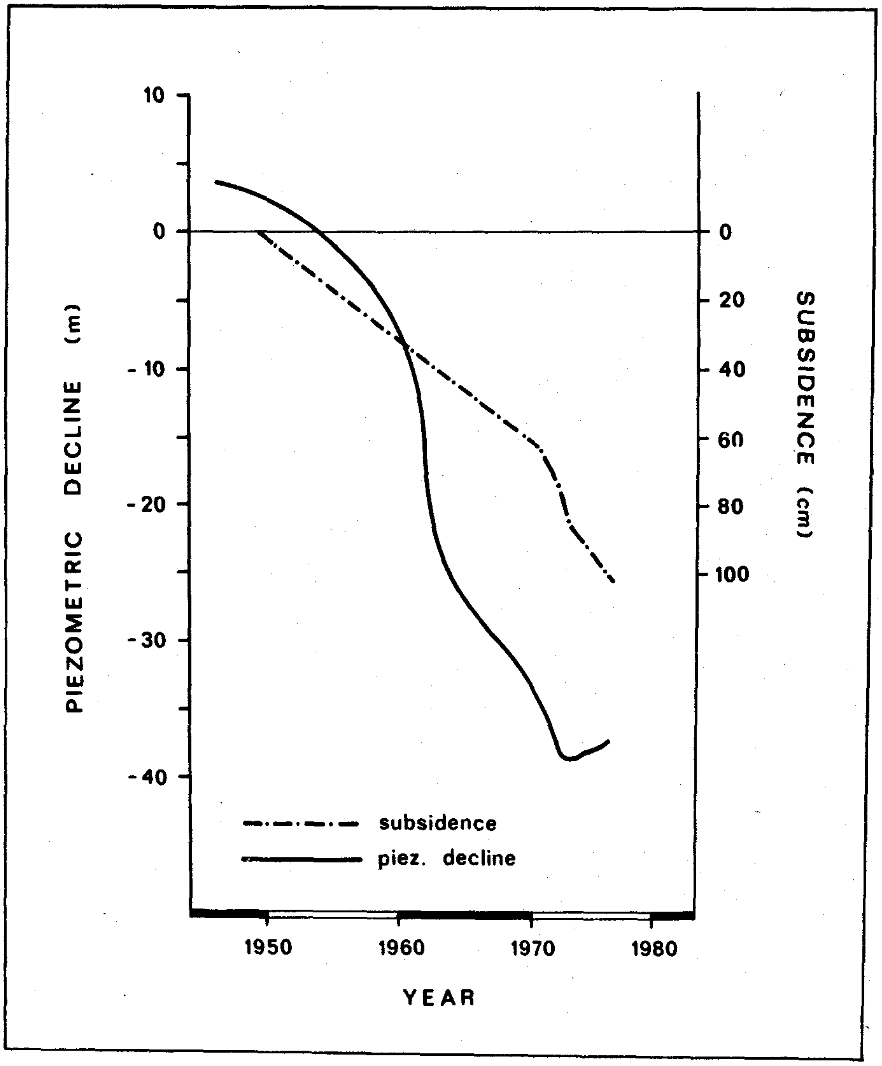


Figure 9.15.15 Comparison between subsidence and drawdown in the industrial area.

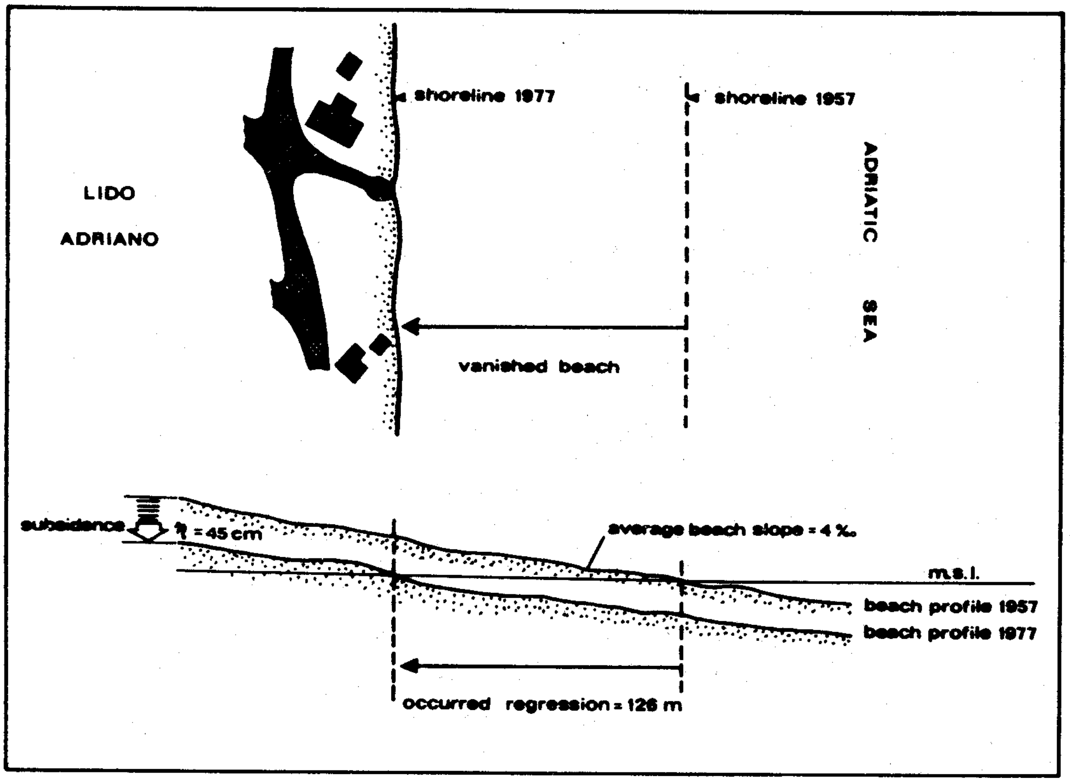


Figure 9.15.16 Schematic representation of the process of the beach regression at Lido Adriano area.