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ANALYSIS OF BEHAVIOR OF EXPANSIVE SOIL FOUNDATIONS

by

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The differential heave of foundation soil from moisture imbibition often causes major damage to overlying structures. Many methods have been proposed for estimating expansive soil heave, but there are no proven guidelines for evaluating the behavior of foundations on swelling soils. Foundations built on expansive soils are often designed without considering the swelling properties of the soil, resulting in unsightly cracking or structural damage to the completed building.

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20. ABSTRACT (Continued).

A practical method for evaluating the behavior of foundations on undisturbed expansive soil involves:

a. A site study.

b. Laboratory tests on typical samples.

c. Prediction of the total and differential heave of the foundation soil beneath various types of foundations.

d. Choice of a suitable and economical foundation design.

The site study will help define the relative vulnerability of the foundation soils to heave, indicate the soil strata that have the most likely potential for swell, and provide samples for laboratory tests. The type of foundation is usually controlled by the structural and architectural requirements and site and subsurface soil characteristics.

Reliable predictions of in situ heave and rate of heave are extremely difficult to obtain. Many factors are involved, such as soil composition, stratification, soil structure, fissures, maximum past pressure, degree of initial desiccation, climatic conditions, availability of water to the soil, lateral forces, surcharge pressure, time, and temperature. Reasonable estimates of heave can be derived from swell tests and appropriate deductions. The constant volume swell and modified swell overburden tests done on undisturbed soil specimens were found practical for approximating the effects of the aforementioned factors on foundations.

The computer code developed to expedite heave predictions contains this option: the final or equilibrium moisture condition may be either (a) a state of full saturation (an active zone with no negative pore-water pressures) or (b) an equilibrium based on a negative hydrostatic head increasing in magnitude with decreasing depth in the active zone. The active zone is defined as the depth of soil below ground surface subject to changing moisture conditions. Differential heave may be simply estimated, given the difference in heave predicted at different locations of the foundation.

Predicting the amount of heave is necessary for designing a heave-resistant structure. Soil-related design features include soil stabilization techniques and drainage. Structurally related design features include type of foundation (such as a reinforced slab or grade beam on piers), joints, diameter and depth of piers, and amount of reinforcing steel in the concrete.

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PREFACE

This behavior analysis of expansive soil foundations is one phase in a continuing study under the work unit "Properties of Expansive Clay Soils." The work unit was started in 1967 under the sponsorship of the Office, Chief of Engineers, U. S. Army, Directorate of Military Construction. The initial studies were performed under the U. S. Army Operations and Maintenance program. The studies are now being performed under RDT&E Work Unit AT04 04 001.

The work reported herein was performed by Dr. L. D. Johnson, Research Group, Soil Mechanics Division (SMD), Soils and Pavements Laboratory (SPL), U. S. Army Engineer Waterways Experiment Station (WES), and Mr. W. R. Stroman, Foundations and Materials Branch, U. S. Army Engineer District, Fort Worth. The report was reviewed by Messrs. R. W. Cunny, W. C. Sherman, Jr., Drs. E. B. Perry and D. R. Snethen, Research Group, SMD, Dr. D. M. Patrick, Engineering Geology and Rock Mechanics Division, S&PL, and Mr. C. L. McAnear, Chief, SMD. Mr. J. P. Sale was Chief, S&PL.

COL G. H. Hilt, CE, was Director of the WES during the conduct of this study and the preparation of this report. Mr. F. R. Brown was Technical Director.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres
square feet per day	0.09290304	square metres per day
pints (U. S. liquid)	0.4731765	cubic decimetres
pounds (mass)	0.4535924	kilograms
pounds (force)	4.448222	newtons
tons (mass)	907.1847	kilograms
tons (force)	8.896444	kilonewtons
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	47.88026	pascals
tons (force) per square foot	95.76052	kilopascals
atmospheres (normal)	101.325	kilopascals
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
degrees (angle)	0.01745329	radians

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

ANALYSIS OF BEHAVIOR OF EXPANSIVE SOIL FOUNDATIONS

PART I: INTRODUCTION

Background

1. Soils that show strong swell and shrinkage characteristics under changing moisture conditions exist in many areas of the world.[1] Such expansive soils within the United States are most commonly recognized in the western and southern states. Swelling soils damage many structures including pavements, walls, and foundations of houses and other buildings, canal and reservoir linings, and retaining walls.[2-6] It is estimated that property losses caused by expansive soils exceed two billion dollars annually.[6-8]

2. The presence of structures often induces heave in expansive clays because the natural transpiration of moisture by vegetation and evaporation is inhibited.[9-12] The amount of heave depends primarily on climatic conditions such as the amount and frequency of rainfall, the water table depth, and the thickness and other characteristics of the clay. Shrinkage, particularly along the perimeter of the structure, can occur during drought seasons.

3. Differential rather than total movements of the foundation soils are generally responsible for major structural damage. Differential heave may be caused by variations in thickness of the clay strata, soil permeability, soil water content, and other soil properties between the center and perimeter of a structure. Variations in soil water content may result from environmental conditions such as rainfall, local watering of grass and other vegetation, broken water and sewer lines, transpiration of moisture by trees, and evaporation of water from soil adjacent to heated areas within the structure. The differential heave can range from zero to the maximum total heave, but is typically between one-quarter and one-half of the total heave.[11,13,14] Differential heave is often the maximum total heave for structures supported on isolated spot footings such as drilled piers.

4. Soils characterized by strong swell or shrinkage commonly contain significant quantities of highly plastic and colloidal clay minerals largely composed of montmorillonite. Most montmorillonites carry the calcium ion as the most abundant exchangeable ion, while a few such as the Wyoming bentonite carry sodium as the dominant ion.[15] Bentonite is a sedimentary material containing appreciable montmorillonite derived from altered volcanic ash. Other less expansive clay minerals in order of decreasing potential for swell are illite, attapulgite, and kaolinite, with kaolinite being relatively nonexpansive.[16] Soils containing clay minerals with less expansive properties than bentonites may also swell significantly under certain field conditions and lead to damages in structures. For practical purposes, Atterberg limits provide a convenient indicator of potential expansion.

Purpose and Scope

5. The prediction of heave behavior for foundations on expansive soil based only on cursory observation and local experience with inadequate consideration of soil characteristics in many cases leads to extensive structural damages. Designs of relatively small structures such as houses and one-story buildings are usually based on the least consideration for potential soil swell.

6. The design of adequate foundations for structures in expansive soil areas should be based on a thorough understanding of such factors as the in situ behavior of the foundation soils, initial groundwater conditions, soil stabilization and drainage techniques, and foundation types suitable for expansive soil subgrades. A realistic approach to achieve this understanding is to: (a) conduct a thorough site study; (b) predict the in situ heave behavior of the expansive soils from results of laboratory swell tests; and (c) compare alternative foundation designs to determine the most suitable and economical design compatible with or adequately resistant to the predicted heave.

7. This report provides guidance on features that should be examined during site investigations and provides guidance on predicting

in situ heave of foundation soils. A computer code, developed to expedite heave predictions for a variety of final moisture and loading conditions, is explained. Some applications of heave predictions to foundation design and various remedial and construction procedures are outlined. The report is essentially limited to analyses of volumetric behavior of undisturbed foundation soils from imbibition of moisture. Analyses of other sources of heave, such as chemical alteration and frost heave, are not included in this report.

PART II: SITE STUDIES

8. The analysis of the swelling behavior of expansive soil foundations should begin with a study of the construction site conditions. Site studies include an evaluation of soil strata behavior, existing structures, climate, and initial groundwater conditions. Borings should be made to provide undisturbed soil samples for identification and swell tests.

9. Figure 1 illustrates approximate locations of clays and shales within the continental United States that may exhibit swell or shrinkage from changing moisture conditions. The distribution of expansive materials was based on the degree of expansiveness and the expected occurrence frequency of the expansive materials. The premises that guided selection of the degree of expansiveness are:

a. Any area underlain by argillaceous rocks, sediments, or soils will exhibit some degree of expansiveness.

b. The degree of expansiveness is a function of the amount of expandable clay minerals present.

c. Generally, the Mesozoic and Cenozoic rocks and sediments contain significantly more montmorillonite than the Paleozoic (or older) rocks. (Damage to structures founded on Permian (Upper Paleozoic) has also been observed.)

d. Areas underlain by rocks or sediments of mixed textural compositions (e.g., sandy shales or sandy clays) or shales or clays interbedded with other rock types or sediments are considered on the basis of geologic age and the amount of argillaceous material present.

e. Generally those areas lying north of the glacial boundary are nonexpansive due to glacial drift cover.

f. Soils derived from weathering of igneous and metamorphic rocks are generally nonexpansive; Worrall[17] indicates montmorillonite may be a weathering product of hornblende, pyroxene, and olivine, but these deposits are usually thin and not extensive.

g. Climate or other environmental aspects are not considered.

h. Argillaceous rocks or sediments originally composed of expandable clay minerals do not exhibit significant volume change when subjected to tectonic folding, deep burial, or metamorphism.

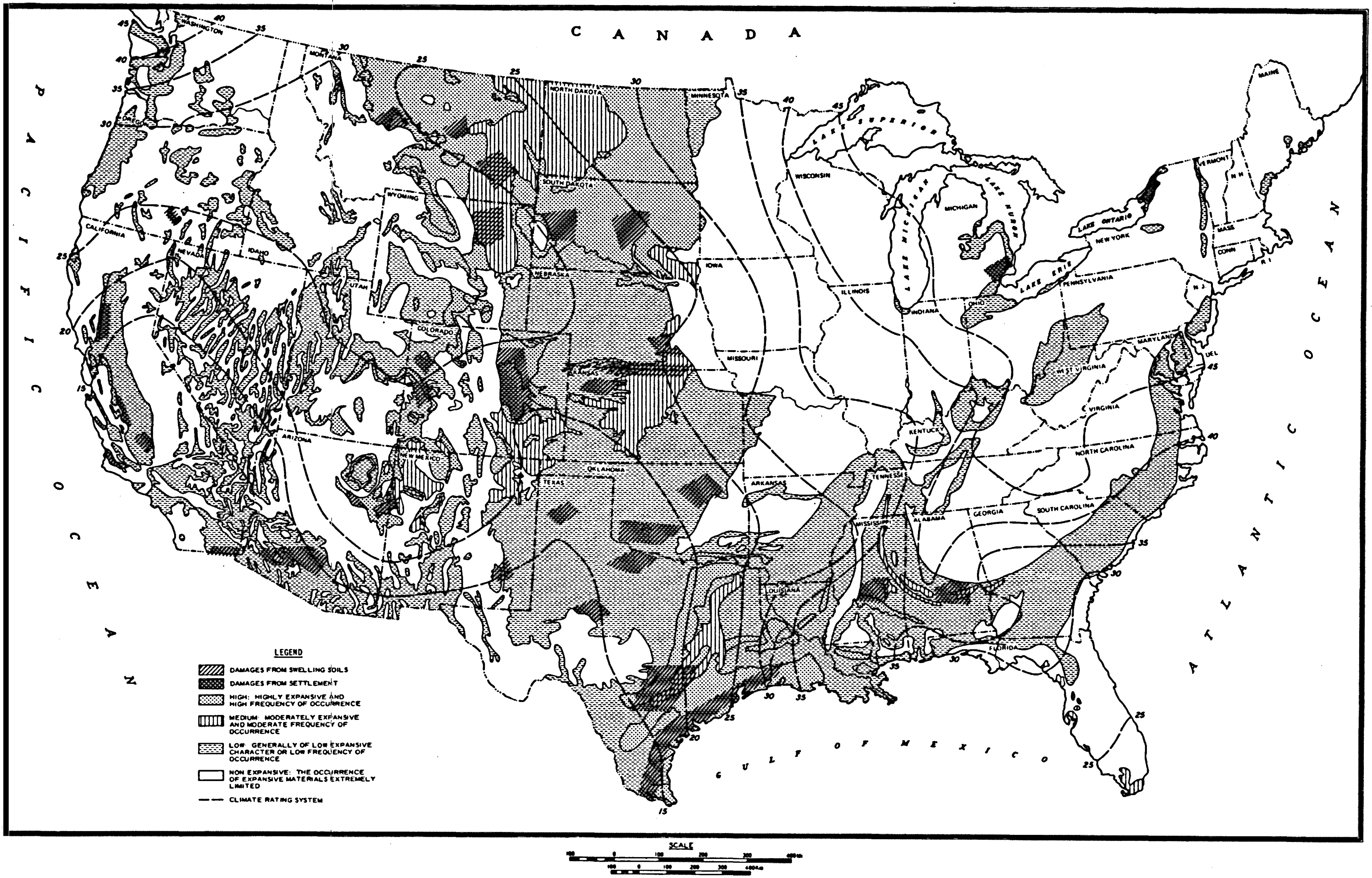

Fig. 1. Locations of expansive clay soil problems

i. Volcanic areas consisting mainly of extruded basalts and kindred rocks may also contain tuffs and volcanic ash deposits which have devitrified and altered to montmorillonite.

j. Areas along the glaciated boundary may have such a thin cover of drift that the expansive character of the materials under the drift may predominate.

The selections for the expected frequency of occurrence were guided by published literature that revealed actual problems or failures due to expansive materials, such as materials maps, soils surveys, and geologic maps and cross sections. Further details of the derivation of the distribution of expansive materials may be found in Reference 18.

10. Superimposed on Figure 1 are: (a) areas where damages have occurred to civilian and military structures from swelling soils;[7,19-21] and (b) a climatic rating system discussed subsequently.[22] Structures constructed in areas denoted with swell potentials of high, medium, and low degrees of expansiveness have been damaged by swelling soils. Figure 1 is meant to show only general trends in swell potential with location; delineations shown between high, medium, and low degrees of expansiveness may not be reliable on a local scale. Damages are most common where climatic ratings are less than 25. However, some areas with a milder rating of about 35, such as Mississippi and Alabama, also have structures damaged by swelling soils. These most vulnerable areas, the southern and central United States, appear in the semiarid and temperate climatic zones, which are favorable to the formation of montmorillonitic soils.[1] Most montmorillonites, however, could have been in place long before the present climatic conditions. Montmorillonites are generally formed from the chemical weathering and diagenesis of volcanic ash.

11. Site studies should be made prior to final design and construction. These are especially recommended in the southern and central United States in areas denoted with low or higher degrees of expansion (Figure 1), and should include estimates of potential heave beneath foundations of major structures. Soil exploration programs for relatively small structures such as houses and one-story buildings may also

be economically significant if construction is planned in the vulnerable areas of Figure 1 and, especially, if a study of the site history and adjacent structures indicates that the proposed building may be subject to damaging heave.

Site History

12. A study of the site history may reveal considerable qualitative information on the probable future behavior of the foundation soils. Maps of the proposed construction site should be examined to obtain information on wooded areas, existence of earlier buildings, ponds and depressions, and watercourses. Removal of trees eliminates an efficient source of evapotranspiration, and the foundation soils may subsequently heave from the accumulation of moisture. Ponds and depressions are often filled with clay sediments accumulated from the drainage of rainwater, particularly sediments of the ultrafine-grained soils (montmorillonite) because of the ability of running water to transport small sized particles.

13. Other construction in the vicinity should be inspected closely to determine past performance and present condition. Structures similar to the proposed building should be especially inspected. The condition of on-site stucco facing, joints of brick and stone structures, and interior plaster walls is a fair indication of the possible degree of swelling. The amount of differential heave exerted on a masonry structure may be estimated by summing the crack widths in the structure. The differential heave that may occur in the foundation soils beneath the proposed structure is not necessarily equal to the differential heave of nearby structures; differential heave depends on local site field conditions such as load distribution, foundation depth, and change in groundwater since construction of the earlier structure.

Climate

14. The climate has a strong influence on the magnitude of heave that may occur after placement of a structure on an expansive clay

soil.[22-25] In areas where shallow water tables do not exist, moisture conditions in the soil are controlled by the moisture balance between rainfall and evaporation.[26] Changes caused by construction are almost certain to upset the original moisture distribution, and a new moisture equilibrium will be established.

15. The climatic rating system indicated in Figure 1 was established by consideration of five meteorological variables.[22,23]

- a. Annual precipitation.
- b. Degree of uniformity in distribution of precipitation.
- c. Number of times precipitation occurs.
- d. Duration of each occurrence.
- e. Amount of precipitation during each occurrence.

The effect of temperature and relative humidity on evapotranspiration is assumed to be of secondary significance. Smaller climatic rating numbers, c_w (Figure 1), represent more unfavorable climates,[23] as shown below:

c_w	Description	Variation in Normal Precipitation	Maximum Period of Drought weeks
45	Favorable	Small	4
35	Intermediate	Moderate	6
25	Unfavorable	Considerable	6 to 12
15	Extremely unfavorable	Large	Over 12

The maximum period of drought in the above tabulation is the probable maximum period during the life of the structure.

Soil Exploration

Initial geological survey

16. Local geological records and publications should be consulted, preferably by an engineering geologist, prior to the sampling operation to obtain and assess information on general foundation conditions at the proposed site. Such information is available in Federal, state, and

institutional surveys, and may also be obtained from the Federal Highway Administration.

17. Soil exploration is performed as a step in determining solutions to the design of foundations for structures and in determining potential construction problems. Representative disturbed and undisturbed samples are obtained, following the initial geological survey, for visual inspection of the soil profile at the construction site and for use in laboratory tests to determine the soil classification, swell or consolidation behavior and bearing capacity of the foundation soil. The undisturbed borings should preferably be 5 in.* or more in diameter. This size will provide suitable specimens for laboratory swell tests performed in the one-dimensional consolidation frame (hereafter referred to as consolidated swell (CS) tests).

Time of sampling

18. Ideal moisture conditions in samples for CS tests should be identical to the moisture conditions of the foundation soil at the start of construction or placement of the foundation for the structure. The soil exploration program, to be of value, must be completed before the final design and initiation of construction and, therefore, moisture conditions may not be exactly duplicated above the depth of seasonal influence. Moisture conditions below the depth influenced by the weather season (10 feet or more below ground surface as discussed subsequently) will not be affected, and CS test results on deeper specimens will not be dependent on the time of the sampling operation.

19. Reasonable simulation of moisture conditions above the depth of seasonal influence might be achieved by timing the sampling operation to be similar to the time that construction is scheduled to begin for long-term construction. To minimize heave after construction, construction may be timed at the end of the rainy season when surface moisture is greatest. Samples may be taken during the dry season when potential heave will be maximum for conservative design. If the structure is

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 5.

constructed immediately following a rainy season, potential heave will tend to be minimal.

Sampling techniques

20. Auger or split spoon samples are often used for visual inspection and determination of water content, grain size, and Atterberg limits. Augering, however, distorts the soil stratification and may alter the soil water content. The water content may be increased if water is added to the borehole or extraneous water from other sources is allowed to reach the sample. Undisturbed samples are needed for accurate visual inspection, water content determinations, and laboratory consolidation, swell, and strength tests.

21. Undisturbed samples are usually obtained in thin-walled or Shelby tube samplers and various piston samplers up to a 6-in. diam. Dry borings above the water table are preferred to boring with a drilling fluid, which could cause changes in sample water content. Boring without drilling fluid is often possible in relatively soft cohesive clay soils by pushing thin-walled samplers. The undisturbed samples are often taken immediately from the sampler, placed in containers such as a 6-in.-diam cardboard cylinder, and sealed with a mixture of paraffin and microcrystalline wax.[27] The temperature of the melted wax should be as low as possible to avoid driving moisture from the soil sample. Expansive soil samples are often fissured. To avoid penetration of the wax into the fissures, samples should be wrapped with thin foil, cheesecloth dipped in wax, or plastic, prior to submerging in wax. A thin coating of wax may be brushed on the sample before wrapping to promote a better seal against moisture loss. The outer perimeter of the sample should be trimmed ~~during preparation of specimens for laboratory tests~~, leaving the more undisturbed inner core.

22. Continuous undisturbed samples should be obtained to determine a complete, detailed picture of the soil profile. The depth of exploration and sampling should extend well below the active zone for heave; i.e., to depths below ground surface of at least 1.5 times the width of the structure and at least 10 ft or more below the base of the foundation footings. The active zone for heave is that depth of soil

(below the ground surface and below the footings of the foundation) subject to changing moisture conditions. The active zone for heave is generally limited to the top 8 to 10 ft of soil,[26,28-30] but can extend deeper.[31-36] An active zone for heave may also penetrate beneath the foundation footings due to infiltration of moisture down the footing walls or piers and into the soil-footing interface.[37-40]

23. The number of borings should be sufficient to permit an adequate estimate of the lateral variations in the soil profile. Fewer borings may be needed to satisfy this requirement if the visual inspection and laboratory tests of boring samples from earlier nearby construction projects show an essentially uniform soil profile. A spacing of 25 ft is usually adequate even for erratic conditions.[41] Large diameter holes and/or trenches are particularly useful for detailed examination of the soil profile.

Sample disturbance

24. Truly undisturbed samples are not possible because boring and removal from its field position alter the condition of the sample. The effects of the sampling operation on sample disturbance were described in detail by Hvorslev[27,41] and they are summarized below.

25. Boring in soft soils. The advance of the borehole and removal of the displaced soil will reduce the normal stresses below the bottom of the hole within a certain zone (bulb) in the soil of about three or more times the diameter of the hole. Large reductions in stress during the boring of deep holes may permit plastic flow of the soil and can cause the soil below the bottom of the hole to be deflected upwards and seriously disturbed. The greatest reduction in stress occurs when the sampler is withdrawn, creating a vacuum below the sampler. The soil within the bulb of reduced stress has a tendency to swell, especially if water is in the hole. Small amounts of water can cause the rate and amount of swelling to be maximum. The sample should be taken immediately after the advance and cleaning of the borehole to minimize progressive swelling.

26. Advancing the borehole by displacing or pushing the soil aside will cause a bulb of increased stresses and downward deflection of

the soil layers below the bottom of the hole. The upper part of the sample will have a concave distortion of soil layers and shear failures for a distance of two or three times the diameter of the hole. Similar conditions may be caused by overdriving the sampler during the previous sampling operation or by advancing the casing ahead of the borehole. The inside wall friction from an advancing casing increases rapidly and forms an immovable plug of soil that soon causes stresses to increase below the casing. Cleaning the casing will partially reverse the stress conditions and reduce compaction and consolidation of soil within the bulb of increased stresses; the stress reversal may cause further disturbance of the soil structure.

27. The borehole should be cleaned before taking each sample to remove pebbles and settled material that could contaminate the sample. Pebbles and stones may damage the sampler or be caught on the cutting edge and partially disturb the entire sample. Methods and equipment for drilling a borehole are often used to clean the hole, but special equipment is usually required when open drive or thin-walled samplers are used. Less thorough cleaning or no cleaning is necessary when piston samplers are used in uncased parts of a borehole, but the sampler should be pushed through disturbed material before the piston is released and sampling begun. Cased boreholes should be cleaned to the edge of the casing since disturbed material in the casing cannot be laterally displaced and will be pushed ahead of the sampler to disturb the soil to be sampled.

28. Forces during driving and withdrawal of the sample, entrance of excess soil, inside and outside wall friction, and pressure over the sample all contribute to soil disturbance. The inside wall friction is the most important single source of soil disturbance during the sampling operation. The thin-wall Shelby tube of hard drawn seamless steel (or brass for softer soils) is simple to use and its small area ratio (area of the annular wall divided by the area enclosed by the annular wall) causes minimum sample disturbance. However, the tubing is easily damaged in hard soils and should be used only once. Coating the tubing with lacquer to keep it clean and smooth is desirable since it reduces

wall friction and prevents corrosion during shipment and storage.

29. Piston samplers are preferred for undisturbed sampling, especially when the soil is soft and the borehole is uncased. The lower end of the sampling tube of a piston sampler is closed with a piston that is released or withdrawn when sampling. The piston prevents shavings from the walls of the borehole and disturbed soil at the bottom of the hole from entering the sampler. The closed sampler can be forced into the undisturbed soil until the desired sampling depth is reached. The piston is effective in reducing pressure over the sample during withdrawal and helps reduce sample loss. Thin-walled piston samplers can be built to minimize sample disturbance.

30. Boring in hard soil. Highly expansive materials are, when in a desiccated state, too hard for efficient push-tube sampling procedures. Rotary core barrels are then used for sampling in these hard soils and brittle clays as well as dense, cohesionless, and partially cemented soils. A coring bit is rotated to cut an annular groove or kerf with sufficient inside and outside clearance for passage of the drilling fluid pumped through the drill rod. The pulverized material is removed by the circulating fluid. Double tube core barrels, consisting of an outer barrel with a cutter shoe to advance the sampler and an inner barrel with a cutter edge to fine-trim and contain the sample, are commonly used in sampling and when the diameter of the core is small to protect against the action of the circulating fluids. Less disturbed samples can be obtained by means of double tube core barrels with bottom discharge and an inner tube extending very close to or, in erodible soils, a little below the coring bit. To avoid disturbance of the soil below the core and entrance of excess soil, the feed pressures should be small at the start of coring and increase with increasing depth of penetration.

31. When drilling fluid is used, the surfaces of the samples are exposed to water from the drilling fluid. Before the sample is sealed with wax the surface should be scraped to remove moisture from the drilling fluid and prevent its migration toward the drier central core of the sample. The extraneous moisture can alter the natural moisture deficiency, particularly in fissured soils. The normal 6-in.-diam core

sample is large enough to provide adequate specimens after the wetted surface material is removed. One-quarter to one-half inch may be safely trimmed from the core sample. The danger of moisture penetration might be avoided if an auger core barrel could be used in a dry borehole; however, this type of core barrel has not been adapted for use in deep boreholes.

32. Removal of the undisturbed sample from the sampler may relieve stresses, especially in overconsolidated clays and shales, leading to additional fissures and sample deterioration. Gases may come out of solution in the pore water and cause partial disturbance of the soil structure. Moreover, fissured and stiff soils are extremely difficult to trim and require much hand labor. Samplers are available which push the sample into a liner during the sampling operation. The liner contains the sample to prevent the relief of lateral stresses and the liner can be inserted directly into the consolidometer assembly, eliminating the need for laboratory trimming. Sample disturbance may still exist near the perimeter, however, due to sampler friction and soil displacement by the sampler. The ends of the sample for a distance of one to two diameters should not be used for undisturbed specimens in laboratory tests. Shock and vibration during transportation of the samples should be avoided to further reduce disturbance.

Groundwater conditions

33. Groundwater conditions should be evaluated during the soil exploration program by making careful observations in boreholes and installing piezometers. A perched water table may exist in a granular soil overlying a relatively impervious and moisture-deficient clay soil, especially if the area is part of a depression or syncline. Perched water tables may cause heave if holes are bored through the perched table down through the moisture-deficient soil. Heave may also result if the foundation is below the perched table without taking measures to inhibit the migration of moisture into the deeper moisture-deficient zones. The distribution of the hydrostatic head in normal and perched water tables is determined by piezometric installations at different depths. Casagrande (porous tube) piezometers with small diameter risers

are usually adequate and they are relatively simple, inexpensive, and good for soils of low permeability.[42] All boreholes should be filled and sealed with a proper grout, such as a 12 percent bentonite and 88 percent cement mixture, to prevent penetration of surface water or water from perched tables down to the deeper strata that may include moisture-deficient expansive clays.

Identification of Expansive Soil

34. An expansive soil can be identified by the potential of the soil to swell independently of field conditions such as water content and surcharge pressures. The potential for swell depends on: (a) the amount and type of clay minerals; (b) soil structure, such as particle arrangement, bonding, and fissures; and (c) nature of the pore fluid and exchangeable cations. The type and amount of clay minerals, pore fluids, and exchangeable cations influence the amount of water that may be attracted into pores and clay mineral platelets. The soil structure can restrict platelet swell due to moisture imbibition and influences the amount and orientation of platelet or particle swell in the mass soil.

35. The most effective methods for identifying an expansive soil on the basis of composition and swell behavior are: (a) mineralogical, (b) soil classification, (c) physical, and (d) chemical analyses.[18] Mineralogical analyses using X-ray diffraction methods can provide information on the amount and type of clay minerals. Soil classification analyses aid in the determination of the amount and composition of clay minerals and help empirically evaluate the relative magnitude of swell on imbibition of free water; soil classification tests do not consider effects of structure. Physical analyses using X-radiography show promise for evaluating the magnitude of fissures and their effect on swell; electron microscopy can provide information on platelet arrangement and fabric. Particle bonds cannot yet be physically observed in soils, but may be functionally understood by the shape of the strength envelope derived from unconsolidated-undrained triaxial tests. CS tests can directly indicate swell potential on imbibition of free water for

specified restraining pressures, initial water contents, and, if the sample is remolded, compactive effort. Chemical analyses can indicate the nature of the pore fluid and the exchangeable cations.

36. Properly combining the above techniques will provide for approximate identification of the swell potential of expansive soils. However, further research techniques such as X-ray diffraction, X-radiography, electron microscopy, chemical analysis of the pore fluid, and CS tests are needed to: (a) achieve a better understanding of swell behavior, (b) quantitatively evaluate swell potential, and (c) establish the relative usefulness of these various tests in practical and economical identification of swell potential. X-ray diffraction tests, for example, are very useful for fast, positive identification of clay minerals, but work is needed to relate composition to various degrees of potential swell. Many of these tests are time-consuming and require expensive equipment with skilled personnel to conduct the tests and interpret the results. These techniques, except for CS testing, are frequently unavailable in local soil mechanics laboratories. For these reasons, Atterberg limits data are commonly used to provide an initial, but quick and useful, estimate of the potential of the foundation soil expansion.

37. Experience[18,43,44] has shown that both lean (CL) and especially fat (CH) clays have expansive characteristics, and that swell correlates to some extent with plasticity index (PI) and liquid limit (LL) data. Plasticity properties of swelling clays typically fall within a band of the plasticity chart below the U-line and above the A-line (Figure 2). Some silty clays with expansive properties were found to have plastic characteristics that fell slightly below the A-line.[45]

38. A very simple and inexpensive method of identifying the swell potential, such as the Dakshanamurthy and Raman[44] (D&R) or Seed et al.[43] (SEED) method, may be reasonably practical simply because more exact methods are not economically available. The D&R method is based on division of the LL horizontal coordinate of the plasticity chart into different degrees of expansion (low, medium, high, very high, and extra high) by vertical lines (Figure 2).

39. The swell potential of some natural soils may also be

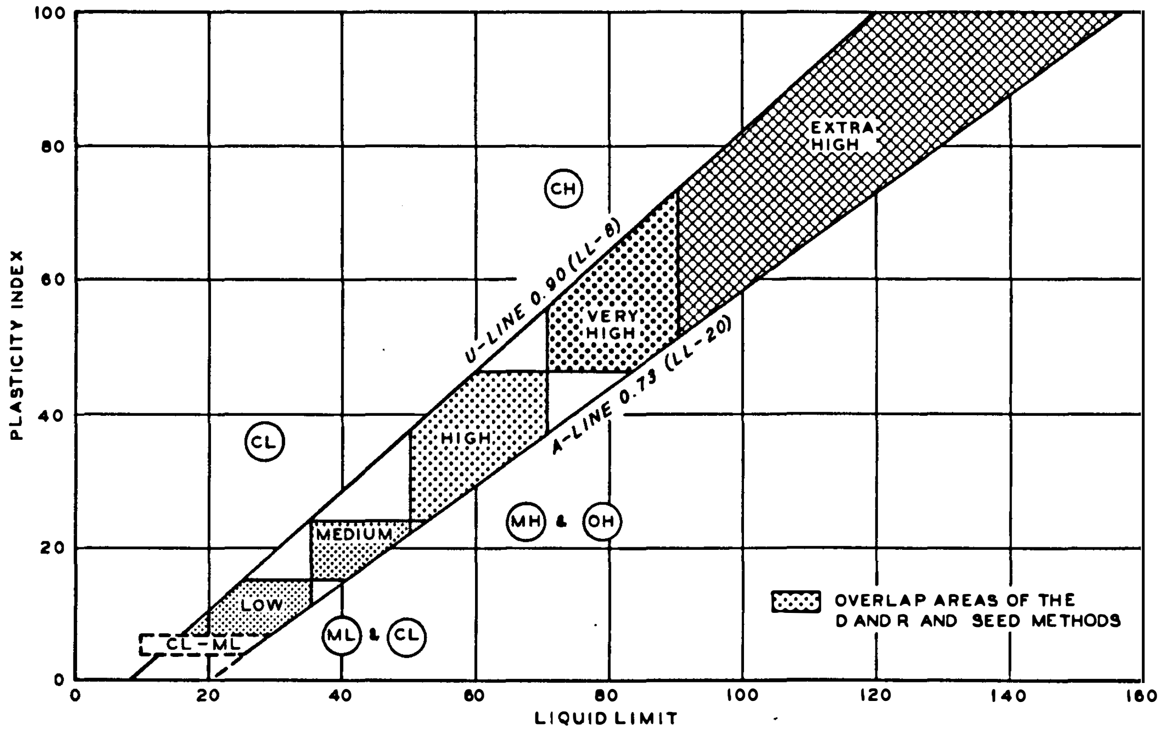

Figure 2. Degree of expansion in expansive soils on the plasticity chart

estimated within 33 percent of the laboratory-determined swell potential for clay contents between 8 and 65 percent (particles less than 2 μm) by the SEED[43] method:

$$S = 0.00216(PI)^{2.44} \text{*} \qquad (1)$$

where S = swell for 1-psi surcharge, percent. Swell potentials S of <1.5, 1.5-5.0, 5.0-25.0, and >25.0 percent are related to degrees of expansion of low, medium, high, and very high, respectively. The degree of expansion is illustrated by horizontal lines or boundary PI values (Figure 2). The dotted spaces in Figure 2 show the regions where the degrees of expansion by the D&R and SEED methods overlap. The degrees of expansiveness indicated in Figure 1 were developed independently of

* For convenience, symbols are listed and defined in the Notation (Appendix D).

Atterberg limits data and may provide a check of the degrees of expansion predicted from Figure 2.

40. Descriptions that also may aid determination of potential soil heave behavior include origin, hardness, fissures, slickensides, particle size, and special features such as roots and lime nodules.[46] For example, montmorillonites usually form from chemical weathering and diagenesis of volcanic ash (paragraph 10). Fissures and slickensides may be especially valuable indicators of swell potential because these may indicate a history of cyclic volume change due to climatic conditions. Severe fissures suggest large seasonal amplitudes in swell and shrinkage. Smaller particle sizes are usually associated with more plastic, montmorillonitic clays, which have greater swell potentials.

41. The actual swell in the field depends on a variety of field conditions discussed subsequently as well as on the potential of the soil for swell. As an illustration, CH clays are more likely to possess significant swell potential than CL clays. However, CL clays are also possible sources of damaging heave because of differences in permeability between CL and CH clays. CL clays with relatively low PI and LL values usually have relatively large permeabilities and could swell more during a single weather season, if adequate water is available, than the highly expansive but less permeable soils.

PART III: PREDICTION OF TOTAL HEAVE

42. Heave usually refers to vertical swell; however, lateral movements may be a factor in the stability of some structures such as basement walls and grade beams. Vertical heave usually occurs: (a) as a general upward movement beginning shortly after the start of construction and ending about five or more years after completion of the structure; (b) as a cyclic expansion and contraction normally occurring at the perimeter of buildings and related to the rainfall and evapotranspiration; and (c) as local heaving resulting from ponding, poor drainage, leaking water and sewer lines, or penetration of a shallow perched water table by elements of the foundation.[19] Heave resulting from the first case is usually a dome-shaped pattern with the greatest upward movement at the center of the structure.[11,47] Lawn watering or poor drainage of surface water, however, may cause the perimeter to heave relative to the center of the structure. Heave can be made to occur more quickly if water is added immediately; that is, by ponding. Heave resulting from infiltration or sorption of water into a deep desiccated zone may be very erratic, depending on the location and distribution of the zones of free water with respect to excavations and the ability of the water to gain access to the desiccated materials. Ponded water, for example, may seep down fissures or foundations, especially down foundations with loosely packed surrounding soils.

43. Predictions of the ultimate heave can be made from the difference in initial and final soil moisture profiles. Determination of the final equilibrium profile (that is, the final moisture content of the soil) presents the greater problem in predicting ultimate heave. Solutions to time rates of heave are often made from moisture diffusion theory[48-50] or an inverse application of the Terzaghi consolidation theory.[51-53] Although methods of predicting heave rates are not well advanced because of insufficient data on swell behavior of unsaturated soils and inability to quantify many field conditions, it is noted that most heave usually accumulates within 5-8 yr following construction.[11,32,33,54,55]

44. Accurate heave predictions may not always be necessary; observations of existing structures or empirical methods can give a good first estimate of the probable magnitude of heave. Heave predictions may not be needed for pile or pier foundations if (a) the foundation base can be placed below the active zone, or (b) seepage of surface moisture (down the soil-pile or soil-pier interface, or through concrete piers) into desiccated expansive subsoils can be avoided.

45. Methods for predicting heave include: (a) observation of existing structures as discussed earlier; (b) empirical relationships based on classification data, surcharge pressure, and thickness of expansive layers; and (c) procedures based on laboratory test results. Empirical procedures such as the McDowell,[4] Van Der Merwe,[56] Parry,[57] and Lambe[58] methods may lead to estimates of potential heave for design purposes. These procedures are often developed for local soil and climatic conditions on the basis of CS test results of undisturbed and/or remolded soils. Empirical procedures may require modifications and additional swell data for practical applications in other areas. Special equipment for measuring swell is sometimes needed for empirical methods.[57,58] The most successful methods for prediction of ultimate rate of heave depend on swell data from CS tests of undisturbed specimens.[12,59-65] CS tests should duplicate as many of the field conditions as possible.

Factors Influencing Heave

46. Reliable predictions of heave are extremely difficult to obtain because of numerous factors that influence the magnitude and rate of in situ swell.[11,18,19,46,47]

- a. Composition. Clay mineralogy, amount of clay mineral, and type and concentration of cations in the pore water.
- b. Structure. Geometry, specific surface area, bonds, platelet arrangement, fissures and slickensides, dry density, and permeability.
- c. Stratigraphy and attitude. Dip and strike of expansive layers, thickness of expansive stratum, depth of stable stratum above expansive soil, bedding, and stratification.
- d. Climate and previous environment. Depth of seasonal

influence, degree of initial desiccation, covering vegetation, and stress history.

e. Availability of water following construction. Rainfall, watering, leaking water lines, drainage pattern, ponding, depth to and character of the water table, amount of covered area, and hydrogenesis.

f. Surcharge pressure. Structure and overburden pressures.

g. Time. The initial and amount of elapsed time for water available to various locations.

h. Temperature. Increasing temperatures cause moisture to diffuse to cooler areas.

Heave prediction procedures have not been able to account fully for the effect of many of the factors above and not at all for such factors as lateral swell, cyclic seasonal climatic influence, and the actual availability of water to the soil. Lateral swell may be significant in desiccated and fissured soils, while practically all volume change from imbibition of moisture in nonfissured or tight soils may occur in the vertical direction. The effects of seasonal fluctuations in climate have been observed within 2 to 3 ft beneath the edge of the structure or pavement.[66]

47. Some assumption must be made about the availability of water when predicting heave. Actual groundwater conditions are often determined by local ponding, amount and frequency of rainfall, drainage, and depth to the water table. Moisture conditions in the foundation soil may vary over the lifespan of the structure from dry to wet depending on the occurrence of droughts and rainy seasons. A structure could be designed for the worst possible situation based on the swell between the possible driest and wettest conditions, resulting in an extremely conservative design. A more economical design could be based on the experience that the moisture balance beneath structures tends to approach equilibrium conditions described later. Localized effects of dry and wet seasons may be observed at the perimeter of existing structures and considered in the overall design of the foundation.

Laboratory Swell Tests

48. The procedure described in this report for predicting heave

is based on results from laboratory swell tests. The two types of swell tests recommended for most practical cases are:

- a. Constant volume swell (CVS). The undisturbed specimen for each stratum is loaded to the original soil overburden pressure, water is added, and the loading arm is restrained from movement until the full swelling pressure is developed. The specimen is unloaded incrementally to obtain the rebound curve from swell.
- b. Modified swell overburden (MSO). The undisturbed specimen for each stratum is loaded to the total surcharge pressure expected in the field following construction, water is added, and swelling is permitted until primary swell is complete. Additional pressure is applied following swell until the original void ratio prior to flooding with free water is reached. The pressure is reduced incrementally to obtain the rebound curve from swell.

The swell pressure measured during the CVS test may be defined as the pressure needed to prevent volume expansion in the soil that is in contact with free water.[67-71] A swell pressure may also be defined from the MSO test as that pressure needed to reduce the void ratio following swell at the total surcharge pressure to the original void ratio. Swell pressures evaluated from MSO tests may be larger than those determined from CVS tests. Further details of the swell tests and descriptions of swell pressure are given in Appendix A.

49. The CVS and MSO swell tests are recommended because: (a) routine consolidometer equipment is usually available, (b) procedures are relatively simple and fairly well known, (c) swell pressures are evaluated, and (d) the total heave can be predicted for a wide range of final loading and soil moisture conditions. The MSO test is preferable if the overburden pressures are known in advance and changes in the overburden pressures or structural loads due to modifications in the foundations are not expected. The MSO test is also adaptable to estimates of the rate of heave as discussed subsequently. The CVS test is preferable if final overburden pressures from the soil and structure weights are not known in advance or during laboratory tests.

50. The soil specimens following the swell tests may be consolidated to allow evaluation of settlements for cases where the total surcharge pressure exceeds the soil-swell pressure. Settlement, and not

swell, will develop for these cases. The computer code described subsequently for computing heave from the results of the above-mentioned swell tests does not contain provisions for the input parameters needed to evaluate settlement from laboratory consolidation tests. Such settlements will normally be minor in the course of evaluating heave predictions. Settlements that may result due to surcharge pressures exceeding the soil swell pressure are approximated in the code, except as noted later, by assuming a compression index C_c given by[72]

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

If the soil is significantly overconsolidated, a surcharge pressure in excess of the in situ overburden pressure, but less than the maximum past pressure, may be applied to the specimen before the swell test to reverse the expansion that probably occurred in the sample after removal from the borehole.

Computation of Total Heave

51. The total heave h in the soil profile is given by

$$h = dx \sum_{i=1}^{NEL} \frac{e_f(i) - e_o(i)}{1 + e_o(i)} \tag{3}$$

where

dx = increment of depth, ft

NEL = number of soil increments

$e_f(i)$ = final void ratio of soil increment i

$e_o(i)$ = initial in situ void ratio of soil increment i

The total heave from Equation 3 is assumed equal to the volumetric swell, and lateral swell is inhibited by the surrounding soils. The initial void ratio of each soil increment i, $e_o(i)$, is determined from the swell tests (Appendix A) for the original total overburden pressure P_o. The final void ratio depends on the final effective pressure $\overline{P}_f$

in each soil depth increment as indicated on the rebound curve of the void ratio-log pressure relationship of the soil (Appendix A). The final effective pressure is a function of the final or equilibrium moisture and loading conditions.

Equilibrium moisture conditions

52. Equilibrium moisture conditions in the soil profile beneath impervious covered areas or structures have been based on: (a) empirical estimates of the final water content, (b) saturation, and (c) negative hydrostatic head conditions. Final water contents from empirical estimates have been found to be about 1.1 to 1.3 times the plastic limit for the local conditions encountered.[59,73-77] These empirical correlations are not able to account for the effect of surcharge pressure on swell which may be important for heavy structures or for deep swelling soil strata.

53. Saturation case. A reasonable and useful equilibrium moisture profile for some practical applications is one of complete saturation (Figure 3).[4,12,78,79] The pore-water pressures are assumed zero in the saturated profile. The standard procedure used in military construction for estimating foundation soil swell assumes a saturated equilibrium profile.[62] Localized saturation of some foundation soils may result from leaking water pipes, drains, sewer lines, lawn watering, and ponding of surface water.

54. The equilibrium pore-water pressure will decrease from an intial negative pressure to approximately zero in the soil profile for the active zone. The active zone may be assumed to extend to the depth of shallow or perched water tables x_{wt} (Figure 3a and b). Shallow perched water table should be at a depth of less than about 20, 10, and 5 ft below ground surface for clays, sandy clays and silts, and sands, respectively.[26,80,81] The positive pore-water pressures in the soil below the surface of the original water table are assumed not to change as a result of the saturation of soils above the water table.

55. The depth of the active zone above deep water tables x_a (Figure 3c) can be difficult to determine in advance of construction and some assumption may be necessary. In cases where a deep foundation may

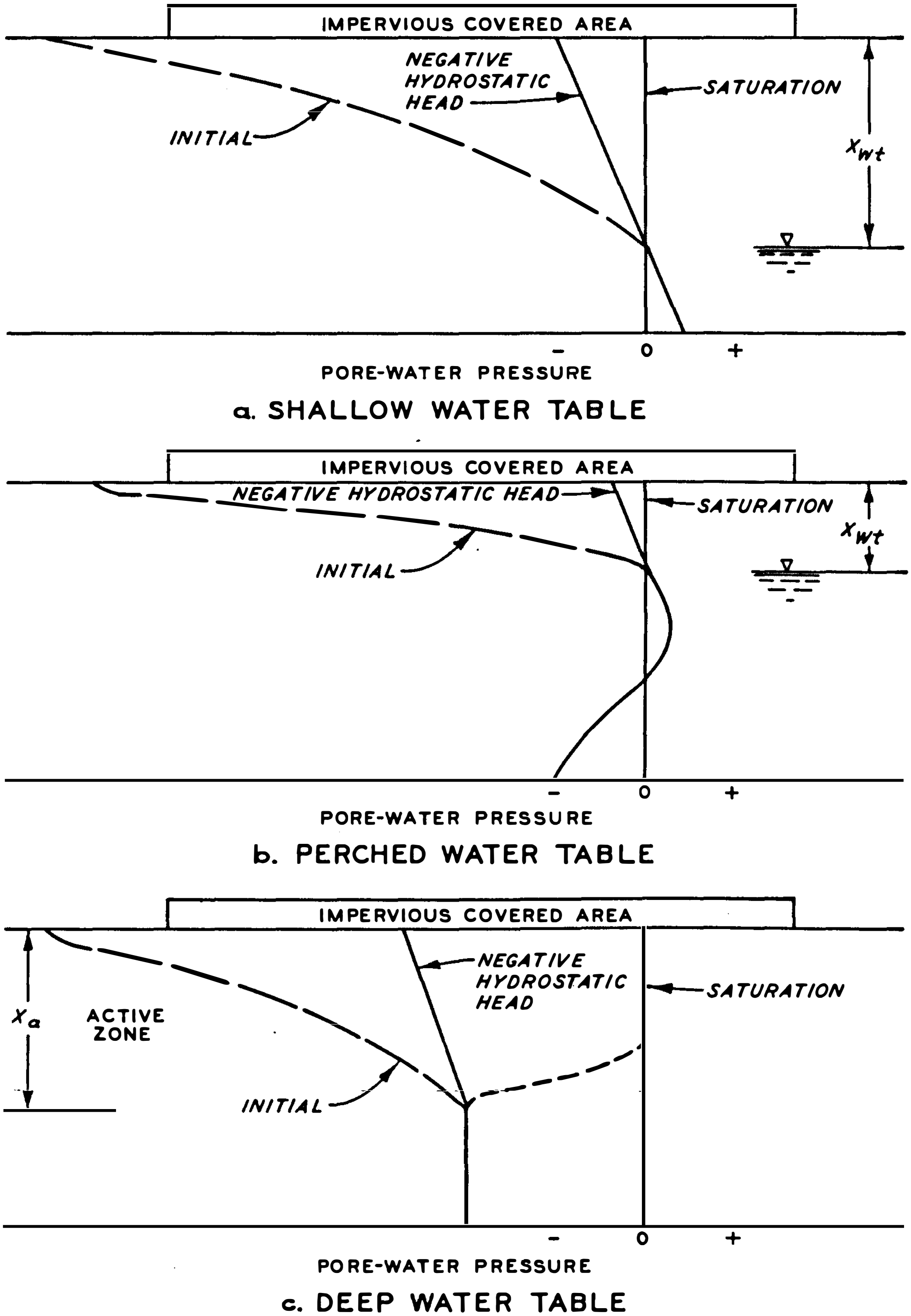

Figure 3. Pore-water pressure profiles beneath impervious covered areas

extend below a perched water table to achieve adequate bearing capacity, or where excavations are necessary to accommodate required features of the structure, water from the perched table may seep below the table and beneath the base of the foundation.

56. Negative hydrostatic head case. An equilibrium soil moisture profile beneath impervious covered areas, or structures for field conditions not subject to local saturation, can be made by assuming a negative hydrostatic head (Figure 3).[25,81-84] The equilibrium negative pore-water head above a shallow or perched water table (Figure 3a and b) is given by

$$\tau_m(x) = x_{wt} - x \tag{4}$$

where

$\tau_m(x)$ = in situ matrix suction head at depth x, ft

x_{wt} = depth to water table, ft

x = depth below ground surface, ft

The water table observed at the initiation of construction or during soil sampling should be taken for x_{wt} in the computer code discussed later.

57. The in situ matrix suction or negative pore-water pressure is a component of total suction, which is an energy term describing the force or thirst of the soil leading to the sorption of water and moisture flow in partially saturated soils. Total suction is often given as the sum of matrix and osmotic components. These components and total suction are defined in Table 1.[41] Matrix suction is related to the geometrical configuration of the soil, capillary tension in the pore water, and water adsorption forces of the clay particles.[48,85] Osmotic suction is a function of the concentration of soluble salts in the pore water. The effect of osmotic suction on imbibition of moisture is not well known, but an osmotic effect may be observed if the concentration of soluble salts in the pore water differs from that of the externally available water.

58. The equilibrium soil moisture profile beneath impervious covered areas with deep water tables (Figure 3c) is related to the

moisture balance between rainfall and evapotranspiration. It is usually established by empirical methods that correlate observed soil suctions with well-known climatic index values.[25,81] The equilibrium moisture profile for deep water tables can be approximated by[26,84]

$$\tau_m(x) = \tau_{ma} + x_a - x \tag{5}$$

where

$\tau_m(x)$ = in situ matrix suction at depth x, ft

τ_{ma} = in situ matrix suction at bottom of active zone, ft

x_a = depth of active zone, ft

The osmotic suction is assumed constant or zero in Equations 4 and 5 and in the following analyses so as not to exert any influence on swell.

59. The in situ matrix suction head τ_m may be found from laboratory suction test results by[86,87]

$$\tau_m = \tau_m^o - \frac{\alpha P_o}{\gamma_w} \tag{6}$$

where

τ_m^o = matrix suction head free of external pressure

α = compressibility factor

P_o = overburden pressure, lb/sq ft

γ_w = unit weight of water, lb/cu ft

A laboratory test to determine the matrix suction free of external pressure τ_m^o is described in Appendix B.

60. The compressibility factor α is the fraction of applied pressure which is effective in changing the pore-water pressure.[86,87] It is obtained by multiplying the unit weight of water (in grams per cubic centimetre) by the slope of a curve relating the reciprocal of the dry density (in cubic centimetres per gram, specific total volume[48]) to water content (in percent of dry weight). This factor will be zero for incompressible soils, such as clean sands at low degrees of saturation, but it will be equal to one for all fully saturated or quasi-saturated soils. The compressibility factor for CH clays is commonly set equal

to one, because the voids of these soils are filled with water within a wide range of water contents (quasi-saturated). The compressibility factor may be roughly estimated from the PI by[87]

$$PI < 5 \quad \alpha = 0 \tag{7a}$$

$$PI > 40 \quad \alpha = 1 \tag{7b}$$

$$5 < PI < 40 \quad \alpha = 0.0275PI - 0.125 \tag{7c}$$

Descriptive terms for different degrees of saturation and the corresponding states of pore-water and pore-air pressure are given in Table 2.[88]

Final effective pressure

61. Saturation case. The final effective pressure in a saturated soil where the pore-water pressure is zero is given by

$$\bar{P}_{fs}(i) = P_{fo}(i) + \Delta P_{st}(i) \tag{8}$$

where

$\bar{P}_{fs}(i)$ = final effective pressure of saturated soil increment i , lb/sq ft

$P_{fo}(i)$ = final soil overburden pressure of soil increment i , lb/sq ft

$\Delta P_{st}(i)$ = increase in pressure at soil increment i due to the structure, lb/sq ft

62. The increase in vertical pressure $\Delta P_{st}(i)$ caused by a structure and exerted on each soil element i (below the center of a footing located at the ground surface) can be approximated by the appropriate Boussinesq equation. The Boussinesq equations for calculation of $\Delta P_{st}(i)$ for rectangular, circular, and long, continuous footings are adopted for this report.[89-91] The derivations for the vertical soil stress assume a uniform pressure distribution exerted by a footing on a homogeneous, elastic, isotropic, and semi-infinite soil.

63. For a foundation placed below the ground surface or for a deep foundation, the increase in soil pressure at the base of the footing caused by the pressure of the structure is

$$\Delta P_b = Q - P_{fo}(NBX) \tag{9}$$

where

ΔP_b = increase in pressure at base of footing, lb/sq ft

Q = total structure pressure, lb/sq ft

$P_{fo}(NBX)$ = overburden pressure of surrounding soils at footing of foundation, lb/sq ft

The pressure Q at the base of the footing is estimated from structure and foundation weights. The increase in pressure at the base of the footing ΔP_b is input into the appropriate Boussinesq expression to evaluate the increase in soil pressure at the depth of soil increment i (below the base of the footing).

64. Swelling of soils surrounding deep foundations such as piers may cause uplift forces ($\underline{T}$) on the shafts and reduce the structure pressure Q at the footing (Figure 4) in addition to the reduction in Q from friction forces. Uplift forces sufficient to reduce the pier loading at the footing to less than the overburden pressure exerted by the surrounding soils at the footing depth (i.e., $0 < Q < P_{fo}(NBX)$) may reduce the total vertical effective soil pressure (in each soil element i beneath the center of the footing $\overline{P}_{fs}(i)$) to less than the in situ pressure prior to construction. An analogous example (previously discussed in paragraph 25) is the pressure reduction (in a bulb of soil beneath the bottom of an open borehole) caused by soil removal during the sampling operation. The amount of reduction in Q is complicated by the relation of the foundation stiffness to the soil stiffness, slippage between soil and the pier foundation, and lengthening of the pier from tension forces. If a void occurs beneath the footing because of sufficiently high uplift forces due to swelling soil or if the pier fractures, heave at the top of the pier will be greater than heave of soils beneath the footing, and heave of the pier will be a function of heave in the surrounding soils. The reduction in soil pressure for cases when Q at the base of the footing is less than the original overburden pressure $P_{fo}(NBX)$ due to uplift is calculated in this report by an inverse application of the Boussinesq equations. Equation 9 is still valid where

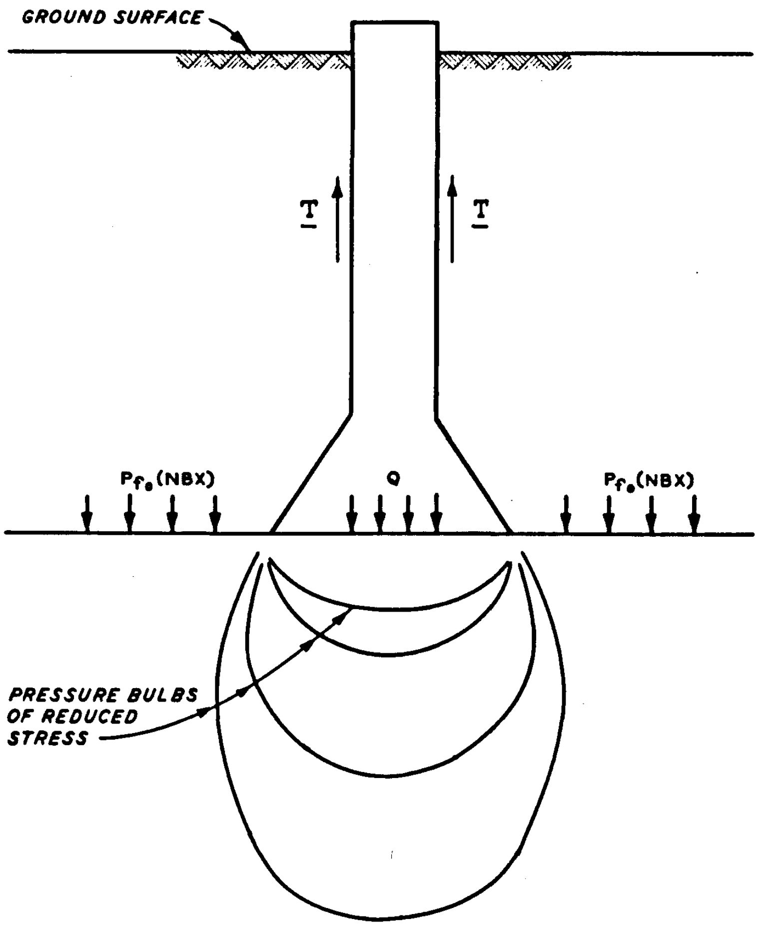

Figure 4. Schematic diagram of pressures near the footing of the pier

ΔP_b is negative. The net change in pressure at the i soil increment $\Delta P_{st}(i)$ is likewise negative, and the final pressure with respect to the soil overburden pressure $P_{fo}(i)$ is reduced (Equation 8).

65. Negative hydrostatic head case. The final effective pressure $\overline{P}_{fu}(i)$ of each soil increment i with an equilibrium profile controlled by hydrostatic conditions may be given by[92]

$$\overline{P}_{fu}(i) = \overline{P}_{fs}(i) + \beta\tau_m(i) \tag{10}$$

where

$\overline{P}_{fu}(i)$ = final effective pressure of partially saturated soil increment i , lb/sq ft

β = a function of the particle contact area, $0 < \beta \leq 1$

$\tau_m(i)$ = in situ matrix suction at soil increment i , lb/sq ft

The final effective pressure of the saturated soil increment $\overline{P}_{fs}(i)$ is given by Equation 8. The in situ matrix suction may be estimated from Equation 4 for shallow water tables and Equation 5 for deep water tables. The β parameter is taken as one for shallow water tables where the soils are assumed quasi-saturated. Unfortunately, the assumption of β equal to one for soil profiles with deep water tables where the degree of saturation may be less than one may not be realistic, and the following procedure is taken to compute the final void ratio of the soil profile for use in Equation 3.

66. The final void ratio in partially saturated soil profiles is

$$e_{fu}(i) = G_s(i)V_T(i) - 1.0 \tag{11}$$

where

$e_{fu}(i)$ = final void ratio in partially saturated soil of increment i

$G_s(i)$ = specific gravity of soil increment i

$V_T(i)$ = specific total volume of soil increment i following swell

The specific total volume $V_T(i)$ due to swell is given by Lytton and Watt[49] as

$$V_T(i) = V_{TI}(i) + \left[V_{TP}(i) - V_{TI}(i)\right]\left[\frac{w_{fu}(i) - w_o(i)}{w_{fs}(i) - w_o(i)}\right]^{QQ} \tag{12}$$

where

$V_{TI}(i)$ = initial specific total volume

$V_{TP}(i)$ = maximum specific total volume

$w_{fu}(i)$ = final water content of soil increment i , percent

$w_o(i)$ = initial water content of soil increment i , percent

$w_{fs}(i)$ = $e_{fs}(i)/G_s(i)$, maximum water content of soil increment i , percent

$e_{fs}(i)$ = maximum in situ void ratio of soil increment i

$$QQ = \left[w_{fs}(i) - w_o(i)\right] \Big/ \left[V_{TP}(i) - V_{TI}(i)\right] (100)$$

The initial specific total volume $V_{TI}(i)$ is

$$V_{TI}(i) = \frac{1 + e_o(i)}{G_s(i)} \tag{13a}$$

and the maximum specific total volume $V_{TP}(i)$ is

$$V_{TP}(i) = \frac{1 + e_{fs}(i)}{G_s} \tag{13b}$$

The maximum in situ void ratio or maximum in situ water content corresponds to saturation or a soil state of zero in situ matrix suction.

67. The final water content $w_{fu}(i)$ may be evaluated from the final matrix suction free of external pressure τ_m^o and the matrix suction free of external pressure-water content relationships from laboratory suction tests (Appendix B).[93,94] The τ_m^o is found from the final in situ matrix suction τ_m (Equation 6) where τ_m is evaluated from Equation 5. The depth of the active zone below ground surface x_a (Equation 5) may be given a reasonable value based on past experience.

Computation of Rate of Heave

68. The rate of heave can only be approximated because it is difficult to predict the location and time of water availability to the foundation soils. In many cases the availability of water may be assumed to occur from only one surface. For example, water may be assumed to: (a) infiltrate from the surface for a saturated equilibrium moisture profile, (b) flow by capillarity forces from the water table, or (c) flow from the bottom of the active zone for a hydrostatic equilibrium moisture profile. For deep foundations and an assumed saturated equilibrium moisture profile, time may be necessary for water to seep

down to the bottom of the foundation before water can infiltrate into the foundation soils beneath the footings. The calculated rates of heave, especially for deep foundations, may overestimate actual field rates of heave and may provide a conservative or minimum time needed to accumulate certain amounts of heave.

69. The rate of heave can be approximated by an inverse application of the Terzaghi consolidation theory

$$t = \frac{T x_a^2}{\underline{c}_{vs}} \tag{14}$$

where

t = time, days

T = time factor for various percentages of ultimate swell

x_a = depth of active zone for sorption of moisture from one surface, ft

$\underline{c}_{vs}$ = average coefficient of swell for the soil in the active zone, sq ft/day

70. The average coefficient of swell in the soils beneath the footings of the foundation subject to the changing moisture conditions (active zone) may be approximated by

$$\underline{c}_{vs} = \frac{\underline{k}}{\underline{m}_{vs}\gamma_w} \tag{15}$$

where

$\underline{k}$ = average coefficient of permeability, ft/day

$\underline{m}_{vs}$ = average coefficient of volume change from swell, sq ft/lb

71. The average coefficient of permeability in a vertical direction of a horizontally layered soil profile is

$$\underline{k} = \frac{x_a}{\sum_{i=1}^{NMAT} \frac{x(i)}{k(i)}} \tag{16}$$

where

$$\text{NMAT} = \text{number of soil layers or materials}$$

$$x(i)\ ,\ (i = 1,2,\ldots\text{NMAT}) = \text{vertical dimension of soil layer } i\ ,\ \text{ft}$$

$$k(i)\ ,\ (i = 1,2,\ldots\text{NMAT}) = \text{coefficient of permeability of soil layer } i\ ,\ \text{ft/day}$$

72. The average coefficient of volume change from swell is estimated by

$$\underline{m}_{vs} = \frac{\sum\limits_{i=1}^{\text{NMAT}} m_{vs}(i)x(i)}{x_a} \qquad (17)$$

where $m_{vs}(i)$, $(i = 1,2,\ldots\text{NMAT})$ = coefficient of volume change from swell of soil layer i , sq ft/lb.

73. Since

$$k(i) = c_{vs}(i)m_{vs}(i)\gamma_w \qquad (18)$$

where $c_{vs}(i)$, $(i = 1,2,\ldots\text{NMAT})$ = coefficient of swell of soil layer i , sq ft/day, the average coefficient of swell for the soil in the active zone may be estimated by

$$\underline{c}_{vs} = \frac{x_a^2}{\left[\sum\limits_{i=1}^{\text{NMAT}} \frac{x(i)}{c_{vs}(i)m_{vs}(i)}\right]\left[\sum\limits_{i-1}^{\text{NMAT}} m_{vs}(i)x(i)\right]} \qquad (19)$$

74. The $c_{vs}(i)$ of each soil layer may be estimated from the results of the modified swell overburden tests. The results from swell tests may underestimate the field coefficient of swell because the mass structure and larger fissures may not be represented by the relatively small specimens. Sample disturbance may also affect the coefficient of swell. Underestimation of the field coefficient of swell may lead to

lower calculated heave rates and tend to counteract errors in overestimating heave rates due to conditions that control the actual availability of water to the soil. The swell (in inches) of the soil specimen subject to the overburden pressure P_o on inundation with water may be plotted as a function of the logarithm of time (in minutes). The coefficient of swell of each soil layer (in square feet/day) may be calculated by using the logarithm of time fitting method.[95]

$$c_{vs}(i) = \frac{T_{90}H^2}{t_{90}} \times 10 \qquad (20)$$

where

T_{90} = time factor to complete 90 percent of the primary swell, 0.848

H = one-half of the thickness of the specimen for sorption from both top and bottom of the specimen, in.

t_{90} = time to complete 90 percent of primary swell, min

PART IV: APPLICATIONS TO FOUNDATION DESIGN AND COMPUTER CODE

Minimizing Foundation Damage from Expansive Soils

75. Types of damages sustained by structures due to differential heave of the foundation expansive soils include[39]

a. Heaving of on-grade floor slabs. Expansion of either the overburden foundation soils and/or deeper foundation materials causes heaving.

b. Cracks in grade beams. Expansive overburden foundation soils can exert enough pressure on the bottom of beams to crack and cause complete failure where voids are not provided. Differential movement between two supporting points can cause cracks in grade beams.

c. Cracks in walls. Differential foundation movement and rigid walls cause cracks.

d. Cracks in pier shafts. Expansion of materials through which insufficiently reinforced pier shafts pass, and upward forces exerted on pier shafts by skin friction developed by surrounding expansive soils, cause cracks from induced tension.

e. Concrete plinth failure. Upward forces on pier shafts and differential movement of adjacent piers induce excessive movement, axial loads, and bending stresses that may cause failure.

Lateral forces may lead to the buckling of subsurface and basement walls, especially in overconsolidated and nonfissured soils.

76. Possible courses of action to eliminate or minimize these types of damage include (a) special types of foundations for structures in expansive soil areas, (b) soil stabilization and control of moisture, and (c) loading to counter soil-swell pressures, or (d) a combination of these alternatives. An expedient course of action for existing structures already subject to damaging heave is to estimate the remaining probable future heave, apply procedures to minimize the heave and its effect, and repair the damage. Most heave may have already occurred if the structure is more than 5 yr old, and measures may consist of cosmetic repairs to the structure as well as repair of any structural

damage. Investigating the cause of the heave is also recommended so that further damage from or repetition of the cause (i.e., broken pipes or poor drainage) can be avoided.

Types of foundations

77. Types of foundations for structures in expansive soil areas can be classified as shallow, shallow with split construction, stiffened mat, and deep (isolation) foundations (Table 3). Details of these foundations are readily available in References 11,19,22,39,57,96-98. Special construction procedures are usually not necessary for predicted differential heaves less than 0.5 in., while deep foundations are common for differential heaves exceeding 2 in. at the ground surface. Predicted differential heaves for various types of foundations should be weighed against tolerances of the proposed structure to differential movement. Split construction with shallow foundations is useful when differential heave cannot be easily eliminated by foundation treatment and/or controlled loading techniques. Split construction with deep foundations can further increase the resistance of the structure to damage from deep-seated, highly expansive foundation soils.

78. Cast-in-place piers. The most commonly recommended deep foundation is cast-in-place underreamed concrete piers. Grade beams should be placed on the piers or concrete plinths above the ground surface to allow a sufficient open space between the structure and the soil surface to accommodate soil heave. The bell-bottomed footings of cast-in-place piers can usually be placed at the desired depth. The bell-bottoms should be constructed quickly to avoid changes in the soil moisture. The bell-bottom should preferably be embedded in a free-water zone or in nonexpansive soil, to reduce heave beneath the pier. Footings may be placed beneath the swelling soil near the top of a granular stratum within the water table to avoid fall-in of material during boring. Straight shafts may be more economical than bell-bottom footings if the bearing stratum is hard or if the overburden material is unstable.

79. The underreamed footing contributes anchorage against uplift forces due to heave of soils surrounding the shaft. Soils lying above the bell also contribute surcharge weight on the underlying foundation

soils in addition to the structural load transmitted through the shaft. The bells of the piers should be underreamed not to exceed three times the shaft diameter. Large bell-to-shaft-diameter ratios minimize uplift forces on pier shafts and provide anchorage.

80. Penetration of moisture down the pier shaft may be minimized by high-density, low-permeability concrete. Care should be exercised while pouring concrete for deep foundations and piers to ensure continuity. Vibration of the concrete will eliminate voids in the pier. High concrete slumps of 4 to 6 in. and limited aggregate size are recommended to facilitate flow of concrete through reinforcement cages and to reduce cavities in the pier. Additional cement should be added to the concrete mix to maintain the strength of high-slump concrete.

81. Widely spaced piers constructed with small shaft diameters and concentration of loading forces consistent with the soil bearing capacity will counteract uplift forces; however, long, slender shafts and shafts less than 12 to 18 in. should be avoided. The diameter of the footings that will transmit structural pressures within the allowable bearing capacity could be evaluated on the basis of end bearing or side shear against the pier shaft located below the active zone (the bottom 5 ft above the bell should be neglected). However, the allowable bearing capacity of the soils is more safely evaluated on the basis of end bearing because piers are often ideally bottomed just below the depth of the active zone and soil shrinkage in the active zone may eliminate side shear. A factor of safety of three for the design load at the base of the pier is usually adequate.[88,99]

82. Uplift forces. Uplift forces will develop against the surfaces of deep foundations when wetting of the surrounding expansive soil occurs. The shaft of pier foundations may be stressed in tension and should be designed with sufficient percentage of reinforcing steel to resist the maximum uplift forces from the adjacent soils. Reinforcing steel should be continuous between the shaft and bell-bottom. Coating the drill hole with a bitumen slip layer[100] may help to reduce skin friction and uplift forces on the shaft, to inhibit migration of moisture

down the soil-shaft interface, and to minimize seepage of moisture from the concrete to adjacent soils.

83. The total maximum tension force in the pier at any depth x may be estimated by[101]

$$\underline{T} = \pi D \int_0^x (C + K\gamma x \tan \phi)\, dx - P \qquad (21)$$

where

D = shaft diameter, ft

C = soil cohesion, lb/sq ft

K = ratio of intergranular pressures on horizontal and vertical planes

γ = unit weight of soil, lb/cu ft

ϕ = angle of internal friction, deg

P = vertical load applied at top of pier, lb

The most appropriate laboratory tests to evaluate the C, ϕ, and K parameters are uncertain at this time. Collins[101] found that shear characteristics from consolidated-drained triaxial tests with K equal to one correlated well with results of field pier tests at Leeuhof. Unfortunately, the low permeability of many expansive soils precludes economical drained triaxial testing.[102] The results of drained ($\overline{S}$) direct shear tests and assuming K equal to one may provide more practical interim values until further information is available.

Foundation treatment

84. Common treatment methods indicated in Table 4 include (a) chemical stabilization, (b) compaction control, (c) moisture control, (d) removal and replacement with nonexpansive backfill, and (e) ponding. Stabilization with 2 to 8 percent lime thoroughly mixed with the foundation soil has been successful in many field situations. Compaction at water contents or soil suctions near the equilibrium moisture conditions may also minimize heave, particularly for moderately swelling soils. For soils with high swelling characteristics, increasing water content and reducing density in order to reduce the potential for heave may be impractical and lead to low bearing capacity or poor workability.[108,109]

85. Foundation treatments are usually limited to surface soils and cannot be applied easily to foundation soils beneath existing structures. Backfilled soil to be placed adjacent to subsurface walls, however, may be easily treated with lime. Vertically placed or sprayed asphalt membranes to encapsulate and isolate expansive soil from surface moisture may be useful to reduce soil swell and lateral forces near subsurface walls and foundations. Catalytically blown asphalt membranes have been effective for minimizing heave of subgrade soils below the membranes in highway construction where the source of moisture is from the surface.[106,107] Plastic membranes may not be successful if punctures, holes, or leaks exist. The asphalt coating may be useful near and around deep foundations and underground water and sewer lines to minimize the penetration of free water into desiccated foundation soils. Local experience should be studied to determine the most successful foundation treatments in the area.

86. Ponding of surface water near the structure should be avoided during and following construction. A small downward slope leading from the structure is useful to help drain surface water away from the structure. If possible, foundation construction should be scheduled near the end of the wet season when foundation soils tend to be moist and close to the equilibrium moisture conditions. This may be impractical in some cases because of timing limitations or poor working conditions. The possibility of excessive settlements should also be checked. Excavations should be covered quickly to avoid drying of the foundation subsoils.

Controlled loading

87. Heave may often be minimized by distributing the surcharge loads over the foundation to counter the swell pressures in the expansive soils that develop on contact with moisture. Beneficial results from the distribution of structural loads required to counter uplift and minimize differential heave depend on the initial groundwater conditions, availability and distribution of water to the foundation soils, and lateral variations in the expansive soil strata. The problem of predicting the time and amount of water accumulation in different foundation soil areas complicates the calculation of the optimum structural load distribution.

The arrangement of the superstructure frequently makes optimum load distribution impossible.

Selection of the Foundation

88. The choice of the type of foundation for new structures is usually made early in the design stage and depends on:[39]

- a. Structure and architecture. Required height of floor above grade, building height, framing type, span between frame or columns, and column loads.
- b. Site features. Drainage and surface topography.
- c. Subsurface features. Soft zones, depth to the bearing stratum, depth to groundwater, and construction feasibility.

The details of the final foundation design depend on the total and differential heave predictions and procedures taken to minimize the effect of the heave on the structure. The groundwater conditions and the probable availability of water to the foundation soils are primarily responsible for total and differential heave, and should influence the selection of final design.

Shallow or perched water tables

89. Capillary rise due to the existence of shallow or perched water tables will probably lead to heave beneath structures located above the water table in expansive soils. A moisture profile approximated by the negative hydrostatic head may eventually develop (Figure 3a and b). Shrinkage may occur if: (a) the initial soil profile is wetter than the negative hydrostatic head moisture profile, and (b) surface groundwater is drained away to prevent penetration into the foundation soils. Heat from the structure may cause further moisture to diffuse from the immediate area. Penetration of surface groundwater from rainfall, watering, and poor drainage may serve to hasten the accumulation of water and may lead to a saturation case moisture profile.

90. Design features may concentrate on minimizing or resisting the differential heave for slab-on-grade or shallow foundations of light structures. An appropriate type of foundation may be the shallow

foundation with split construction or the stiffened mat foundation (Table 3). Heave may be avoided entirely for deep foundations bottomed in the shallow or perched water table. Foundation treatment techniques (Table 4), such as proper drainage, may also help to minimize heave.

91. Deep foundations, such as concrete piers for major structures that must be bottomed below a perched water table to achieve adequate bearing capacity, may lead to heave of desiccated expansive subsoils due to moisture seepage from the perched table down the pier. The pier and bell diameter may be selected to achieve high column loads and bearing pressures on the soil beneath the footing, balance the soil-swell pressures, and minimize heave of the deep foundation (in spite of moisture seepage beneath the footing). For a given structural weight, the column loads can be increased by increasing the span between footings as well as by decreasing the size of the footings. An added advantage of increasing the span distance between footings is that a smaller angular rotation of the structural member will occur for a given amount of vertical movement. This reduces the degree of disturbance that may occur in the structure. Variations in pier diameters should be minimized in the foundation to simplify construction, reduce contractor equipment on the site, and minimize cost.

Deep water tables

92. The absence of a shallow or perched water table may lead to heave from capillary rise if the soil suction in the initial moisture profile is greater than the negative hydrostatic head (Figure 3c). Heave or shrinkage may not occur if the initial moisture profile is about the same as the negative hydrostatic head. Shrinkage may occur if heat from the structure causes moisture to diffuse from the soil around the structure. Surface groundwater penetration may cause heave of the foundation soils and lead to localized saturation.

93. Design features may include minimizing the penetration of surface groundwater by, for example, proper drainage and use of watertight joints in drains, water lines, and sewer lines. An appropriate foundation for light structures may again be the shallow foundation with split construction or stiffened mat. Differential heave may be

minimized by construction of impervious membranes or covered areas attached to the structure for some distance from the structure, i.e. 10 ft or more in width. Surface water collection near deep foundations such as concrete piers may allow water to seep down into desiccated foundation soils. Therefore, special measures such as good drainage away from piers and impervious surfaces adjacent to piers should be provided.

Examples

Computer program

94. Capabilities. The computer program called ULTHE1 (Appendix C) was developed to expedite the prediction of the total ultimate heave and rate of heave for saturation and negative hydrostatic head moisture profile cases with shallow or deep water tables. Computation of heave for a soil profile containing a perched water table is similar to: (a) a shallow water table, if the foundation does not pass through the perched table, and (b) a deep water table, if the foundation passes through the perched table.

95. The ultimate heave is the total vertical heave at the base of the footing. The rate of heave is indicated by the time (in days) required to reach 20, 40, 60, 80, and 90 percent of the ultimate heave. Estimates of differential heave may be made by comparing differences in heave calculated at different locations.

96. The difference in swell pressure and the total surcharge pressure (denoted as EXCESS SWELL PRESSURE in the output data) is computed as a function of depth to indicate the additional surcharge pressure needed to prevent the calculated heave. This calculation applies to the saturation cases with shallow and deep water tables and to the negative hydrostatic head case (denoted as hydrostatic case in the code) with a shallow water table. The EXCESS SWELL PRESSURE for the hydrostatic case with a deep water table represents the suction remaining in the soil following swell. The suction is greater than the swell pressure if the degree of saturation is less than one and for quasi-saturated soils it is equivalent to the swell pressure.[108]

97. The structural surcharge pressures at the footing necessary to reduce heave to negligible values may be calculated by taking advan- of the capability to perform a number of loading cases through NPROB (Appendix C). This trial-and-error procedure allows a variety of structure loads Q to be input to determine the optimum Q for reducing heave to tolerable levels.

98. Assumptions. The previously discussed equations for predicting heave and rate of heave are used in the code. Other assumptions are:

a. Total vertical heave equals volumetric swell.

b. Settlement, if pressure exceeds the swell pressure, for saturation cases with shallow and deep water tables and the hydrostatic case with shallow water table is computed assuming a compression index $C_c = 0.007(LL - 10)$ where LL is the liquid limit.[72]

c. Settlement for the hydrostatic case with a deep water table is based on a specific total volume V_T (Equation 12) where the final water content is less than the original water content.

d. The rate of heave is determined for sorption of moisture from one surface.

99. Input data. The input data consist of various parameters for defining the scope of the problem and a number of soil parameters for each soil stratum (Appendix C). Increasing the number of layers or strata and laboratory tests will help improve reliability of heave predictions from the code. Laboratory tests are necessary to determine for each soil stratum:

a. Specific gravity.

b. LL.

c. PI.

d. Initial water content, percent.

e. Initial void ratio at the original average overburden pressure P_o of the stratum.

f. Void ratio after swell at P_o .

g. Swell pressure.

h. Coefficient of swell if rate of heave is needed.

i. Suction parameters A,B for a deep water table.

Parameters a through d are obtained from standard laboratory tests.[103] Parameters e through h are obtained from the CVS and/or MSO swell tests (Appendix A). The suction parameters of line i (described in Appendix B) are used to evaluate the suction-water content relationship and are needed only for the hydrostatic case with a deep water table (OPTION = 1 and NWAT = 1 in the computer code). The A,B suction parameters may be roughly correlated with the LL, PI, and natural water content, and may be calculated by the computer code with NSUCT = 0 .

Soil properties
for example problems

100. Field soil exploration programs were conducted at the Lackland test pier site, Lackland Air Force Base, Texas (LAFB), in support of the subject study. The primary soil formations at LAFB were transported, probably from two or more sources on two or more occasions. The profile at the test pier site includes about 13 ft of residual silty and limy clays overlying the Upper Midway formation.[37] The Upper Midway formation is weathered and fissured deeper than 50 ft. Piezometric readings indicated a perched water table at approximately 8 ft below ground surface.

101. The soil parameters were selected on the basis of laboratory test results on several samples from two different borings taken at different times (Table 5). Boring PU-7 was obtained in December 1970 near the end of a long, dry period of several years, while boring LAFB 1 was obtained in April 1973 after rainfall. Heave computations for soil samples from both borings permitted estimates of differential heave, assuming that each boring was taken from a different location in the foundation soils of the proposed structure. The difference in the sampling time probably contributed to the difference in heave computed for samples from each boring. The specific gravity, liquid limit, plasticity index, and water content of a specimen from an undisturbed sample of each of the borings within each of the depth intervals in Table 5 were determined by standard tests.[102] The void ratios e_o (initial at P_o), e_{po} (after saturation at P_o), and e_s (after saturation at 0.1 ton/sq ft); initial overburden pressure P_o ; and swell pressure S_p were

evaluated from CVS and MSO tests described in Appendix A. The suction parameters A and B were evaluated by the test described in Appendix B. The coefficients of swell were evaluated from Equation 20 and swell-time plots of MSO tests similar to that described in Figure A5, step 3.

102. The reliability of laboratory measurements of the coefficient of swell c_{vs} in representing field conditions for computation of rates of heave is not established. The coefficients in Table 5 are four times larger than those computed from laboratory test results, but the listed coefficients are satisfactory for purposes of illustration. They may actually be more representative of field conditions, since the relatively small laboratory specimens may eliminate larger discontinuities existing in the soil mass.

103. A study of the central south area of Texas in Figure 1 shows that the area of LAFB is subject to a high degree of expansiveness, and structures are vulnerable to damages from heave. On the basis of the plasticity data in Table 5 and using the D&R method in Figure 2, the degrees of expansion of the Lackland soil are high or very high, except for an 8- to 12-ft layer of a chert and limestone gravel bed (derived from nearby cretaceous formation[110]) in a sample of boring PU-7. The degree of expansion in this layer is low. To illustrate the capabilities of the computer code, the data in Table 5 are used subsequently to evaluate total heave for slab foundations and a variety of deep foundations, for both saturation and hydrostatic moisture profiles.

Slab foundations

104. Slab at ground surface. A lightly loaded structure 100 by 100 ft square is to be constructed with a slab foundation on the ground surface. The bearing pressure is 144 lb/sq ft (1 lb/sq in.) uniformly distributed over the entire area. The schematic of the slab and soil profile is illustrated in Figure 5. A shallow water table is observed 8 ft below ground surface and an active depth to the water table x_a of 8 ft is assumed.

105. The results of the computer analysis (Figure 6) show that the west end of the proposed structure will heave from moisture sorption to an equilibrium given by a negative hydrostatic head much more quickly

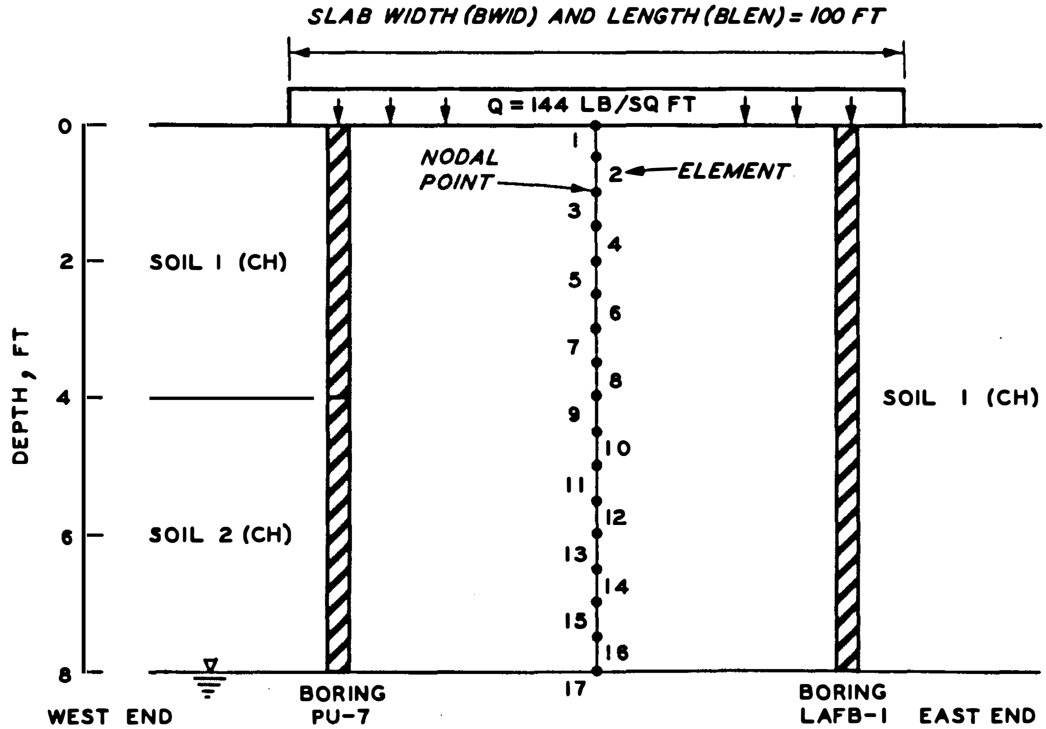

Figure 5. Slab foundation on ground surface with shallow water table 8 ft below ground surface

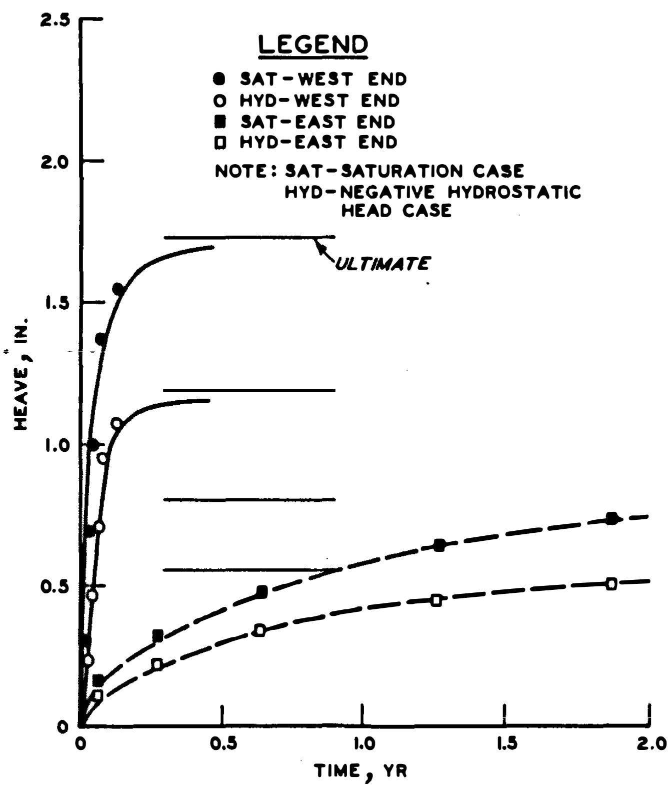

Figure 6. Heave with time of slab foundation on ground surface with shallow water table 8 ft below ground surface

than the east end. The differential heave between the west and east ends after 3 months will be about 1 in. The differential heave will decrease slightly with time after several years. Heave will be somewhat greater if the foundation soils become saturated (SAT, Figure 6) from infiltration of surface water.

106. A stiffened mat foundation will probably be satisfactory. Surface moisture from rainfall, watering, and runoff should be drained away from the structure to minimize penetration of surface moisture into the foundation soils.

107. <u>Slab in excavation.</u> A major structure with a 100- by 100-ft slab is to be constructed in an excavation 12 ft below ground surface to provide a basement (Figure 7). The excavation is below a perched water table. The slab will exert a bearing pressure of 144 lb/sq ft and will be placed directly on grade if feasible. The bearing capacity of the soil at this depth was determined to be adequate to support footings for the structure. A possible active depth x_a of 10 ft below the slab is assumed.

108. The results of the computer analysis (Figure 8) show that the west end of the slab will swell much more than the east end and will result in a differential heave of more than 4 in. after 5 yr if water from the perched table or other sources seeps into the soils beneath the slab (SAT, Figure 8). If water can be prevented from diffusing into the soils beneath the slab, heave may be negligible and, in fact, the soil beneath the east end may dry slightly and result in some settlement (HYD, Figure 8).

109. A slab-on-grade permitted to float on the ground independently of the footings appears to be a reasonable choice if proper drainage is available. A drainpipe underlain by an impervious membrane glued to the wall should be constructed around the outside perimeter of the structure just above the footings to collect any seepage moisture. All water and sewer lines should be placed near the east end of the structure, if possible, and constructed with flexible, watertight joints.

110. If the footings must be located at a depth below the slab to achieve adequate bearing capacity, and the perched water table also

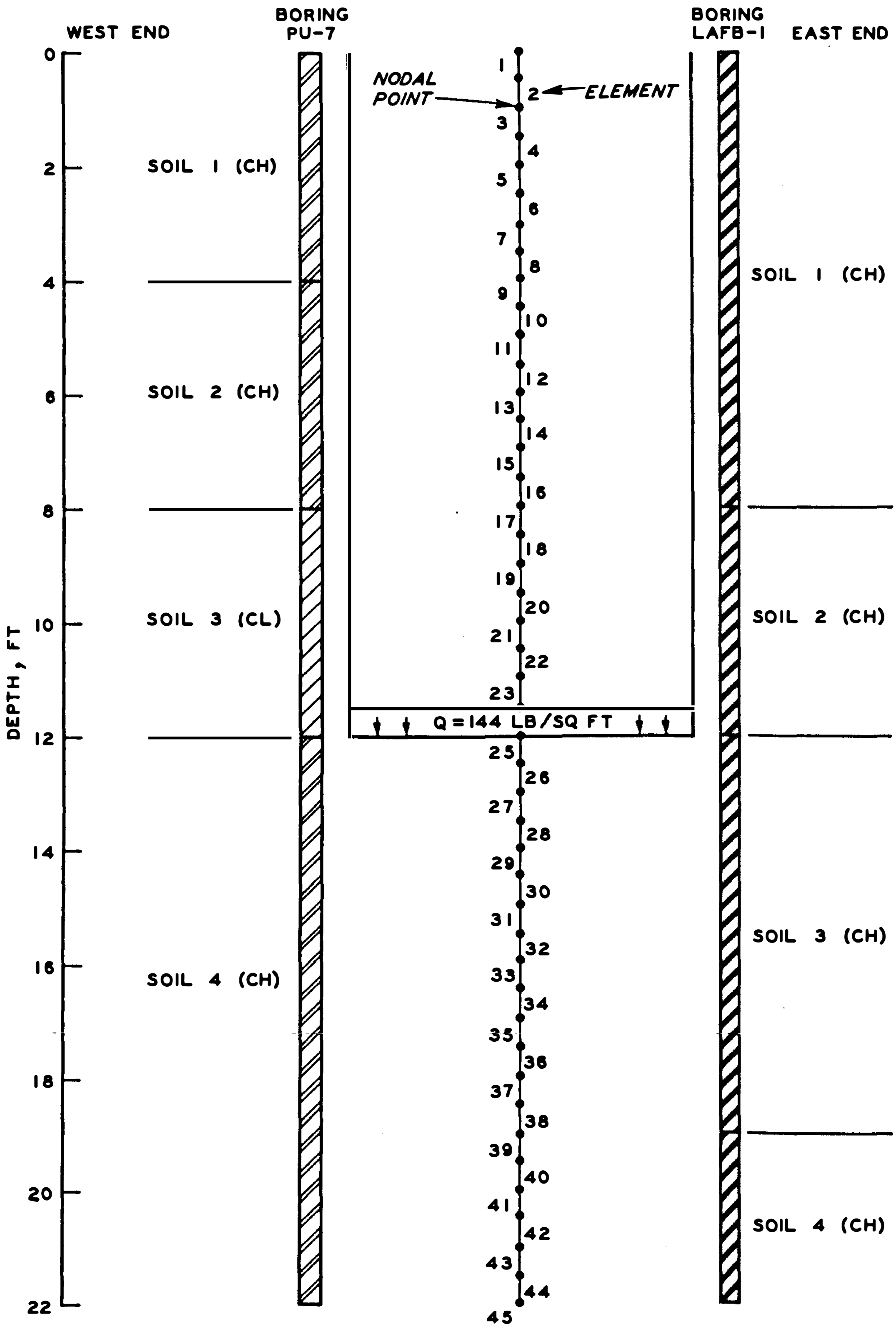

Figure 7. Slab foundation in excavation 12 ft below ground surface with deep water table

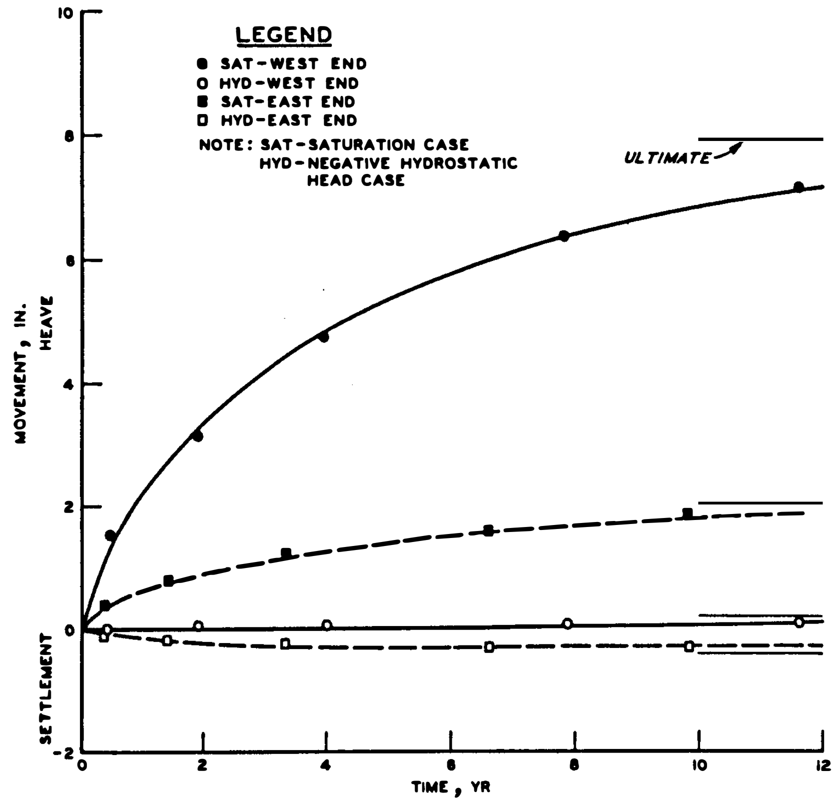

Figure 8. Heave with time of slab foundation in excavation 12 ft below ground surface with deep water table

extends below the excavation and slab, the slab may need to be isolated from the ground by a void space and supported on the footings to avoid heave from possible seepage down the footings. This analysis is discussed in the following example.

Deep foundations

111. A major structure is to be constructed on cast-in-place underreamed piers with the footings 30 ft below ground surface and passing through a perched water table (Figure 9). Seepage of water from the perched table down the piers is expected to eventually saturate the subsoils to a depth of 10 ft beneath the footings (saturation case). Beam span and footing diameters are to be adjusted to raise the bearing pressure to the allowable bearing capacity of 6 tons/sq ft.[37] Uplift forces are assumed to develop eventually against a 15-ft length of the pier shafts due to sorption of moisture into adjacent expansive soils. The

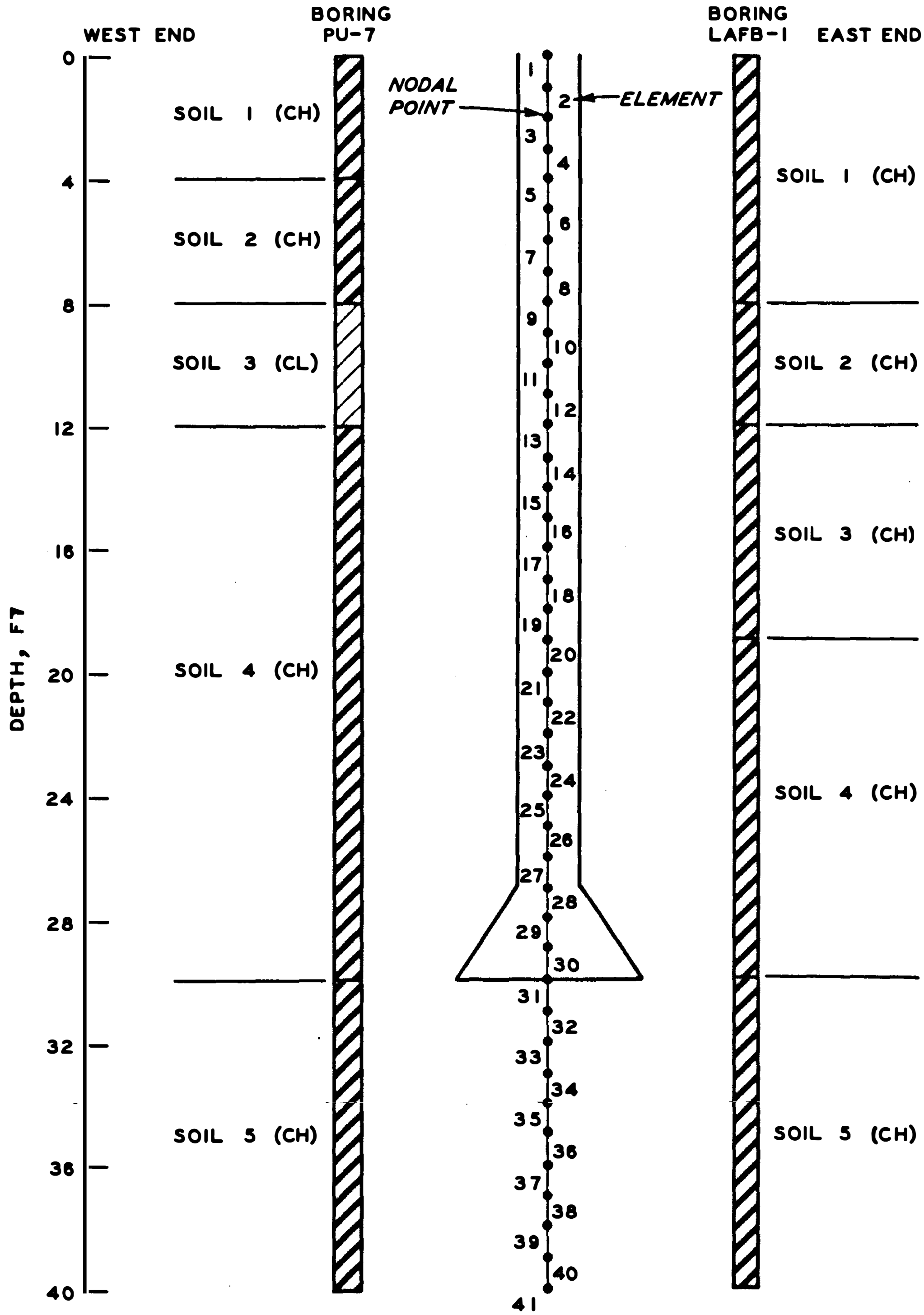

Figure 9. Deep foundation pier 30 ft below ground surface with deep water table

soil cohesion is 1 ton/sq ft and the angle of internal friction is zero.

112. Pier sizes available from the contractor are 18-, 24-, and 36-in.-diam shafts with bells three times the shaft diameter. Total loads at the footing needed for a bearing pressure of 6 tons/sq ft are 95.4, 169.6, and 381.7 tons for the 18-, 24-, and 36-in.-diam shafts, respectively. At some time during the life of the structure, uplift forces may reduce loads at the footing (Equation 20) by 84.8, 94.2, and 141.3 tons for 18-, 24-, and 36-in.-diam shafts, respectively. The actual vertical downward force at the footing may become quite small if the soils surrounding the shaft develop uplift forces from moisture imbibition or if the transfer of applied loads from the shaft to surrounding soils is significant.[111]

113. The results of the analysis (Figure 10) show that the west piers will heave more than the east piers if moisture seeps into the soils beneath the footings, and heave will be greater if uplift forces develop. Larger diameter piers are more effective in reducing heave for the same bearing pressure of 6 tons/sq ft; however, larger loading forces are necessary. Smaller diameter piers may be necessary if loads cannot be concentrated enough to reach the allowable bearing pressure with larger piers. Minimal reinforcing steel is adequate because tension forces are not expected in the shafts.

114. Differential heave between the west and east ends of several inches is possible after 5 yr or more. The building should be constructed with grade beams on piers and sufficient joints to accommodate the possible differential heave in the superstructure. Floors should be suspended above the ground to isolate the floors from foundation soil expansion.

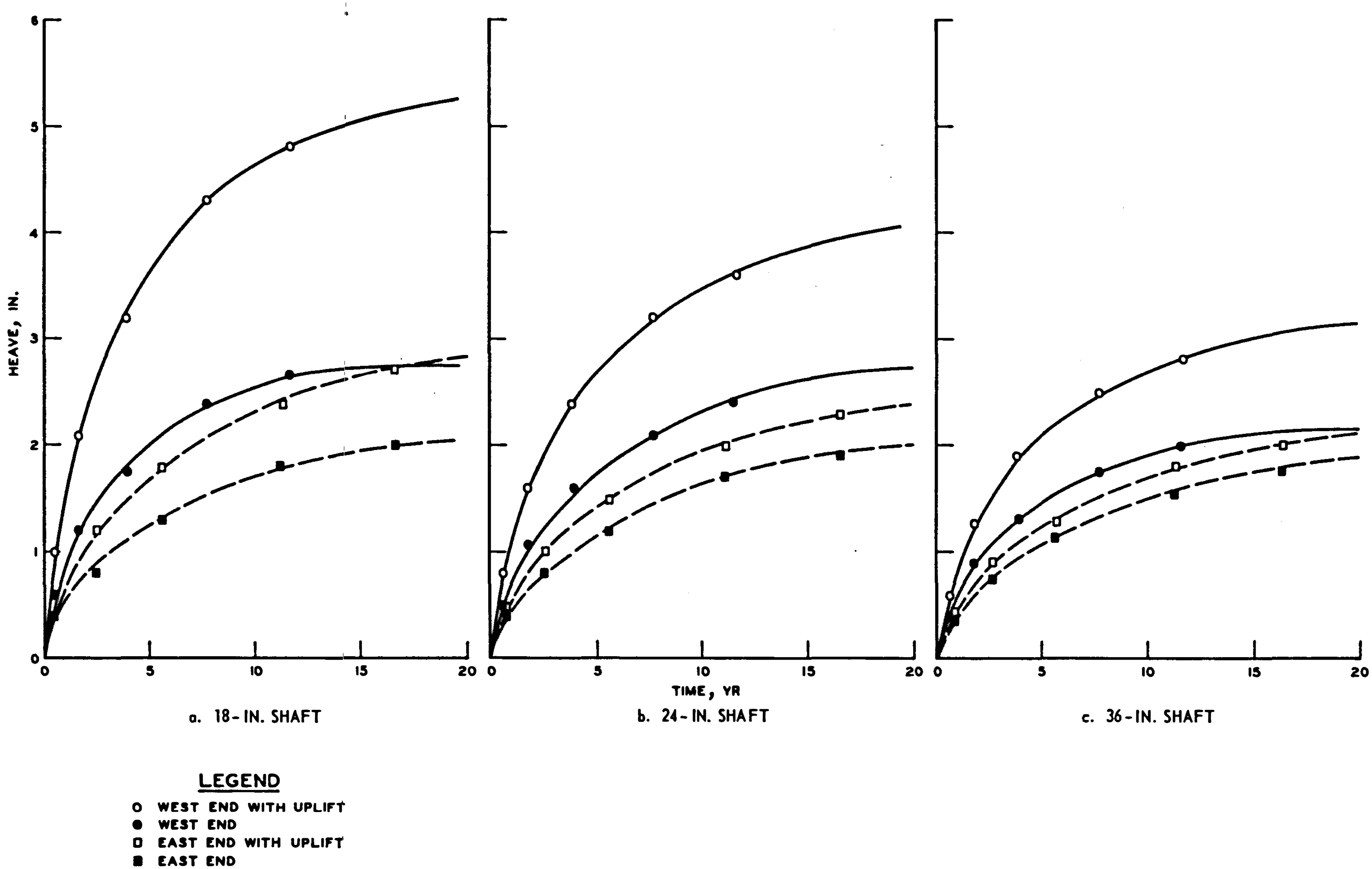

Figure 10. Heave with time of pier foundations with footings 30 ft below ground surface and deep water table for saturation case

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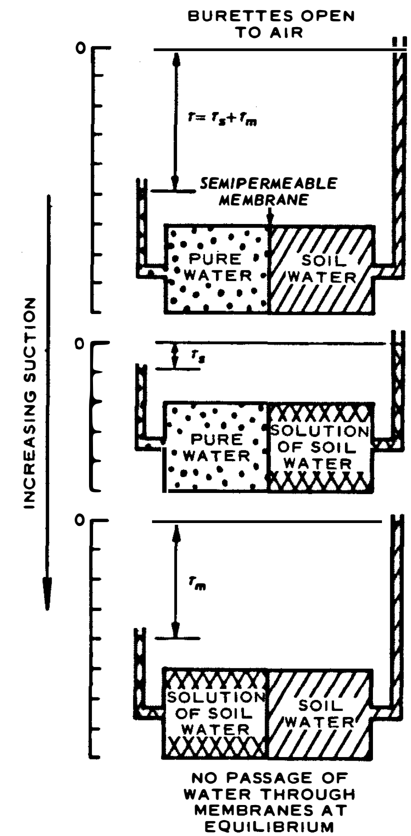
Table 1

Definitions of Suction

Term	Symbol	Definition*	Illustration
Total suction	τ	The negative gage pressure, relative to the external gas pressure** on the soil water, to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable (permeable to water molecules only) membrane with the soil water	
Osmotic (solute) suction	τ_s	The negative gage pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the soil water	
Matrix (soil water) suction	τ_m	The negative gage pressure, relative to the external gas pressure** on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water	

* From Reference 28 of text.

** The magnitude of the matrix suction is reduced by the magnitude of the external gas pressure. The osmotic suction is determined by the concentration of soluble salts in the pore water and can be given by $\tau_s = \frac{RT}{v_w} \log_e \frac{P}{P_o}$ where R is the universal gas constant, T is absolute temperature, v_w is volume of a mole of liquid water, P is vapor pressure of the pore-water extract, and P_o is vapor pressure of free pure water.

Table 2

Saturation of Soil (After Reference 88)

Description	Degree of Saturation percent	Pore-Water Pressure	Pore-Air or Gas Pressure Relative to Atmospheric Pressure
Fully saturated	100	+, 0	No air present
Quasi-saturated	100	--	No air present
Partially saturated	<100	+, 0	+
Unsaturated	<100	--	+
Unsaturated	<100	--	Air drained to atmosphere
Unsaturated	<100	--	Trapped air, +

Table 3

Types of Expansive Soil Foundations (from References 11,19,22,39,57,96-98)

Foundation	Description	Application
Shallow	Continuous wall, individual spot and spread footings	<1/2-in. differential heave, stable stratum, semirigid framing system
Shallow with split construction	Structure built into several independent units, joints between units and in walls, suspended floors, ceiling isolated from walls, reinforced masonry	1/2- to 2-in. differential heave, suitable for wood or reinforced masonry structures
Stiffened mat	On-grade reinforced concrete floor slabs	1/2- to 2-in. differential heave, masonry buildings with load bearing walls or moderate to small column loads, metal structures
Deep (isolation)	Underreamed, reinforced, cast-in-place concrete piers, grade beams span between piers and suspended about 1 ft above ground level; all water pipes and drains into structure equipped with flexible joints	>2-in. differential heave, suitable for split construction or framing system and structural loads resulting in moderate to high column loads, building configuration and functional requirements or economics that preclude a mat foundation

Table 4

Foundation Treatment Methods

Method	Reference	Remarks
Chemical lime cement	19, 103, 104	2-5 percent lime thoroughly mixed is most successful chemical agent. In-place mixing feasible up to 36 in. thick. Montmorillonites should be conditioned with lime if cement is also added
Compaction control	19, 105	Compact by kneading (sheepsfoot roller) to 90-95 percent optimum density at water contents 2-5 percent greater than optimum
Moisture control	19, 21, 106, 107	Horizontal plastic membranes of controversial value. Catalytically blown asphalt membranes effective in minimizing heave below membrane. Ground surface should slope slightly from structure. Drains should not be installed in desiccated soils as moisture from drains will be drawn into soil
Removal and replace with nonexpansive backfill	7, 23	Useful for replacing surface expansive soils to about 4-ft depths. Backfill should be impervious
Ponding	19, 107	Time-consuming, more effective with vertical sand drains or open boreholes to aid water penetration

Table 5

Soil Properties For Example Problems

Depth ft	Specific Gravity	Liquid Limit	Plasticity Index	Water Content %	Swell Test Results* e_o	e_{po}	e_s	tons/sq ft P_o	tons/sq ft S_p	Suction Parameters A	Suction Parameters B	c_{vs} sq ft/day
					Boring PU-7, Dec 1970							
0-4	2.70	57	39	17.9	0.800	0.847	0.855	0.18	2.20	6.75	0.25	0.463
4-8	2.70	60	40	23.8	0.745	0.752	0.770	0.40	0.66	6.75	0.25	0.871
8-12	2.72	27	14	31.0	0.838	0.860	0.910	0.90	2.40	4.20	0.13	0.020
12-30	2.75	78	48	29.7	0.820	0.908	1.060	1.80	10.80	5.00	0.14	0.020
30-40	2.73	82	61	28.0	0.760	0.820	0.960	2.40	9.90	4.40	0.12	0.020
					Boring LAFB 1, April 1973							
0-8	2.69	69	46	31.5	0.930	0.941	0.951	0.36	1.20	6.75	0.25	0.040
8-12	2.72	60	40	23.8	0.745	0.752	0.770	0.50	0.66	4.20	0.13	0.871
12-19	2.76	73	50	31.9	0.902	0.924	0.948	1.00	8.00	4.68	0.13	0.026
19-30	2.75	78	48	30.4	0.820	0.867	0.907	1.60	33.00	5.46	0.15	0.020
30-40	2.73	82	61	30.4	0.793	0.832	0.879	2.30	33.00	4.43	0.12	0.014

* e_o = void ratio at soil overburden pressure P_o prior to addition of free water.

e_{po} = void ratio at soil overburden pressure P_o following rebound from swell.

e_s = void ratio at 0.1 ton/sq ft pressure following rebound from swell pressure S_p .

c_{vs} = coefficient of swell.

APPENDIX A: SWELL TESTS AND PRESSURES FOR BUILDING FOUNDATIONS

Specimen Preparation

1. Standard tests[102*] such as visual description, water content, Atterberg limits, specific gravity, and grain-size distribution with hydrometer should be performed on scraps from each undisturbed specimen to be tested for swell behavior in the consolidometer. Evidence of slickensides and fissures in the soil should be noted.

2. Each undisturbed specimen is to be identically trimmed (i.e., 4.25 in. in diameter by 1.15 in. in height or standard dimensions of available consolidation equipment) and seated in the consolidometer between air-dry porous stones with a small load (approximately 0.02 ton/sq ft). The porous stones should be ground smooth and filter paper should not be used. The seating load (step 1) is to be maintained for not more than 1/2 hr. The inside of the reservoir should be moistened and the specimen and consolidometer assembly covered with impervious thin plastic to maintain constant moisture conditions. The swell tests should be performed with distilled water to simulate sorption of rainwater. The swell observed from swell tests may be small and corrections for deformations of the equipment due to the applied loads may be necessary. The procedures for the following swell tests are suggested as general guidelines.

Constant Volume Swell (CVS) Test (Figure A1)

3. From the seating load, the specimen is loaded to the original soil overburden pressure P_o in one increment and held for 2-4 min and not more than 1/2 hr (step 2) to obtain e_o, the original void ratio; free water is added to the reservoir and sufficient load applied in small increments to prevent swelling until the swelling pressure S_p is

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* Raised numbers refer to similarly numbered items in "References" at end of main text.

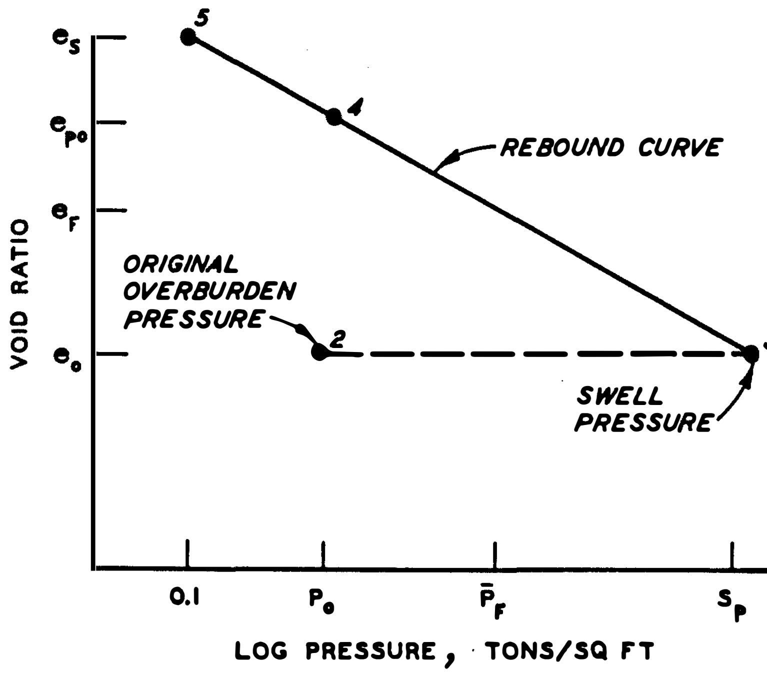

Figure A1. CVS test

fully developed (step 3). Both top and bottom surfaces of the specimen should be subject to free water. The submerged specimen is unloaded to the overburden pressure P_o and the seating pressure (two decrements, steps 4 and 5). Each decrement is held until primary swell is complete as verified by examination of the time versus swell plot.

4. The final effective overburden pressure $\bar{P}_f$ (Figure A1) is calculated from either Equation 8 (saturation case) or Equation 10 (negative hydrostatic head case) depending on the final moisture condition. The final void ratio e_f is obtained from the rebound curve at the $\bar{P}_f$. The results of a CVS test performed on a specimen from 29 to 30 ft of depth at the test pier site of Lackland Air Force Base (LAFB) are shown in Figure A2.

Modified Swell Overburden (MSO) Test (Figure A3)

5. After the specimen is loaded for not more than 1/2 hr under the seating pressure, the specimen is loaded to the overburden pressure

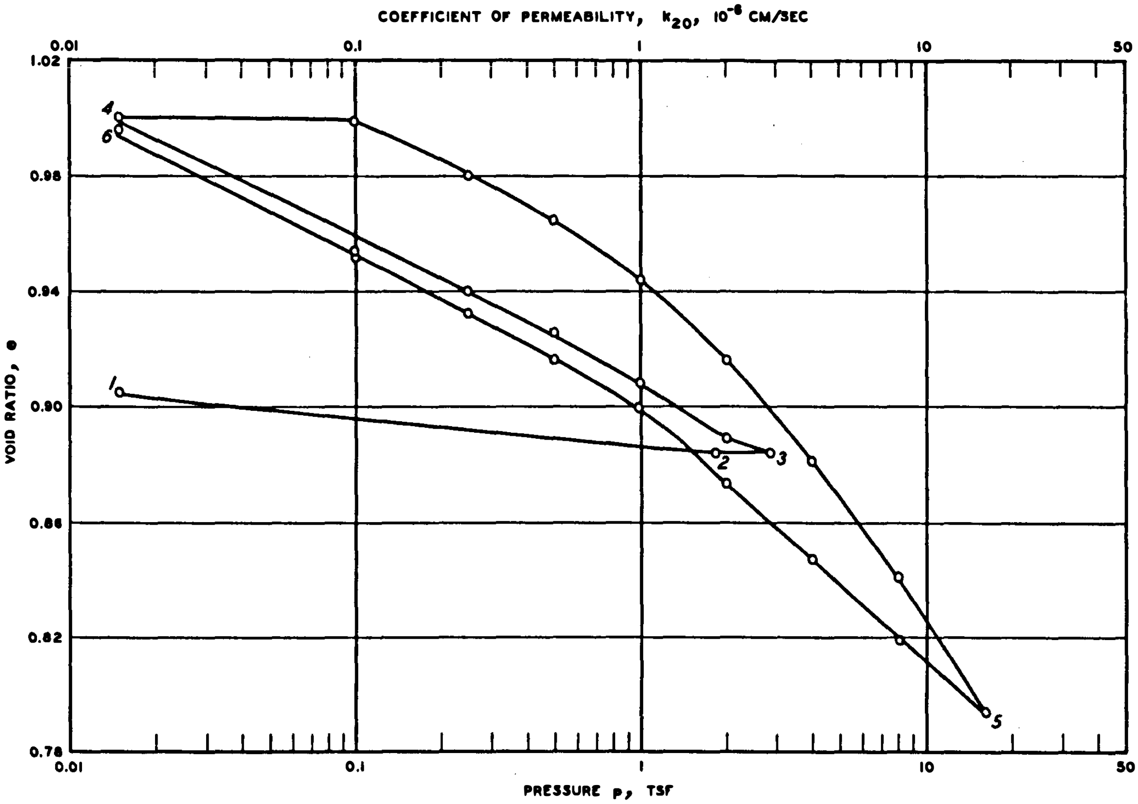

Figure A2. Void ratio-pressure relationship of constant volume swell test, LAFB; boring 1, sample 17, 29-30 ft (8.8-9.1 m)

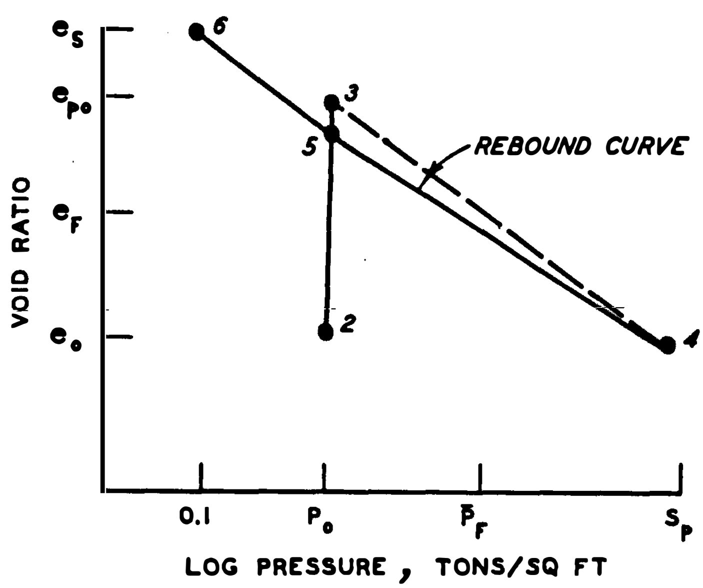

Figure A3. Modified swell overburden test

P_o and held for not more than 1/2 hr to determine e_o (step 2). Distilled water is added to the top and bottom porous stones, and the swell under this overburden pressure is observed until primary swell is complete (step 3), as verified by the time versus swell plot. Increments of load are applied to achieve consolidation until the specimen has consolidated to the void ratio e_o (step 4) to obtain the swell pressure. The sample is rebounded to P_o and the seating pressure (steps 5 and 6). The results of an MSO test are shown in Figure A4, except the swell pressure was not obtained. The time required to achieve primary swell at the overburden pressure (step 2) was about 1440 min or 1 day (Figure A5).

Swell Pressure

6. Swell pressure S_p can develop in clay soils on contact with water and can lead to extensive damages to overlying structures and pavements from the swell that accompanies relief of the swell pressure. Considerable swell pressure, up to 16 tons/sq ft, has been observed depending on the nature of the soil.[112] The magnitude of the swell pressure that can be made to develop in the laboratory depends on the definition in the following tabulation:

Method	Reference	Definition
A	67, 68, 100, 112	Pressure required to bring soil back to the original volume after the soil is allowed to swell completely without surcharge.
B	68	Pressure applied to the soil so that neither swell nor compression takes place on inundation. A specimen may be confined at a fixed volume and pressure inferred from deflection of the confining vessel.
C	68	Pressure necessary to permit no change in volume upon inundation when initially under applied pressure equal to total overburden pressure. Various loads are applied to the soil after inundation to maintain no volume change.
D	67, 68	Pressure required for preventing volume expansion in soil in contact with water. Various loads are applied to the soil after inundation to maintain no volume change.

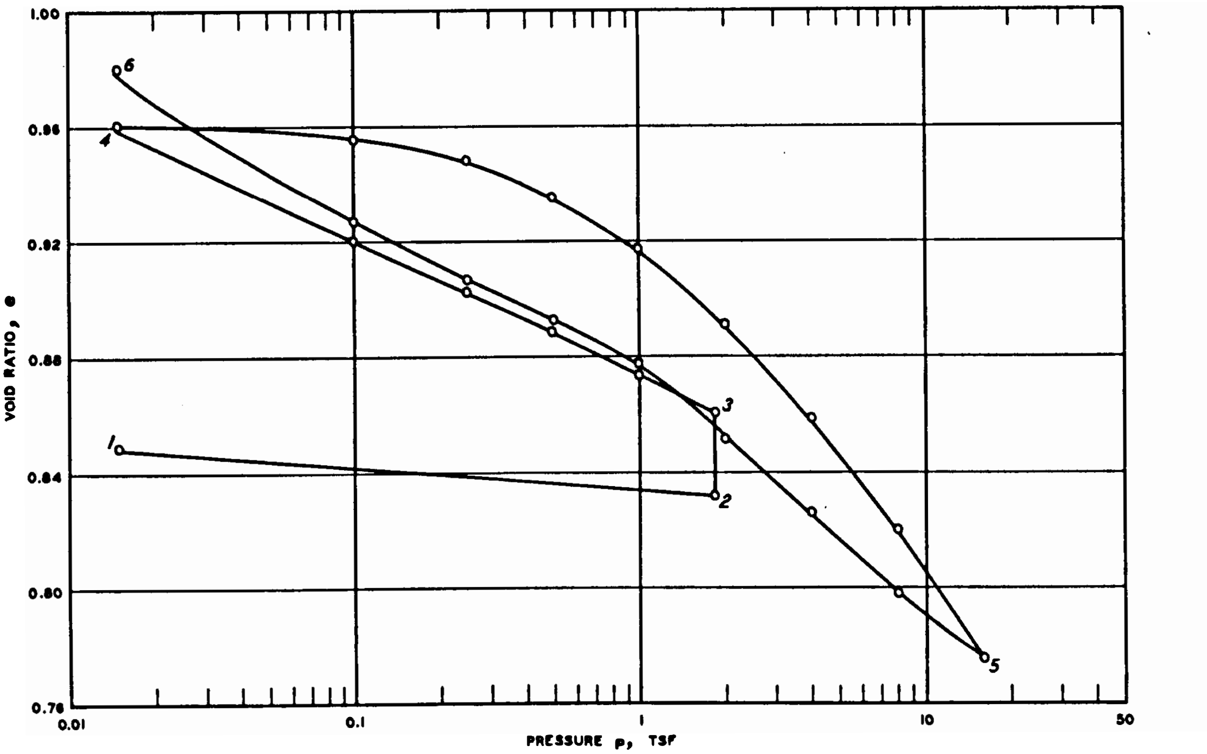

Figure A4. Void ratio-pressure relationship of swell overburden test, LAFB; boring 1, sample 17, 29-30 ft (8.8-9.1 m)

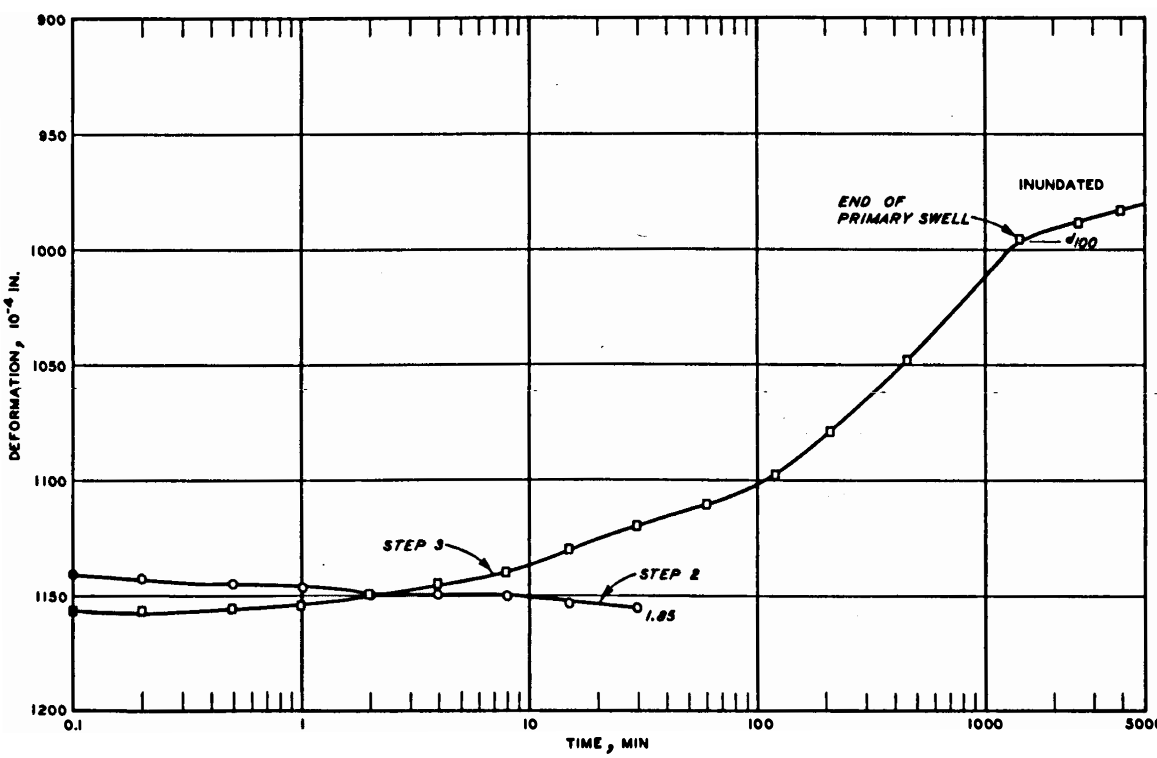

Figure A5. Deformation-time relationship of swell overburden test, LAFB; boring 1, sample 17, 29-30 ft (8.8-9.1 m)

The magnitude of the swell pressure depends on the degree of confinement[113] and usually decreases in the order of method A > B > C > D > flat dynamometer > ring dynamometer apparatus.[114,115] Greater stiffness in the system helps to increase the swell pressure. The porous disk usually used in the consolidometer is relatively compressible and should be replaced with smoothly ground stainless steel or ceramic disks. Filter paper should not be used because this paper undergoes significant compression.[116]

7. The most appropriate definition of swell pressure should be compatible with in situ conditions in the field. In actual field situations, a swell pressure equivalent to the confining soil overburden and lateral pressures may develop when free water is available, but any additional swell pressure will be relieved through soil expansion. The swell pressure, according to method B, could conceivably develop in the field on availability of free water if the in situ confining pressure is just sufficient to prevent any volume change. Excessive confining pressure may lead to collapse or shrinkage, while insufficient confining pressure may lead to swell. The probability of the development in the field of any of the other swell pressures defined above appears unlikely.

8. Swell pressure develops from the hydration of clay platelets and exchangeable cations. This pressure tends to push the soil particles apart.[60] The extent of hydration leading to the development of swell pressure depends on:[67-71,117-121]

- a. Ion concentration in soil solution. Swell pressure decreases with increasing ionic concentration.
- b. Valency of adsorbed cation. Swell pressure decreases with increasing valency.
- c. Temperature. Swell pressure increases with increasing temperature.
- d. Surface charge density of clay mineral. Swell pressure decreases with increasing surface charge density.
- e. Void ratio or dry density. Swell pressure increases with decreasing void ratio or increasing density. Preloading increases swell pressure due to increased density.
- f. Surface tension or suction. Swell pressure increases with increasing suction.

_g_. Structure. Flocculated structure (compacted dry of optimum) exhibits greater swell pressure than dispersed structure (compacted wet of optimum).

9. Swell pressure in undisturbed soil is usually less than that in remolded soils due to the bonds in undisturbed soil.[117] Undisturbed soils may also contain minute fissures allowing some swelling forces to dissipate, thus tending to yield smaller S_p than that in remolded soils.[70] Swell pressure in remolded soil, however, may eventually decrease while aging due to development of bonds from cross-links.[117]

10. Development of swell pressure on inundation with water is time dependent, perhaps extending over a period of 4 to 7 days or longer, because of the slow rate of water sorption, low hydraulic conductivity, readjustment of particles, and specimen size and thickness.[60] Swell pressure may decrease with time after reaching a maximum value due to rearrangement of particles along the direction of water flow or because of interparticle collapse.[60,119]

APPENDIX B: MATRIX SUCTION-WATER CONTENT RELATIONSHIPS

Determination of Matrix Suction by Thermocouple Psychrometers

1. The thermocouple psychrometer measures the relative humidity in the soil by a technique called Peltier cooling. By causing a current to flow through a single thermocouple junction in the proper direction, that particular junction will cool, causing water to condense on it when the dewpoint temperature is reached. Condensation of this water inhibits further cooling of the junction, and the voltage developed between the thermocouple and reference junctions is measured by the proper readout equipment.

2. The output of the thermocouple psychrometer (in microvolts) is calibrated by tests with salt solutions, such as potassium chloride, that produce a given relative humidity for known concentrations. The relative humidities are converted to total suction by[28]

$$\tau^{o} = -\frac{RT}{v_w} \ln \frac{p}{p_o} \tag{B1}$$

where

τ^{o} = total suction free of external pressure except atmospheric pressure, atm

R = universal gas constant, 82.06 cc - atm/Kelvins-mole

T = absolute temperature, Kelvins

v_w = volume of a mole of liquid water, 18.02 cc/mole

p/p_o = relative humidity

p = pressure of water vapor, lb/sq ft

p_o = pressure of saturated water vapor, lb/sq ft

The matrix suction is determined as the difference between osmotic and total suctions (Table 1, main text). The osmotic suction can be estimated by adding distilled water to the soil specimen and evaluating the total suction at high water contents, Figure B1. Hysteretic effects from cyclic changes in water content are ignored.

3. Laboratory measurements to evaluate total suction may be made

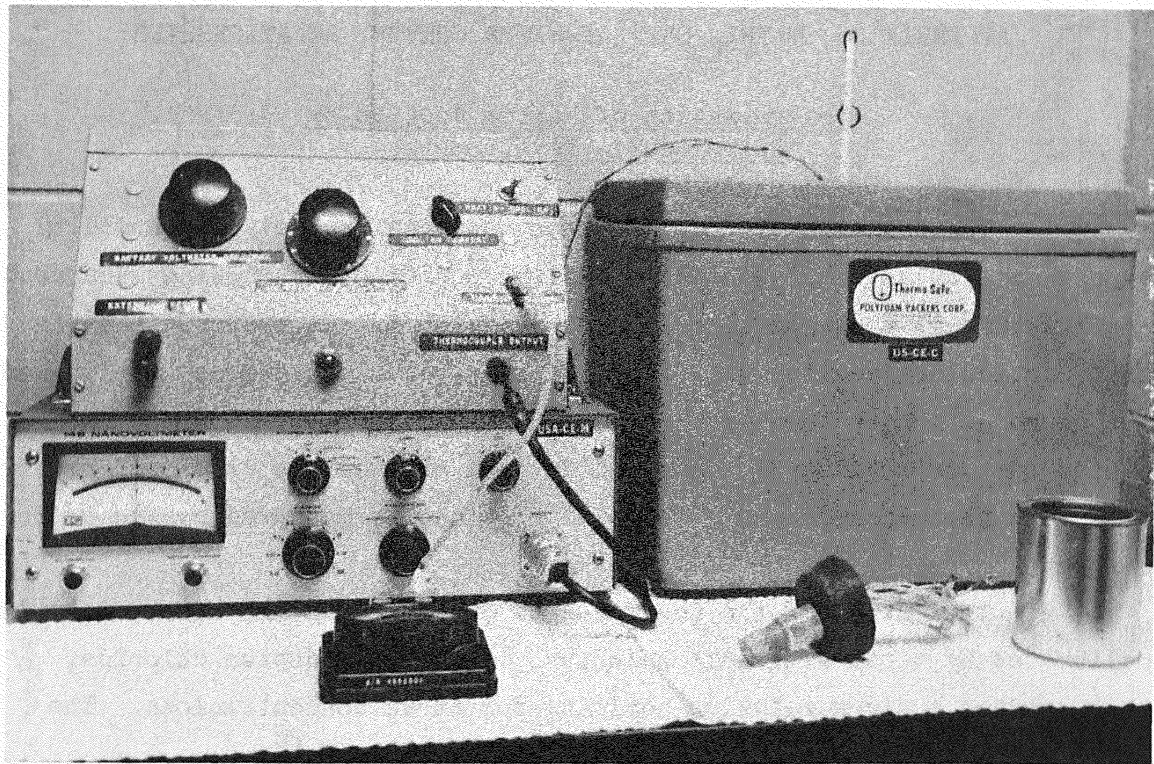

Figure B1. Monitoring system

with the apparatus illustrated in Figure B1. Thermocouple psychrometers are inserted into pint-capacity metal containers with the soil specimens and the assembly sealed with No. 13-1/2 rubber stoppers. The assembly is inserted into a 1- by 1- by 1.25-ft chest capable of holding six pint-sized containers and insulated with 1.5 in. of foamed polystyrene. Cables from the psychrometers are passed through a 0.5-in.-diam hole centered in the chest cover. Temperature equilibrium is attained within a few hours after placing the lid. Equilibrium of the relative humidity in the air measured by the psychrometer and the relative humidity in the soil specimen is usually obtained within 24-48 hr. Further details for evaluating total suction by this procedure are available in Reference 92.

Some Matrix Suction-Water Content Relationships

4. Matrix suction free of external pressure-water content relationships were evaluated for some expansive clay soils from Fort Carson,

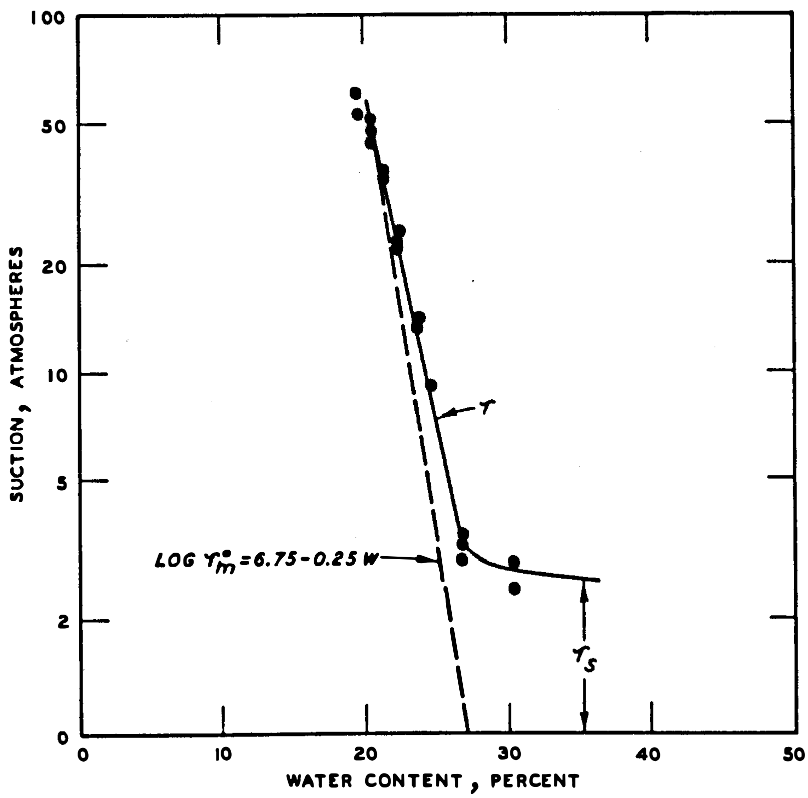

Figure B2. Suction-water content relationship of Lackland soil at 3.2-4.2 ft

Colo.; Jackson, Miss.; and Lackland Air Force Base, Tex. (Table B1). The data were plotted as indicated in Figure B2. The results can be expressed by the empirical equation

$$\log \tau_m^o = A - Bw \tag{B2}$$

where

τ_m^o = matrix suction, free of external pressure except atmospheric pressure, atm

A,B = parameters

w = water content, percent

within a limited range of suction near the natural water content. The suction parameters may be related with the Atterberg limits and natural water contents (Figure B3) and given by

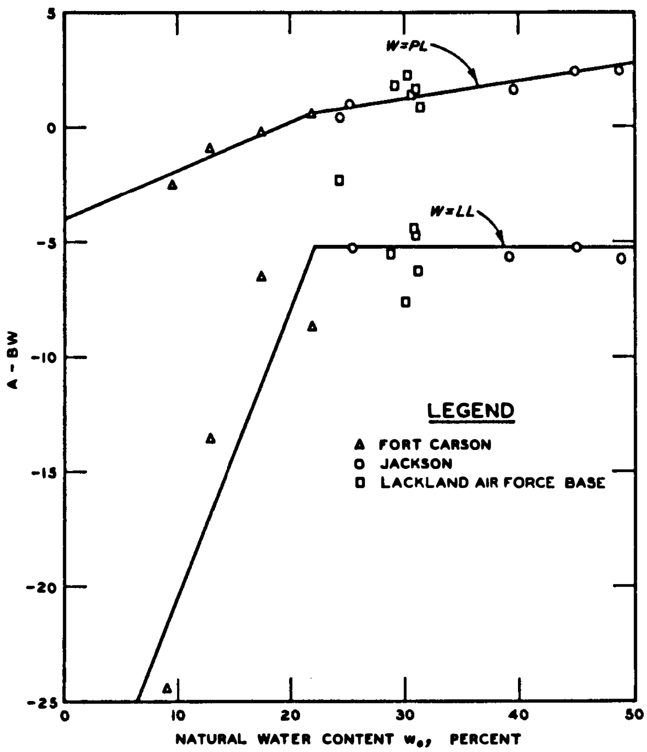

Figure B3. Relationships of suction parameters

$$\left.\begin{aligned} A &= -33 + 1.25w_o + 29\,\frac{LL}{PI} - 1.045w_o\,\frac{LL}{PI} \\ B &= \frac{29 - 1.045w_o}{PI} \end{aligned}\right\} 0 < w_o \leq 22 \qquad \text{(B3)}$$

$$\left.\begin{aligned} A &= -5.5 + 4.5\,\frac{LL}{PI} + 0.075w_o\,\frac{LL}{PI} \\ B &= \frac{4.5 + 0.075w_o}{PI} \end{aligned}\right\} w_o > 22 \qquad \text{(B4)}$$

where

w_o = natural water content, percent

LL = liquid limit

PI = plasticity index

Additional suction tests are needed to determine relationships for general applications.

Table B1

A and B Factors for Some Expansive Clay Soils

Depth, ft	Natural Water Content percent	Plasticity Index	Liquid Limit	Dry Density lb/cu ft	A	B
		Fort Carson BOQ-3				
5.7- 7.0	22.0	30	49	102	6.38	0.31
14.7-15.7	17.3	21	43	121	4.25	0.25
24.7-26.0	13.0	51	70	103	3.85	0.25
34.2-35.2	9.6	54	73	133	5.21	0.41
		Jackson, Miss.				
3.5- 4.9	24.2	21	42	96	3.12	0.13
6.0- 7.0	24.7	48	68	88	3.67	0.13
10.1-11.1	39.5	72	97	77	4.10	0.10
16.1-17.1	48.8	82	111	72	5.28	0.10
30.0-31.2	45.0	70	100	76	5.64	0.11
		Lackland Air Force Base				
3.2- 4.2	30.1	39	57	83	6.75	0.25
14.3-15.3	31.0	50	73	88	4.68	0.13
27.3-28.3	31.2	48	78	92	5.46	0.15
37.4-38.7	28.9	61	82	92	4.43	0.12
46.5-47.4	30.8	50	74	93	4.24	0.12

APPENDIX C: COMPUTER CODE

Input Data

Line (card) 1

NWAT = option for water table; = 0 for shallow, = 1 for deep.

NSUCT = option for suction parameters A and B; = 0 if not used or generated by code; = 1 for input A,B.

NBPRES = option for footing; = 1 for circular, = 2 for rectangular, = 3 for long continuous.

OPTION = option for moisture condition; = 0 for saturated, = 1 for hydrostatic.

NRATE = option for rate of heave; = 0 if not computed, = 1 if computed.

NNP = total number of nodal points.

NBX = number of nodal points at bottom of footing.

NMAT = total number of soils.

NPROB = number of Q loading cases.

Line (card) 2

Read in description of problem and/or loading case.

Line (card) 3

DX = increment of depth, ft.

Q = structure pressure at bottom of footing, lb/sq ft.

BLEN = radius of footing, ft, if NBPRES = 1; = length of footing, ft, if NBPRES = 2; = 0.0 if NBPRES = 3.

BWID = 0.0 if NBPRES = 1; = width of footing, ft, if NBPRES = 2,3.

Line (card) 4 to
line (card) 3 + NMAT*

M = number of the soil.

G = specific gravity.

LL = liquid limit.

PI = plasticity index.

---

* If NWAT = 1, OPTION = 1, and NSUCT = 1, then number of the soil and suction parameters A and B must be read in on the line following the data on each soil.

WC = initial water content, percent.

EO = initial void ratio at pressure PO.

EPO = void ratio at PO on rebound curve from SP.

ES = void ratio at 0.1 ton/sq ft on rebound from SP.

PO = original surcharge pressure on soil specimen, tons/sq ft.

SP = swell pressure of soil, tons/sq ft.

CVS = coefficient of swell, sq ft/day.

Lines (cards) following soil data

These lines denote the number of the soil that belongs to the element N.

N = number of soil element.

M = number of soil in the element.

The number of the first element and the soil type must be read on individual lines for each succeeding stratum. The last element number and the number of the soil type in the deepest stratum must also be read in on a line, which is also the last line of input data.

Output Data

Output data are in the form of:

```
I  DEPTH, FT  DEL VOL/VOL     EXCESS SWELL PRESSURE, TON/FT2

DELH = FEET,

TIME,DAYS   DELH,FT
```

I = number of element.

DEPTH, FT = depth of center of element I, ft.

DEL VOL/VOL = fractional change in volume of element I.

EXCESS SWELL PRESSURE, TON/FT2 = difference between swell pressure and total surcharge pressure in element I. If NWAT = 1 and OPTION = 1, the in situ suction pressure is listed.

DELH = heave at ground surface, ft.

TIME,DAYS = time needed to heave DELH for sorption from one surface, days.

Example Problems

Slab foundations

1. Slab at ground surface.

```
*RUN
SOURCE LINE    1930
<W>1470 EQUALITY OR NON-EQUALITY COMPARISON MAY NOT BE MEANINGFUL I
N LOGICAL IF EXPRESSIONS
NWAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB
=0,0,2,0,1,17,1,2,1
=SLAB AT G.S. - SHALLOW TABLE AT 8 FT - SAT CASE - PU-7
 DX,Q,BLEN,BWID
=0.5,144,100,100
M,G,ALL,PI,WC,EO,EPO,ES,PO,SP,CVS
=1,2.7,57,39,17.9,.8,.847,.855,.18,2.2,.463
=2,2.7,60,40,23.8,.745,.752,.77,.4,.66,.871
ELEMENT,NO. OF SOIL
=1,1
=9,2
=16,2

 I    DEPTH,FT    DEL VOL/VOL    EXCESS SWELL PRESSURE,TON/FT2
 1     0.3         0.358E-01      0.215E 01
 2     0.8         0.372E-01      0.216E 01
 3     1.3         0.334E-01      0.213E 01
 4     1.8         0.308E-01      0.210E 01
 5     2.3         0.289E-01      0.208E 01
 6     2.8         0.274E-01      0.205E 01
 7     3.3         0.261E-01      0.202E 01
 8     3.8         0.247E-01      0.199E 01
 9     4.3         0.795E-02      0.424E 00
10     4.8         0.707E-02      0.395E 00
11     5.3         0.628E-02      0.365E 00
12     5.8         0.556E-02      0.335E 00
13     6.3         0.491E-02      0.305E 00
14     6.8         0.431E-02      0.276E 00
15     7.3         0.374E-02      0.246E 00
16     7.8         0.318E-02      0.216E 00
 DELH=     0.144E 00    FEET
 TIME,DAYS   DELH,FT

0.353E 01 0.287E-01
0.143E 02 0.575E-01
0.327E 02 0.862E-01
0.645E 02 0.115E 00
0.965E 02 0.129E 00
```

```
*RUN
SOURCE LINE   1930
<W>1470 EQUALITY OR NON-EQUALITY COMPARISON MAY NOT BE MEANINGFUL I
N LOGICAL IF EXPRESSIONS
NWAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB
=0,0,2,1,1,17,1,2,1
=SLAB AT G.S. - SHALLOW WATER TABLE - HYD CASE - PU-7
 DX,Q,BLEN,BWID
=0.5,144,100,100
M,G,ALL,PI,WC,EO,EPO,ES,PO,SP,CVS
=1,2.7,57,39,17.9,.8,.847,.855,.18,2.2,.463
=2,2.7,60,40,23.8,.745,.752,.77,.4,.66,.871
ELEMENT,NO. OF SOIL
=1,1
=9,2
=16,2

 I    DEPTH,FT    DEL VOL/VOL    EXCESS SWELL PRESSURE,TON/FT2
 1     0.3          0.211E-01       0.191E 01
 2     0.8          0.220E-01       0.193E 01
 3     1.3          0.215E-01       0.192E 01
 4     1.8          0.211E-01       0.191E 01
 5     2.3          0.207E-01       0.190E 01
 6     2.8          0.203E-01       0.188E 01
 7     3.3          0.199E-01       0.187E 01
 8     3.8          0.195E-01       0.186E 01
 9     4.3          0.495E-02       0.307E 00
10     4.8          0.466E-02       0.293E 00
11     5.3          0.438E-02       0.279E 00
12     5.8          0.410E-02       0.265E 00
13     6.3          0.383E-02       0.251E 00
14     6.8          0.356E-02       0.237E 00
15     7.3          0.330E-02       0.223E 00
16     7.8          0.304E-02       0.209E 00
 DELH=      0.989E-01    FEET
 TIME,DAYS   DELH,FT

0.353E 01 0.198E-01
0.143E 02 0.395E-01
0.327E 02 0.593E-01
0.645E 02 0.791E-01
0.965E 02 0.890E-01
```

2. <u>Slab in excavation.</u>

```
*RUN
SOURCE LINE   1930
<W>1470 EQUALITY OR NON-EQUALITY COMPARISON MAY NOT BE MEANINGFUL I
N LOGICAL IF EXPRESSIONS
NWAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB
=1,0,2,0,1,45,25,4,1
=SLAB AT 12 FT DEPTH - DEEP WATER TABLE - SAT CASE - PU-7
 DX,Q,BLEN,BWID
=0.5,144,100,1[illegible]3
M,G,ALL,PI,WC,EO,EPO,ES,PO,SP,CVS
=1,2.7,57,39,17.9,.8,.847,.855,.18,2.2,.463
=2,2.7,60,40,23.8,.745,.752,.77,.4,.66,.871
=3,2.72,27,14,31,.838,.86,.91,.9,2.4,.02
=4,2.75,78,48,29.7,.82,.908,1.06,1.8,10.8,.02
ELEMENT,NO. OF SOIL
=1,1
=9,2
=17,3
=25,4
=44,4

 I    DEPTH,FT    DEL VOL/VOL    EXCESS SWELL PRESSURE,TON/FT2
25    12.3          0.916E-01        0.104E 02
26    12.8          0.737E-01        0.101E 02
27    13.3          0.726E-01        0.100E 02
28    13.8          0.715E-01        0.999E 01
29    14.3          0.704E-01        0.996E 01
30    14.8          0.693E-01        0.993E 01
31    15.3          0.683E-01        0.990E 01
32    15.8          0.673E-01        0.987E 01
33    16.3          0.664E-01        0.984E 01
34    16.8          0.655E-01        0.980E 01
35    17.3          0.646E-01        0.977E 01
36    17.8          0.637E-01        0.974E 01
37    18.3          0.628E-01        0.971E 01
38    18.8          0.620E-01        0.968E 01
39    19.3          0.611E-01        0.964E 01
40    19.8          0.603E-01        0.961E 01
41    20.3          0.596E-01        0.958E 01
42    20.8          0.588E-01        0.955E 01
43    21.3          0.580E-01        0.951E 01
44    21.8          0.573E-01        0.948E 01
 DELH=     0.662E 00    FEET
 TIME,DAYS  DELH,FT

0.155E 03 0.132E 00
0.630E 03 0.265E 00
0.143E 04 0.397E 00
0.283E 04 0.530E 00
0.424E 04 0.596E 00
```

```
*RUN
SOURCE LINE   1930
<W>1470 EQUALITY OR NON-EQUALITY COMPARISON MAY NOT BE MEANINGFUL I
N LOGICAL IF EXPRESSIONS
NWAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB
=1,1,2,1,1,45,25,4,1
=SLAB AT 12 FT DEPTH - DEEP TABLE - HYD CASE - PU-7
 DX,Q,BLEN,BWID
=0.5,144,100,100
M,G,ALL,PI,WC,EO,EPO,ES,PO,SP,CVS
=1,2.7,57,39,17.9,.8,.847,.855,.18,2.2,.463
 M,A,B
=1,6.2,.25
=2,2.7,60,40,23.8,.745,.752,.77,.4,.66,.871
 M,A,B
=2,6.2,.25
=3,2.72,27,14,31,.838,.86,.91,.9,2.4,.02
 M,A,B
=3,4.2,.127
=4,2.75,78,48,29.7,.82,.908,1.06,1.8,10.8,.02
 M,A,B
=4,5.0,.14
ELEMENT,NO. OF SOIL
=1,1
=9,2
=17,3
=25,4
=44,4

 I    DEPTH,FT    DEL VOL/VOL    EXCESS SWELL PRESSURE,TON/FT2
25    12.3         0.413E-02        0.115E 02
26    12.8         0.186E-02        0.109E 02
27    13.3         0.176E-02        0.104E 02
28    13.8         0.167E-02        0.986E 01
29    14.3         0.157E-02        0.936E 01
30    14.8         0.147E-02        0.889E 01
31    15.3         0.137E-02        0.845E 01
32    15.8         0.127E-02        0.805E 01
33    16.3         0.117E-02        0.767E 01
34    16.8         0.107E-02        0.733E 01
35    17.3         0.967E-03        0.702E 01
36    17.8         0.866E-03        0.674E 01
37    18.3         0.764E-03        0.649E 01
38    18.8         0.662E-03        0.627E 01
39    19.3         0.560E-03        0.608E 01
40    19.8         0.457E-03        0.592E 01
41    20.3         0.354E-03        0.580E 01
42    20.8         0.251E-03        0.571E 01
43    21.3         0.149E-03        0.564E 01
44    21.8         0.480E-04        0.561E 01
 DELH=     0.112E-01    FEET
 TIME,DAYS   DELH,FT

0.155E 03 0.224E-02
0.630E 03 0.448E-02
0.143E 04 0.672E-02
0.283E 04 0.896E-02
0.424E 04 0.101E-01
```

Deep foundations

```
*RUN
SOURCE LINE    1930
<W>1470 EQUALITY OR NON-EQUALITY COMPARISON MAY NOT BE MEANINGFUL I
N LOGICAL IF EXPRESSIONS
NMAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB
=1,0,1,0,1,41,31,5,6
=PIER FOUNDATION - 18 IN. SHAFT - SAT CASE -              PU-7
 DX,Q,BLEN,BWID
=1.0,12000,2.25,0.0
M,G,ALL,PI,WC,EO,EPO,ES,PO,SP,CVS
=1,2.7,57,39,17.9,.8,.847,.855,.18,2.2,.463
=2,2.7,60,40,23.8,.745,.752,.77,.4,.66,.871
=3,2.72,27,14,31.0,.838,.86,.91,.9,2.4,.02
=4,2.75,78,48,29.7,.82,.908,1.06,1.8,10.8,.02
=5,2.73,82,61,28.0,.76,.82,.96,2.4,9.9,.02
ELEMENT,NO. OF SOIL
=1,1
=5,2
=9,3
=13,4
=31,5
=40,5

 I    DEPTH,FT    DEL VOL/VOL    EXCESS SWELL PRESSURE,TON/FT2
31    30.5          0.125E-01       0.401E 01
32    31.5          0.149E-01       0.456E 01
33    32.5          0.192E-01       0.543E 01
34    33.5          0.233E-01       0.615E 01
35    34.5          0.264E-01       0.660E 01
36    35.5          0.285E-01       0.687E 01
37    36.5          0.298E-01       0.703E 01
38    37.5          0.305E-01       0.712E 01
39    38.5          0.309E-01       0.716E 01
40    39.5          0.310E-01       0.718E 01
 DELH=      0.247E 00     FEET
 TIME,DAYS   DELH,FT

0.155E 03 0.494E-01
0.630E 03 0.988E-01
0.143E 04 0.148E 00
0.283E 04 0.198E 00
0.424E 04 0.222E 00
```

=PIER FOUNDATIPN - 18 IN. SHAFT WITH UPLIFT - SAT CASE - PU-7
DX,Q,BLEN,BWID
=1.0,1333,2.25,0.0

I	DEPTH,FT	DEL VOL/VOL	EXCESS SWELL PRESSURE,TON/FT2
31	30.5	0.637E-01	0.916E 01
32	31.5	0.569E-01	0.894E 01
33	32.5	0.498E-01	0.862E 01
34	33.5	0.450E-01	0.835E 01
35	34.5	0.420E-01	0.815E 01
36	35.5	0.399E-01	0.800E 01
37	36.5	0.383E-01	0.787E 01
38	37.5	0.371E-01	0.777E 01
39	38.5	0.361E-01	0.768E 01
40	39.5	0.351E-01	0.760E 01

DELH= 0.444E 00 FEET

TIME,DAYS	DELH,FT
0.155E 03	0.888E-01
0.630E 03	0.178E 00
0.143E 04	0.266E 00
0.283E 04	0.355E 00
0.424E 04	0.400E 00

=PIER FOUNDATION - 24 IN. SHAFT - SAT CASE - PU-7
DX,Q,BLEN,BWID
=1.0,12000,3.0,0.0

I	DEPTH,FT	DEL VOL/VOL	EXCESS SWELL PRESSURE,TON/FT2
31	30.5	0.122E-01	0.394E 01
32	31.5	0.134E-01	0.423E 01
33	32.5	0.162E-01	0.485E 01
34	33.5	0.195E-01	0.550E 01
35	34.5	0.225E-01	0.602E 01
36	35.5	0.249E-01	0.638E 01
37	36.5	0.266E-01	0.663E 01
38	37.5	0.278E-01	0.679E 01
39	38.5	0.286E-01	0.689E 01
40	39.5	0.291E-01	0.695E 01

DELH= 0.221E 00 FEET

TIME,DAYS	DELH,FT
0.155E 03	0.442E-01
0.630E 03	0.884E-01
0.143E 04	0.133E 00
0.283E 04	0.177E 00
0.424E 04	0.199E 00

<u>Listing</u>

```
C PREDICTION OF ULTIMATE AND RATE OF HEAVE
C BASED ON CONSTANT VOLUME SWELL OR SWELL OVERBURDEN TESTS
C DEVELOPED BY L. D. JOHNSON
C NPROB=NUMBER OF  Q  LOADING CASES
C Q=STRUCTURE PRESSURE AT BOTTOM OF FOOTING,LB/FT2
C  NWAT=1 FOR DEEP OR 0 FOR SHALLOW WATER TABLE
C NSUCT = 1 FOR INPUT  A,B; = 0 IF GENERATED OR NOT USED
C NBPRES=1 FOR CIRCULAR FOOTING, 2 FOR RECTANGULAR FOOTING,
C OR  3  FOR LONG CONTINUOUS FOOTING
C OPTION=1 FOR HYDROSTATIC CASE OR  0  FOR SATURATED CASE
C NRATE=1 FOR RATE OF HEAVE OR  0  FOR NO RATE OUTPUT
C NNP=NO. OF NODAL POINTS;NBX=NODAL POINT AT FOOTING BOTTOM
C NMAT=NO. OF MATERIALS;DX=INCREMENT OF DEPTH,FT;
C BLEN=RADIUS(NBPRES=1),=LENGTH OF FOOTING(NBPRES=2),=0.0
C (NBPRES=3);IN FEET;BWID=0.0(NBPRES=1),=WIDTH OF FOOTING
C (NBPRES=2,3);M=NO. OF SOIL FROM 1 TO NMAT; G=SP. GRAVITY
C WC=WATER CONTENT,X; EO=INITIAL VOID RATIO AT PRESSURE
C PO; EPO=VOID RATIO AT PO AFTER SATURATION; ES=VOID RATIO
C AT .1 TON/FT2 AFTER SATURATION; PO=ORIGIONAL OVERBURDEN
C PRESSURE,TON/FT2; SP=SWELL PRESSURE,TON/FT2; CVS=
C COEFFICIENT OF SWELL,FT2/DAY; A,B=SUCTION PARAMETERS;
C IE(N,1)=NO OF SOIL  M  OF ELEMENT  N
  DIMENSION A(10),B(10),G(10),WC(10),EO(10),EPO(10),ES(10),
& PO(10),SP(10),P(81),IE(81,1),CVS(10),AMV(10),ALL(10),PI(10)
  GAW=62.43
  NP=1
  PRINT 5
  5 FORMAT(49H NWAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB)
  READ,NWAT,NSUCT,NBPRES,OPTION,NRATE,NNP,NBX,NMAT,NPROB
  400 READ 4
  4 FORMAT(30H
  PRINT 10                                                    )
  10 FORMAT(20H  DX,Q,BLEN,BWID        )
  READ,DX,Q,BLEN,BWID
  IF(NP.GT.1)GO TO 115
  PRINT 15
  15 FORMAT(35H M,G,ALL,PI,WC,EO,EPO,ES,PO,SP,CVS )
  16 READ,M,G(M),ALL(M),PI(M),WC(M),EO(M),EPO(M),ES(M),
& PO(M),SP(M),CVS(M)
  IF(OPTION.EQ.1.AND.NWAT.EQ.1)GO TO 25
  GO TO 20
  25 IF(NSUCT.EQ.0)GO TO 22
  PRINT 8
  8 FORMAT(10H  M,A,B      )
  READ,M,A(M),B(M)
  GO TO 20
  22 IF(WC(M).LE.22.0)GO TO 23
  A(M)=-5.5+(4.5*ALL(M)/PI(M))+(0.075*WC(M)*ALL(M)/PI(M))
  B(M)=(4.5+0.075*WC(M))/PI(M)
  GO TO 20
  23 TEMP=-33.0+1.25*WC(M)+(29.0*ALL(M)/PI(M))
  A(M)=TEMP-(1.045*WC(M)*ALL(M)/PI(M))
```

```
      B(M)=(29.0-1.045*WC(M))/PI(M)
   20 IF(NMAT-M)26,27,16
   26 PRINT 17,M
   17 FORMAT(20H  ERROR IN MATERIAL   ,I5)
      STOP
   27 NEL=NNP-1
      L=0
      PRINT 30
   30 FORMAT(20HELEMENT,NO. OF SOIL     )
   40 READ,N,IE(N,1)
   50 L=L+1
      IF(N-L)60,60,70
   70 IE(L,1)=IE(L-1,1)
      GO TO 50
   60 IF(NEL-L)80,80,40
   80 CONTINUE
C     CALCULATION OF SURCHARGE PRESSURE
  115 P(1)=0.0
      DO 110 I=2,NNP
      MTYP=IE(I-1,1)
      WCC=WC(MTYP)/100.
      GAMM=G(MTYP)*GAW*(1.+WCC)/(1.+EO(MTYP))
      P(I)=P(I-1)+DX*GAMM
  110 CONTINUE
      DXX=0.0
      BPRES=Q-P(NBX)
      DO 120 I=NBX,NNP
      IF(NBPRES.EQ.1)GO TO 122
      IF(NBPRES.EQ.3)GO TO 127
      IF(DXX.LT.0.01)GO TO 123
      AN=BLEN/(4.0*DXX)
      AM=BWID/(4.0*DXX)
      ANM=AN*AN*AM*AM
      BNM=AM*AM+AN*AN+1.0
      CNM=2.0*AM*AN*(BNM**0.5)
      DNM=CNM/(BNM+ANM)
      ENM=DNM*(BNM+1.0)/BNM
      FNM=CNM/(BNM-ANM)
      P(I)=P(I)+BPRES*(ENM+ATAN(FNM))/3.1416
      GO TO 125
  122 IF(DXX.LT.0.01)GO TO 123
      PS=1.+(BLEN/DXX)**2.0
      PS=PS**1.5
      P(I)=P(I)+BPRES*(1.0-1./PS)
      GO TO 125
  127 PS=-.25*DXX/BWID
      PS=10.**PS
      P(I)=P(I)+BPRES*PS
      GO TO 125
  123 P(I)=P(I)+BPRES
  125 IF(OPTION.EQ.1.AND.NWAT.EQ.0)GO TO 130
      GO TO 140
```

```
      130 CONTINUE
          NN=NNP-1
          BN=NN
          P(I)=P(I)+BN*DX*GAW
      140 DXX=DXX+DX
          P(I)=P(I)/2000.
      120 CONTINUE
          IF(OPTION.EQ.1.AND.NWAT.EQ.1)GO TO 145
          GO TO 150
      145 NTYP=IE(NEL,1)
          SNNP=A(NTYP)-B(NTYP)*WC(NTYP)
          SNNP=10.**SNNP
          IF(PI(NTYP).LE.5.)ALPHA=0.0
          API=PI(NTYP)
          IF(API.GT.5.0.AND.API.LE.40.)ALPHA=0.0275*API-0.125
          IF(PI(NTYP).GT.40.)ALPHA=1.0
          SI=SNNP-ALPHA*P(NNP)
          IF(SI.LE.0.0)GO TO 300
      150 NN=NBX-1
          AN=NN
          DXX=AN*DX+DX/2.
C         ALL SATURATED CASES AND HYDROSTATIC CASE WITH SHALLOW TABLE
C         NNP AT TABLE DEPTH FOR SHALLOW TABLE
          PRINT 190
      190 FORMAT(/,43H   I    DEPTH,FT    DEL VOL/VOL    EXCESS SWELL,
     &  17H PRESSURE,TON/FT2,/)
          DELH=0.0
          NEL=NNP-1
          DO 200 I=NBX,NEL
          MTYP=IE(I,1)
          PR=(P(I)+P(I+1))/2.0
          IF(PR.GT.SP(MTYP))GO TO 211
          C1=0.1/PO(MTYP)
          CS1=(ES(MTYP)-EPO(MTYP))/ALOG10(C1)
          IF(PO(MTYP).LT.0.1)GO TO 205
          C2=PO(MTYP)/SP(MTYP)
          CS2=(EPO(MTYP)-EO(MTYP))/ALOG10(C2)
          GO TO 207
      205 C2=0.1/SP(MTYP)
          CS2=(ES(MTYP)-EO(MTYP))/ALOG10(C2)
      207 CONTINUE
          CSS=(CS1+CS2)/2.0
          CSCH=0.0027*SI(MTYP)
          IF(CSS.LE.CSCH)CSS=CSCH
          AMV(MTYP)=(0.435*CSS)/((1.0+EO(MTYP))*PO(MTYP))
          IF(PO(MTYP).LT.0.1)GO TO 215
          IF(PR.GT.PO(MTYP))GO TO 210
          C3=PR/PO(MTYP)
          E=EPO(MTYP)+CSS*ALOG10(C3)
          GO TO 220
      215 IF(PR.GE.0.1)GO TO 210
          C3=PR/0.1
```

```
      E=ES(MTYP)+CS1*ALOG10(C3)
      GO TO 220
  210 C4=PR/SP(MTYP)
      E=E0(MTYP)+CS2*ALOG10(C4)
      GO TO 220
  211 CC=0.007*(ALL(MTYP)-10.)
      CS=PR/SP(MTYP)
      E=E0(MTYP)-CC*ALOG10(CS)
  220 DEL=(E-E0(MTYP))/(1.+E0(MTYP))
      DELP=SP(MTYP)-PR
      IF(DEL.LT.0.0)GO TO 250
      IF(OPTION.EQ.1.AND.NWAT.EQ.1)GO TO 240
      GO TO 250
C     HYDROSTATIC CASE AND DEEP WATER TABLE
  240 NN=NNP-I
      NO=NNP-I-1
      BN=NN
      BO=NO
      PR1=P(I)+(BN*DX*GAW)/2000.
      PR2=P(I+1)+(30*DX*GAW)/2000.
      PR=(PR1+PR2)/2.
      IF(PI(MTYP).LE.5.)ALPHA=0.0
      API=PI(MTYP)
      IF(API.GT.5.0.AND.API.LE.40.)ALPHA=0.0275*API-0.125
      IF(PI(MTYP).GT.40.)ALPHA=1.0
      S=SI+ALPHA*PR
      WCON=(A(MTYP)-ALOG10(S))/B(MTYP)
      VTI=(1.+E0(MTYP))/G(MTYP)
      VTP=(1.+E)/G(MTYP)
      WMAX=((VTP*G(MTYP)-1.)*100.)/G(MTYP)
      QQ=(WMAX-WC(MTYP))/((VTP-VTI)*100.)
      WR=(WCON-WC(MTYP))/(WMAX-WC(MTYP))
      IF(WR.LT.0.0)GO TO 242
      GO TO 244
  242 WR=(WC(MTYP)-WCON)/(WMAX-WCON)
      WW=WR**QQ
      VT=(VTI-VTP*WW)/(1.0-WW)
      GO TO 246
  244 WW=WR**QQ
      VT=VTI+(VTP-VTI)*WW
  246 DEL=(VT-VTI)/VTI
      DELP=SI+((BN+BO)*DX*GAW/4000.)
  250 PRINT 260,I,DXX,DEL,DELP
  260 FORMAT(I3,F7.1,3X,2E15.3)
      DELH=DELH+DX*DEL
      DXX=DXX+DX
  200 CONTINUE
      PRINT 270,DELH
  270 FORMAT(10H DELH=    ,E10.3,8H  FEET  )
      GO TO 310
  300 NNP=NNP-1
      NEL=NNP-1
```

```
      PRINT 320,NNP
  320 FORMAT(30H NNP BELOW WATER TABLE,NNP=     ,I5)
      IF(NNP.EQ.1)GO TO 330
      GO TO 145
  310 CONTINUE
      IF(NRATE.EQ.0)GO TO 311
C      COMPUTE RATE OF HEAVE
      AKT=0.0
      BKT=0.0
      DO 312 I=NBX,NEL
      MTYP=IE(I,1)
      AKT=AKT+DX/(CVS(MTYP)*AMV(MTYP))
      BKT=BKT+AMV(MTYP)*DX
  312 CONTINUE
      AB=AKT*BKT
      T2=0.031*AB
      T4=0.126*AB
      T6=0.287*AB
      T8=0.567*AB
      T9=0.848*AB
      H2=0.2*DELH
      H4=0.4*DELH
      H6=0.6*DELH
      H8=0.8*DELH
      H9=0.9*DELH
      PRINT 314
  314 FORMAT(20H  TIME,DAYS  DELH,FT )
      PRINT 316,T2,H2,T4,H4,T6,H6,T8,H8,T9,H9
  316 FORMAT(/,2E10.3,/,2E10.3,/,2E10.3,/,2E10.3,/,2E10.3,/)
  311 NP=NP+1
      IF(NP.GT.NPRJBY)GO TO 330
      GO TO 400
  330 STOP
      END
```

APPENDIX D: NOTATION

A,B	Suction parameters
$\underline{c}_{vs}$	Average coefficient of swell for the soil in the active zone, sq ft/day $c_{vs}(i)$, (i =
$c_{vs}(i)$, (i = 1,2...NMAT)	Coefficient of swell of soil layer i , sq ft/day
C	Soil cohesion, lb/sq ft
C_c	Compression index
dx	Increment of depth, ft
D	Shaft diameter, ft
$e_f(i)$	Final void ratio of soil increment i
$e_o(i)$	Initial in situ void ratio of soil increment i
e_s	Void ratio at 0.1 ton/sq ft pressure following rebound from swell pressure S_p
$e_{fs}(i)$	Maximum in situ void ratio of soil increment i
$e_{fu}(i)$	Final void ratio in partially saturated soil of increment i
e_{po}	Void ratio at soil overburden pressure P_o following rebound from swell
$G_s(i)$	Specific gravity of soil increment i
h	Total heave, ft
H	One-half of the thickness of the specimen for sorption from both top and bottom of the specimen, in.
i	Soil increment
$\underline{k}$	Average coefficient of permeability, ft/day
k(i), (i = 1,2...NMAT)	Coefficient of permeability of soil layer i , ft/day
K	Ratio of intergranular pressures on the horizontal and vertical planes
LL	Liquid limit
$\underline{m}_{vs}$	Average coefficient of volume change from swell, sq ft/lb
$m_{vs}(i)$, (i = 1,2...NMAT)	Coefficient of volume change from swell of soil layer i , sq ft/lb

NBX	Number of soil increment at the footing of the foundation
NEL	Number of soil increments
NMAT	Number of soil layers or materials
NPROB	Number of loading cases
p	Pressure of water vapor, lb/sq ft
p_o	Pressure of saturated water vapor, lb/sq ft
p/p_o	Relative humidity
P	Vertical load applied at the top of the pier, lb; also, vapor pressure of the pore-water extract
P_o	Original total overburden pressure, lb/sq ft; also, vapor pressure of free pure water
$P_{fo}(i)$	Final soil overburden pressure of soil increment i, lb/sq ft
$P_{fo}(NBX)$	Overburden pressure of surrounding soils at footing of foundation, lb/sq ft
PI	Plasticity index
$\bar{P}_f$	Final effective pressure, lb/sq ft
$\bar{P}_{fs}(i)$	Final effective pressure of saturated soil increment i, lb/sq ft
$\bar{P}_{fu}(i)$	Final effective pressure of partially saturated soil increment i, lb/sq ft
Q	Total structure pressure, lb/sq ft
QQ	Parameter
R	Universal gas constant, 82.06 cc-atm/Kelvins-mole
S	Swell for 1-psi surcharge, percent
S_p	Swell pressure, ton/sq ft
t	Time, days
t_{90}	Time to complete 90 percent of the primary swell, minutes
T	Time factor for various percentages of ultimate swell; also, absolute temperature
$\underline{T}$	Tension force, lb; also, uplift force
T_{90}	Time factor to complete 90 percent of the primary swell, 0.848

v_w	Volume of a mole of liquid water, 18.02 cc/mole
$V_T(i)$	Specific total volume of soil increment i following swell
$V_{TI}(i)$	Initial specific total volume
$V_{TP}(i)$	Maximum specific total volume
w	Water content, percent
w_o	Natural water content, percent
$w_o(i)$	Initial water content of soil increment i , percent
$w_{fs}(i)$	$e_{fs}(i)/G_s(i)$, maximum water content of soil increment i , percent
$w_{fu}(i)$	Final water content of soil increment i , percent
x	Depth below ground surface, ft
x_a	Depth of active zone for sorption of moisture, ft
x_a	Depth of active zone, ft
$x(i)$, $(i = 1,2...NMAT)$	Vertical dimension of soil layer i , ft
x_{wt}	Depth to the water table, ft
α	Compressibility factor
β	Function of the particle contact area
γ	Unit weight of soil, lb/cu ft
γ_w	Unit weight of water, lb/cu ft
ΔP_b	Increase in pressure at base of footing, lb/sq ft
$\Delta P_{st}(i)$	Increase in pressure at soil increment i due to the structure, lb/sq ft
τ^o	Total suction free of external pressure except atmospheric pressure, atm
τ_m^o	Matrix suction head free of external pressure, ft
τ_m	In situ matrix suction head, ft
τ_s	Osmotic suction, atm
τ_{ma}	In situ matrix suction head at bottom of active zone, ft
$\tau_m(i)$	In situ matrix section at soil increment i , lb/sq ft

$\tau_m(x)$	In situ matrix suction head at depth x , ft
T	Absolute temperature, Kelvins
ϕ	Angle of internal friction, deg