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METHODOLOGY FOR FOUNDATIONS ON EXPANSIVE CLAYS[a]

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INTRODUCTION

The purpose of this paper is to survey recent developments concerning shallow foundations on expansive clays that can be applied to practical situations. Its intent is to develop an increased awareness of rational concepts in the analysis and design of shallow foundations for expansive clay sites.

BASIC METHODOLOGY FOR FOUNDATION DESIGN

Prior to arriving at a final foundation design on an expansive clay site, at least four distinct steps must be taken by the designer.

1. Identification—Determine whether potentially expansive clays exist.
2. Classification—If expansive clays do exist, what degree of attention needs to be paid to them?
3. Quantification—If clays are sufficiently prone to volume change, a rational assessment of numerical values of potential or probable vertical movement should be made.
4. Evaluation of Design Alternatives—Compare alternative foundation designs

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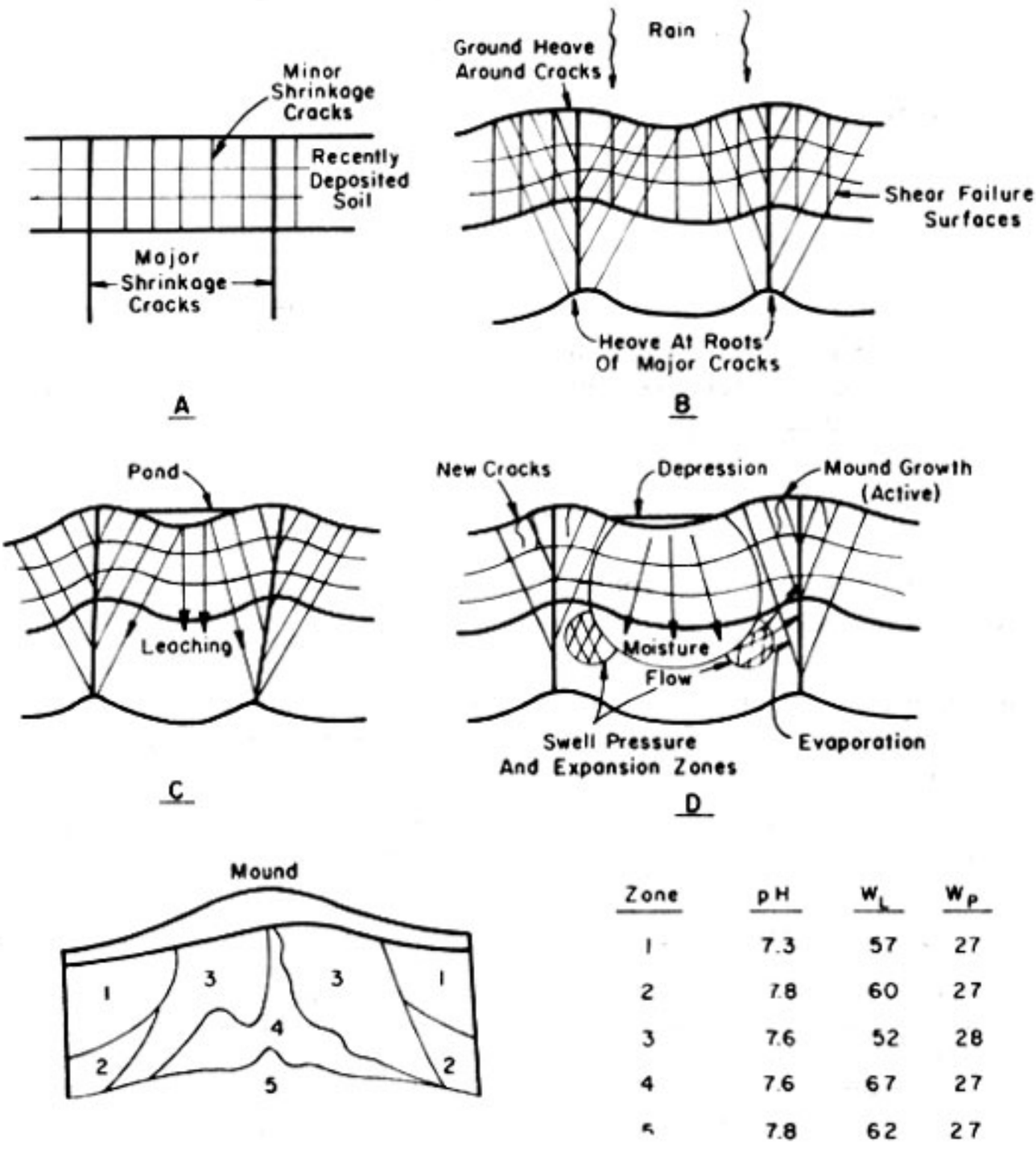
to determine the most suitable and economical design compatible with the predicted vertical movement. Three basic approaches can be followed: (a) Alter the condition of the expansive clay; (b) bypass the expansive clay by isolating the foundation from its effect; and (c) provide a shallow foundation capable of withstanding differential movements and mitigating their effect in the superstructure.

These four steps are examined in more detail.

Identification of Expansive Clays

Visual.—Site visits or analyses of aerial photographs, or both, are useful to assess the possibility of expansive clays being present on a given site. Some



Zone	pH	w_L	w_P
1	7.3	57	27
2	7.8	60	27
3	7.6	52	28
4	7.6	67	27
5	7.8	62	27

FIG. 1.—Stages in Development of Gilgai and Characterization of Samples from Texas Gilgai (16)

of the indicators that can help to identify naturally occurring potentially expansive clays are (13,16) covered in the following paragraphs.

Soil Clod Characteristics.—Very hard when dry; glazed when cut by scraper or shovel; cracks in regular pattern. Soft and sticky when wet and leaves a powdery residue after molding with hands.

Terrain Characteristics.—Evidence of creep on slopes; wide, deep shrinkage cracks with fairly regular spacing; gilgai structure.

Gilgai structure (10,15) is a mound-depression feature visible on the surface of expansive clays that have undergone weathering in a semiarid environment. The development of gilgai structure and variations of soil properties therein are presented in Fig. 1. Often a regular gilgai spacing from 6 m–21 m is observable, and depths can reach 2 m. Gilgai mounds tend to be more expansive than depressions due to higher mass permeability, more dispersed microstructure, and lower pH. The existence of gilgai structure complicates the design of shallow foundations because of pointwise variation of swell potential across a site. Leveling of gilgai fields is usually ineffective in preventing differential soil movement.

Existing Vegetation.—Vegetation on a site can be indicative of the presence of expansive clays. Local experience usually reveals that certain trees or shrubs tend to occur more frequently on soils having either high or low expansion potential. For example, in central Texas, mesquite cover can usually be associated with expansive clays, where oak trees are associated with soils of relatively low potential expansion.

Shallow root systems continuously remove moisture from near-surface soil and desiccate expansive clays. When vegetation is removed and building floor slabs are placed on grade, the clays begin to regain moisture and swell more than the clays at locations where such vegetation was absent. This effect can produce structural distress even in soils that are relatively inert.

Climate.—The extent to which potentially expansive clays undergo volume change is greatly influenced by local climate. Locations prone to long dry periods followed by periods of wet weather are most susceptible to expansive clay activity. In areas where shallow water tables do not exist, moisture conditions in the clays are controlled by moisture balance between rainfall and evaporation. One rational means of classifying climate is the Thornthwaite Moisture Index (TMI) (32) which is defined as the difference in mean annual rainfall in inches and the amount of water in inches that would be returned to the atmosphere by evaporation from the ground surface and transpired by plants if there were an unlimited supply of water to the plants and soil. A positive TMI indicates a net soil moisture deficit. Locations having TMI values between about +20 and −20 are most prone to experiencing difficulty with expansive clays. The TMI provides a basis for correlating certain quantitative parameters that are described later. TMI values for representative locations are: Denver, −10; Dallas, 0; and Houston, +18.

Laboratory Tests.—Simple laboratory tests, including liquid limit, plastic limit, shrinkage limit, and hydrometer tests can be used to identify potentially expansive clays. Natural soils having liquid limits in excess of 40, and plasticity indices in excess of 15 can be considered potentially expansive. Other, more elaborate tests, such as X-ray diffraction and cation exchange capacity tests, are useful in determining the predominant clay mineral present. Some procedures for quantifying swell, described later, require knowledge of the predominant clay mineral.

Local Experience.—Despite the best attempts of identification mentioned in the foregoing sections, some expansive soil sites may go unidentified. Experience with behavior of structures in a given geological region may be useful in identifying potential problems. For example, water for irrigation of lawns may have different pH and solutes from those of rainfall. Such waters can be the source of free ions that stimulate a base exchange in the soil, amplifying its expansive characteristics (13).

Cut and fill operations in expansive clays may create foundation problems. Expansive clays become more prone to volume change when remolded and compacted, largely because of the breakup of cementation and possible production of high negative pore water pressure that may later be relieved. Thus, the same clay may vary in its expansion characteristics from one side of a site to another, depending on whether the clay is in its natural state or has been compacted (13). It is therefore desirable to conduct site investigations for the purpose of classifying and quantifying expansive soil behavior after rough site grading has been accomplished.

Classification of Expansive Clays

Classification is an attempt to ascertain "how bad" potentially expansive clays are. Proper classification should involve the use of index properties coupled with in-situ properties such as dry density, water content, and soil suction.

Suction is a parameter that characterizes the pulling pressure (tension) exerted by soil on free water which comes into contact with it. It represents a potential for water uptake by the soil and is extremely useful for calculating rates of water movement through partially saturated soils. Suction is the sum of two components: matrix and osmotic. The matrix component is that pressure produced when the free water has the same chemical concentrations as the soil's pore fluid. Matrix suction is both water content and surcharge pressure dependent. Osmotic suction arises from differences in concentrations of soluble salts in the pore water and free water and is independent of water content and surcharge pressure. Matrix suction is the most important component with respect to volumetric behavior of expansive clays.

A relationship exists between initial suction, external pressure, and volume change upon allowing a given soil free access to water. A recent study (10,28) has shown that a numerical relationship does not appear to be universal for all soils but that general correlations may exist within soils within a geological and climatic zone. One useful result of that study was the development of a parametric relationship between suction and initial water content, which can be used to classify severity of potentially expansive clays:

$$\log \tau_m = A - Bw \ (\tau_m > 1 \text{ ton/sq ft}) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (1)$$

in which τ_m = matrix suction without surcharge pressure, in tons per square feet (1 ton/sq ft = 95.8 kN/m^2); A, B = constants; and w = water content, as a percentage.

Statistically, swell measured in the laboratory using the swell overburden test (considered later) can be correlated to parameters A and B. Typical A and B values for a given physiographic region (10,28) are:

Physiographic Region	A	B
Moist subhumid ($0 < TMI < 20$)	2.75–8.5	0.10–0.34
Semiarid ($-40 < TMI < -20$)	2.5–5.8	0.12–0.25

Closer correlation presumably can be obtained locally within a given formation. In gilgai fields, suction may vary from point to point, being highest in the mound soil.

Suction is a more definitive measure of potential expansion than water content and index properties. Total suction has been successfully measured in the laboratory and under favorable conditions in the field using psychrometers (5,9,10), although in-situ psychrometers tend to be corroded in soils with other than neutral pH. An indirect method has also been developed whereby soil clods are wrapped in a dry filter paper at 20° C (10,22). After several hours (to several days), the filter paper is removed, its moisture content obtained, and suction determined from a moisture content-suction calibration curve for the filter paper. For standard S and S No. 589 white ribbon filter paper:

$$\text{Suction in } pF = 6.246 - 0.0723\, w_p \qquad (w_p < 54\%) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which w_p = moisture content of the filter paper as a percentage; and pF = log of suction pressure in centimeters of water.

TABLE 1.—USAEWES Classification System (29)

w_L, as a percentage (1)	I_p, as a percentage (2)	$\tau_{initial}$, in tons per square feet* (3)	Potential swell, as a percentage (4)	Potential swell classification (5)
<50	<25	<1.5	<0.5	low
50–60	25–35	1.5–4.0	0.5–1.5	marginal
>60	>35	>4.0	>1.5	high

*$\tau_{final} = 0$.
[b]Vertical swell of a confined sample with vertical pressure equal to overburden pressure. Expressed as a percentage of sample height.
Note: 1 ton/sq ft = 95.8 kN/m^2.

It should be understood that suction (and thereby swell) is influenced by interparticle soil attraction (electrostatic or cementation bonds) and that sampling tends to affect those bonds. Thus, suction measurements (or direct swell measurements made on samples) in the laboratory may not correlate closely to in-situ suction unless sampling is conducted with care.

United States Army Engineer Waterways Experiment Station (USAEWES) (29) recommends that classification of potential swell can most reliably be done using liquid limit (w_L), plasticity index (I_p), and initial (in-situ) suction ($\tau_{initial}$). Their proposed classification system is presented in Table 1.

This is one of numerous classification systems available to the practicing engineer. The writers believe it to be reliable. It should be applied as follows: (1) For "low" swell potential, volume change will be minimal, so use normal construction procedures; (2) for "marginal" swell potential, consider past

experience with the soil (if unavailable, treat as "high" swell potential); and (3) for "high" swell potential, quantify the swell, using procedures described in the following section and proceed with a rational foundation design.

Quantification of Swell

Quantitative values for swell can be obtained by two approaches described in the following. Total swell values are useful in choosing appropriate foundation systems. Differential swell must be estimated either directly or indirectly in order to make rational designs of slabs on grade.

Laboratory Measurement of Total Swell from Oedometer Tests (12).—Essentially undisturbed samples are taken from representative locations and depths at a

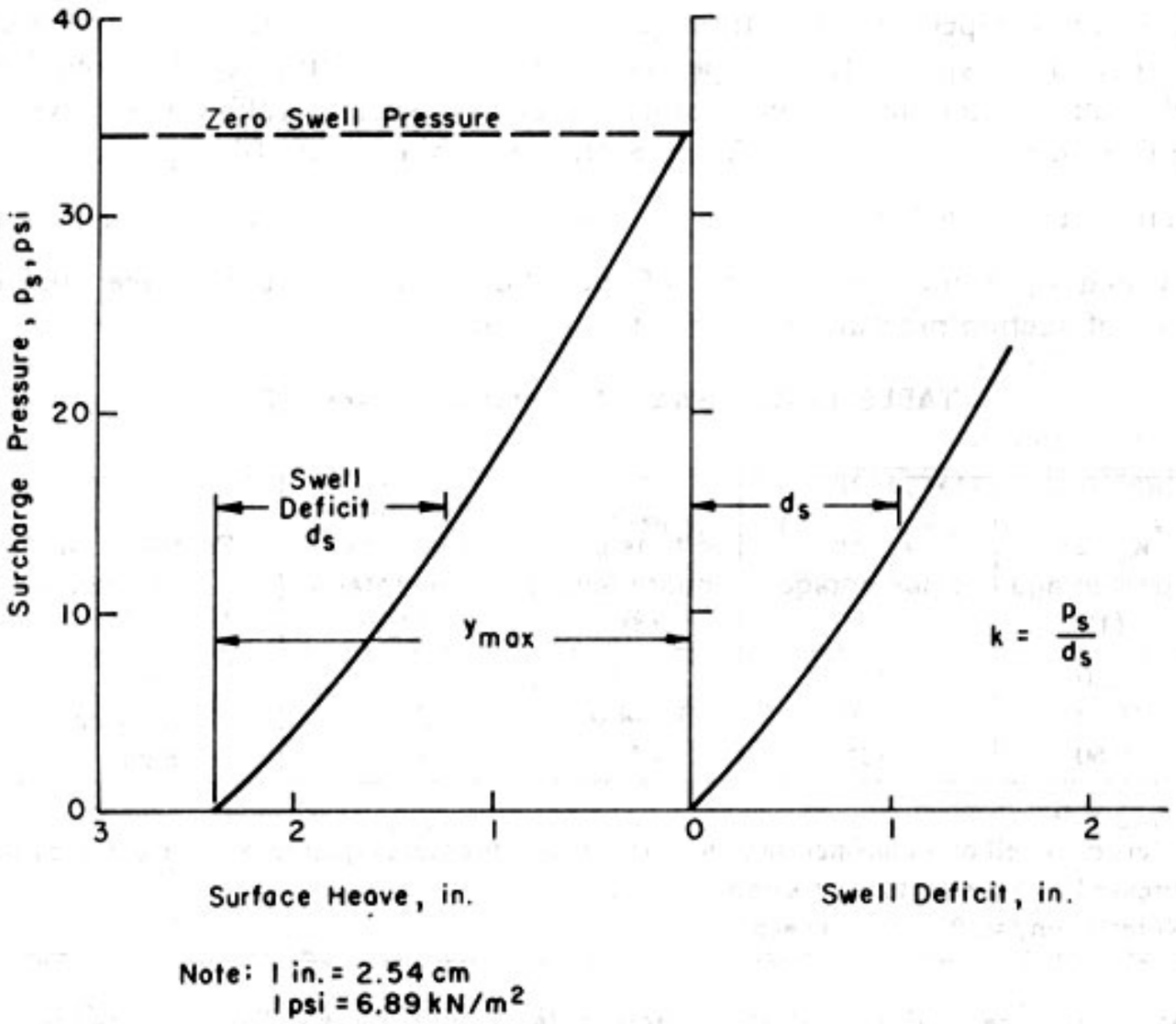


FIG. 2.—Pressure Versus Swell Deficit (17)

site after rough grading has taken place and during the season of foundation construction. Each sample is then placed in an oedometer, and pressure equal to the total overburden plus anticipated surcharge pressure, is applied. Free water is then added to the sample, and sufficient additional load is applied in small increments to prevent swelling until the swell pressure is fully developed. The specimen is then unloaded back to the overburden pressure in decrements. Each decrement is held until primary swell is complete as verified by examination of the time versus swell plot. Results of this test, termed a "constant volume swell test," can then be applied to compute surface heave as a function of ground surcharge:

$$y = \sum_{i=1}^{N} s(\%)\, h_i(0.01) \quad \ldots \quad (3)$$

in which $s(\%)$ = swell as a percentage in soil sublayer i under an applied pressure equal to the ground surcharge pressure (representative of structure load) plus total overburden pressure; h_i = thickness of sublayer; y = ground heave under the given ground surcharge pressure; and N = total number of sublayers down to the zone of constant suction.

By obtaining s values for various values of ground surcharge pressure in each sublayer and successively applying Eq. 3, a plot of ground surcharge pressure versus ground heave can be developed for a given location as shown in Fig. 2. Assuming one-dimensional soil movement, with an adequate number of tests, a profile of potential swell for any given ground surcharge pressure can be developed for a site. A coefficient of long-term subgrade reaction which relates ground pressure to swell can also be obtained for a later rational foundation analysis as shown in Fig. 2. Free field (without structure present) differential swell can be estimated by repeating the procedure at several points on a site. A reasonable estimate of differential swell in terms of percent of maximum total swell can be obtained, but care should be exercised in estimating numerical values for differential swell from such on approach because the total swell is dependent on the moisture conditions at time of sampling. Potential or probable total swell values for typical site conditions in a locality should be obtained over a wide range of seasonal moisture conditions. They should be cataloged where possible so that total swell measurements for a specific site can be compared with expected reasonable upper limits for the area.

Other laboratory swell tests are possible. The exact amount of swell obtained is dependent on test procedure. Depth to constant suction (or depth to which swelling occurs) can be measured directly in a given locale where the water table is deep (≥5 m) by taking numerous suction measurements over several wet and dry seasons. Where a high water table exists, it can conveniently be taken as the depth to the water table. Another simple method is to plot liquidity index versus depth, where soil samples have been obtained in various seasons of the year.

Typical liquidity indexes versus depth data are shown for the Beaumont (CH), and Montogomery (CL) formations in the Houston, Tex., area in Fig. 3. It is apparent that the formation characteristics influence this depth to some degree. Typical depths of active zones for some of the American cities are:

City	Depth of Active Zone, in feet
Houston	5 ft–10 ft (1.5 m–3 m)
Dallas	7 ft–15 ft (2.1 m–4.6 m)
San Antonio	10 ft–30 ft (3 m–9 m)
Denver	10 ft–15 ft (3 m–4.6 m)

In some moisture-deficient clays and clay shales of the southern and western United States, evidence exists that swell can occur to depths of at least 30 ft–50 ft (9 m–15 m) when the moisture content of the soil is increased by placing a membrane (e.g., a slab) on the site, and thereby impeding evaporation of pore water from the site (8,10).

The prediction of heave could be made by use of the Jennings (7) double oedometer test. In this procedure, two consolidation tests are performed on identical samples. One sample is allowed to swell under loads with access to water. The other sample is sealed at natural water content in a plastic membrane and incrementally loaded simultaneously with the previously saturated sample. The e-log p curves are plotted and the compression lines at high pressure determined from these samples are adjusted to coincide. The amount of swell or change in void ratio for a given load can be predicted at any pressure. Although the predictions of heave by this test have correlated fairly well with field measurements, this test seems to generally overestimate heave.

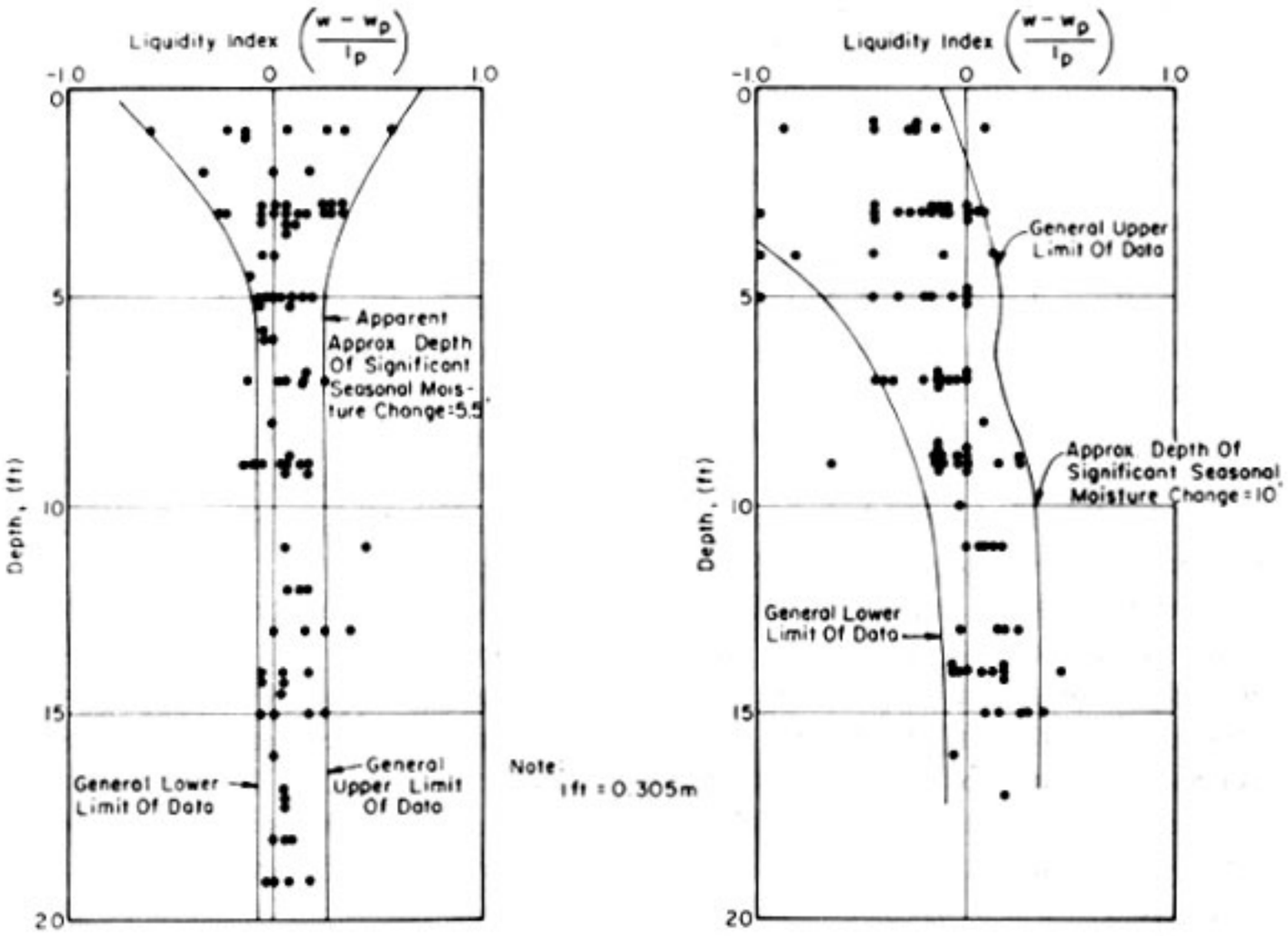


FIG. 3.—Liquidity Index Versus Depth for Samples taken over Several Seasons in Houston, Texas Area (Data Courtesy Southwestern Laboratories): (*a*) Beaumont Formation; (*b*) Montgomery Formation

Published Methods.—Numerous methods exist whereby the total potential swell that can occur on a site can be calculated from simple parameters. Most of these methods are correlative and they relate simple soil properties empirically to the vertical heave. They indirectly and crudely include the effect of soil suction through the use of initial water content as a parameter. A few of the more prominent methods are examined in the following.

McDowell Method (19).—This method yields a simple pressure-volume change relationship for swelling clays based on water content, index properties, vertical pressures (surcharge plus soil), and depth of active zone. Correlations were developed mostly for central Texas soils. McDowell proposed a reasonable minimum initial moisture content of 0.2 w_L + 9% for highway subgrades.

Presumably, this value can also be applied to sites on which shallow foundations are installed.

The first writer and Ghazzaly (23) attempted to correlate swell potential computed from this method to observed distress in structures with shallow

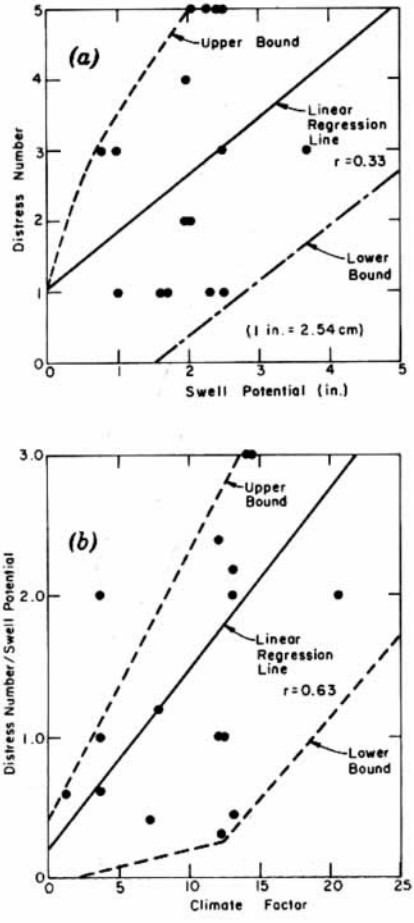


FIG. 4.—Correlation of Building Distress to Swell Potential by McDowell Method (23): (*a*) Distress Versus Swell Potential; (*b*) Normalized Distress Versus Climate Factor

foundations at a large, closed site in Houston, Tex. Correlation curves obtained are depicted in Fig. 4(*a*), in which degree of distress has been quantified by a distress number. Although a general trend to higher distress was found with

increasing swell potential, the correlation was poor. The addition of a climate variation factor (CF), defined as the absolute value of rainfall in inches in the year of foundation construction minus mean annual rainfall for the site, improved the correlation when distress was considered in normalized form, Fig. 4(*b*). However, distress clearly could not be predicted with accuracy using this method.

Vijayvergiya-Ghazzaly Method (35).—This method yields percent swell of a

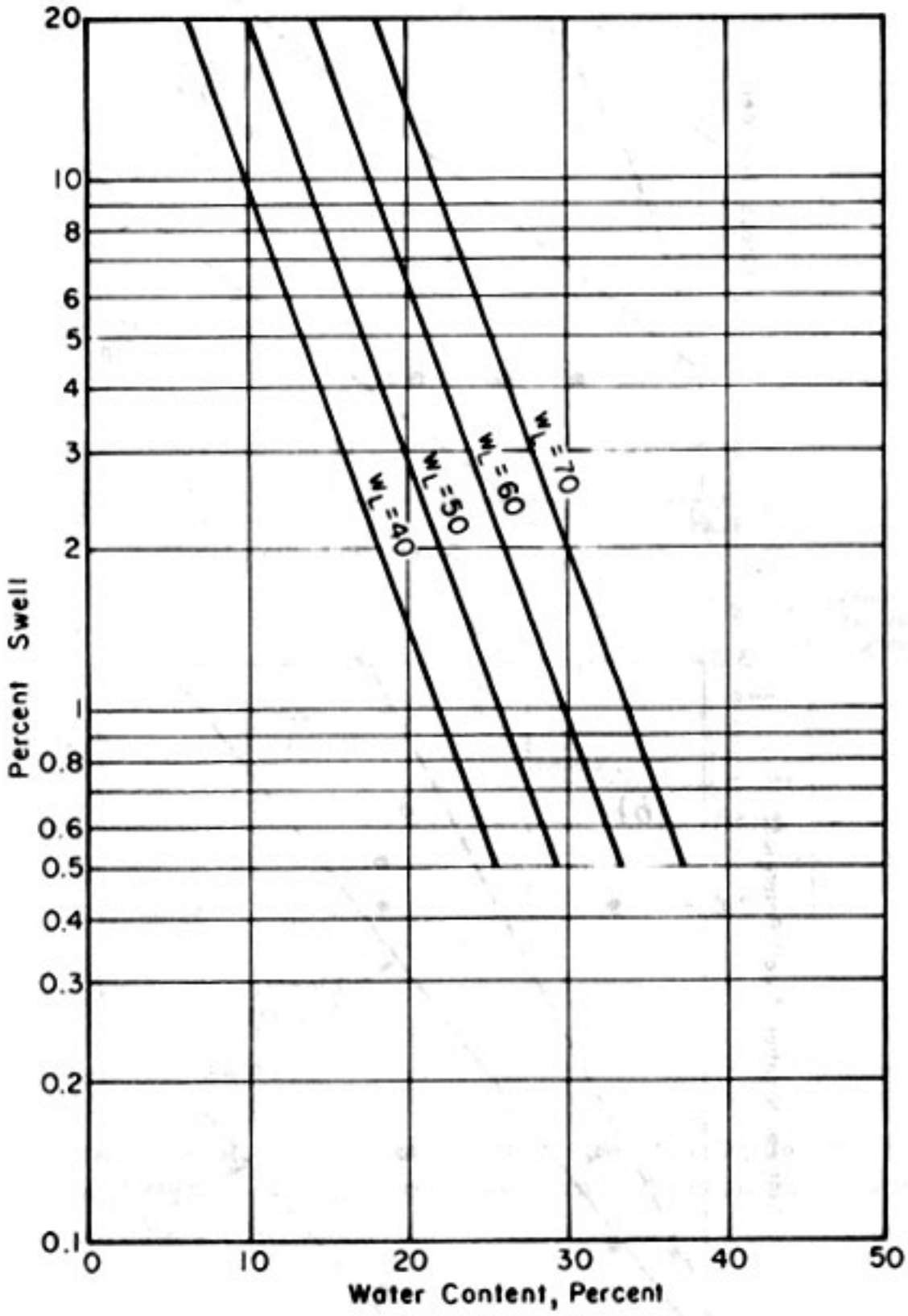
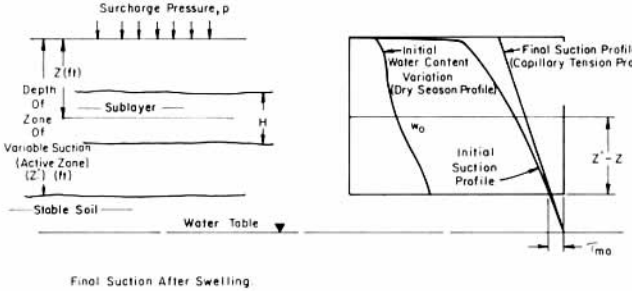


FIG. 5.—Correlation of Percent Swell with Liquid Limit and Initial Water Content (35)

clay sample under a 0.1 ton/sq ft (9.6 kN/m^2) surcharge in a consolidometer as a function of moisture content and liquid limit as shown in Fig. 5. The correlation covers a wide data base. In order to obtain surface free swell potential, y, using this method, Eq. 4 is used:

$$y = 0.0033\, z'\, s \quad \ldots\ldots\ldots\

Seed-Woodward-Lundgren Method (26).—Unlike the foregoing two methods, which pertain to natural soils, this method gives free swell as a percentage, which is the percent swell of a clay sample under 0.1 ton/sq ft (9.6 kN/m^2) surcharge in a consolidometer in a compacted clay near optimum moisture content, as follows: $s = 3.6 \times 10^{-5}\ D^{2.44}\ C^{3.44}$; in which $D\ I_p/(C - 5)$; and C = soil fraction finer than 2 μ, as a percentage.



Final Suction After Swelling

$$\tau_{mf} = \tau_{mo} + (Z' - Z)\gamma_w \qquad \text{(Valid For Shallow Water Table, } \leq 5\text{m Below Site Grade)}$$

Where τ_{mo} = Measured Stable Suction Value At Base Of Zone Of Variable Suction

(If Soil In Active Zone Can Become Saturated, e.g., Due To Local Ponding, τ_{mf} Should Conservatively Be Taken As Zero)

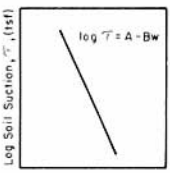
$$\Delta s \text{ (In Sublayer)} = H \frac{C_\tau}{1+e_o}\left[\log\left[\tau_m^o/(\tau_{mf} + \alpha(p+\gamma Z))\right]\right]$$

$$= H \frac{C_\tau}{1+e_o}\left[(A - Bw_o) - \log\left[\tau_{mf} + \alpha(p+\gamma Z)\right]\right]$$

$$\text{Swell} = \sum \Delta s \quad \text{(For All Sublayers)}$$

A = Intercept Of Suction – Water Content Curve

B = Slope Of Suction – Water Content Curve



τ_m^o = Initial Matrix Suction Without Surcharge

C_τ = Suction Index = $\alpha G_s / 100\,B$

G_s = Specific Gravity Of Soil Solids

e_o = Initial Void Ratio

w_o = Initial Dry Season Moisture Content

p = Surcharge Pressure (tsf)

γ = Total Unit Weight Of Soil (tcf)

α = Soil Compressibility Factor = 0 For $I_p < 5$

= $0.0275\,I_p - 0.125$ For $5 \leq I_p \leq 40$

= 1 For $I_p > 40$

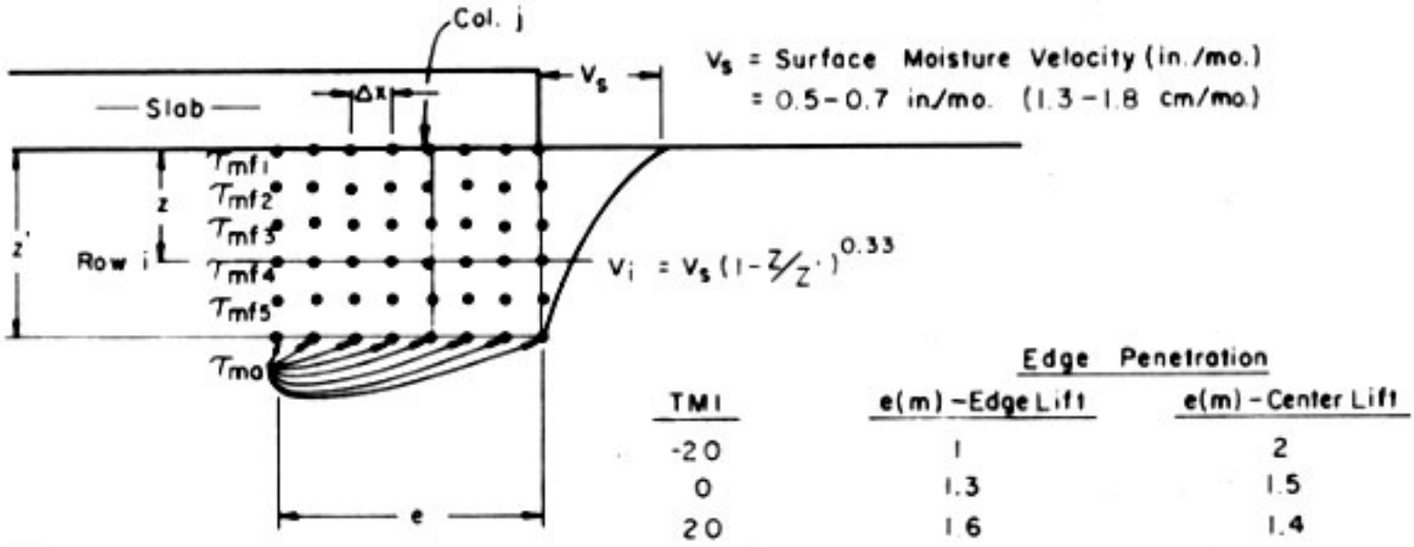
Note: 1 tsf = 95.8 kN/m^2, 1 tcf = 314 kN/m^3

FIG. 6.—Synthesis of Vertical Swell from Soil Suction (27)

Suction Change Method.—This method employes the change in suction from initial to final conditions to compute swell (10,17). A summary of this method is presented on Fig. 6 for a high water table condition (stable suction = 0 at depth of water table). The initial and final suctions for each representative sublayer must be known. For a shallow water table (≤5 m) the final equilibrium suction profile that might occur after a site has been covered for a long time can reasonably be assumed to be as shown in Fig. 6. For areas having deep

water table conditions, equilibrium suction should be measured in-situ over several seasons to establish reasonable values. In humid to semiarid climates, equilibrium $pF \approx 3.5$ in clays. McKeen (20) suggests that the cracking pattern strongly influences swell in-situ. Calculations made using Fig. 6, which assumes no cracks in the soil, may thus be expected to yield good results for a close-jointed soil but may yield heave values that are too high (by a factor of up to 3) for soils having frequent, wide, and deep cracks.

PTI *Method (6,24,35).*—Lytton and Wray (with data from McKeen) have developed a procedure to obtain total probable swell (as opposed to potential swell), based on the soil suction method and correlation of moisture penetration



V_s = Surface Moisture Velocity (in./mo.)
= 0.5–0.7 in/mo. (1.3–1.8 cm/mo.)

TMI	Edge Penetration e(m) – Edge Lift	e(m) – Center Lift
-20	1	2
0	1.3	1.5
20	1.6	1.4

τ_{mfi} = Final Suction In Row i In Stable Zone ≥e From Slab Edge (See FIG. 6)
τ_{mo} = Stable Suction (See FIG. 6)
τ_{ij}° = Initial Suction At i,j
τ_{ij} = Temporary Suction At ij Under Soil Moisture Velocity V_i

SOLVE ITERATIVELY (From Left To Right On Each Row):

$$\tau_{ij} = \tau_{i,j-1} + \frac{V_i \Delta x}{k_{ij}}, \text{ Where}$$

$k_{ij} = k_o / (1 + b|\tau_{ij}|^n)$: $k_o = 2.7 \times 10^{-6}$ cm/sec., $b = 10^{-9}$, $n = 3$, (τ_{ij} is In cm Of Water)

$$\Delta s \text{ (\% Swell At Depth } z) = 100R \left[\log \tau_{ij}^{\circ} - \log \tau_{ij} \left(\frac{P + \gamma z}{\sigma_o} \right)^r \right]$$

$R \approx 0.00056$ (% Montmorillonite) + 0.00047 (% illite) + 0.00018 (% koolinite) / $1 + e_o$

$r = R/C_c$ Where C_c = Compression Index Of Overconsolidated Clay From Oedometer Test

σ_o = Vertical Geostatic Stress At Depth Of 1 ft (.305 m).

FIG. 7.—Probable Swell Calculations (6,25)

and moisture velocity with TMI. This method also yields the probable profile of the soil surface near the edge of a slab on grade, where seasonal moisture changes can occur. The method, a simple iterative numerical technique, is depicted in Fig. 7, using definitions from Fig. 6. The equation shown in the box is solved iteratively to yield suction values at each point in the array. Then the percent swell at each elevation is computed from the equation near the bottom of the figure. This equation differs from the Δ_s equation in Fig. 6 in that it relates the suction index to percentages of various clay minerals rather than to the parameters B and α. It is evident that the l_i values differ for different values of i, so that a swollen (or shrunken) surface profile is obtained. This profile forms a basis for the PTI method for structural design referred to later.

The moisture velocity at the surface, V_s, can be obtained from measurement of monthly moisture change in a column of soil. This velocity should be doubled when the root zones of large trees reach the edge of the slab. If data are unavailable, the value indicated in Fig. 7 may be used. In the recommendations made by Wray (36), the various clay mineral components can be obtained from United States Department of Agriculture (USDA) maps, Soil Conservation Service (SCS), soil surveys or by an experimental procedure using measured clay activity and cation exchange capacity.

Carothers (3) suggests, based on studies of Beaumont clay, that

$$C_c \approx 0.0009\ [w_L(\%) - 10] \quad \ldots\ldots\ldots\

thus arrive at a more economical foundation by altering the clay suction. Several means of achieving this are examined in the following, although it is difficult to assess numerically in general the effects of these procedures without making direct measurements on the altered clays.

Lime Treatment.—It has been demonstrated that thorough mixing of hydrated lime with surficial clay can reduce swell in the zone most prone to swell (zone with least vertical pressure). Addition of lime to soil supplies an excess of multivalent calcium cations which tend to replace such monovalent cations such as sodium and potassium and improve shrinkage, swell, and workability characteristics. Lime-soil possolanic reaction takes place between the lime and the silicates or alumina constituents of clay, or both, to form cementing agents which are primarily hydrated calcium silicates or calcium aluminates, or both. These cementing reactions provide a major increase in strength which occurs in soil-lime mixtures that is useful in retarding swell forces (8,10). Intimate

TABLE 2.—General Relationship of Foundation Type to Total Computed Swell (Modified after Ref. 7)

y, in inches		
Length-to-Height Ratio of Wall Panel		
1.25 (1)	2.5 (2)	Preferred foundation construction (3)
0.0–0.25	0.0–0.5	No special precautions
0.25–0.5	0.5–2.0	Stiffened mat; strip footings with high bearing pressure
0.5–2.0	2.0–4.0	Stiffened mat; cellular mat (several independent sections); drilled piers with suspended grade beams but floor slabs supported on fill with flexible joints; three-point support where feasible
2.0	4.0	Drilled piers with suspended floors

Note: 1 in. = 2.54 cm.

mixing of soil with lime can be carried out practically only to a depth of about 1 m, but it is usually confined to the top 15 cm. Normally 5%–7% lime by weight provides adequate alteration. Occasionally, 10%–15% may be required in soils that have been subjected to ammonium fertilizers, since ammonium is a cation that is difficult to replace.

Pressure injected lime (PIL) may be an effective swell control and rehabilitation procedure under certain circumstances. The conditions most favoring the achievement of successful PIL treatment of expansive clays are the presence of an extensive fissure network into which the lime slurry can be successfully injected (31), and a site large enough to allow injection on a grid several points wide in each direction (24). Lime slurry pressure injection when done properly (e.g., multiple injections at different times) can be relatively costly (23). The major effect of PIL is that of prewetting, similar to ponding, described in the following, and not lime reaction.

Ponding.—Ponding (covering a construction site with water prior to construction) is an attempt to raise the moisture content (lower the suction) of near-surface

clays before foundations are installed. Ponding has been shown to be effective in mitigating, but not eliminating, distress in highways (8,30). Ponding may not be effective for shallow foundation systems, expecially in arid or semiarid sites (TMI < 10), for two reasons:

1. Water diffuses through soil in various parts of a site at different rates, depending on the variation in the soil's mass permeability. Furthermore, clays tend to swell in two stages: a primary stage (rapid swelling), similar to primary consolidation, and a secondary stage (slow swelling), similar to secondary consolidation. Primary and secondary swell characteristics can be identified from laboratory oedometer swell tests of the type described earlier. Clays with high secondary swell characteristics and variable mass permeability (e.g., gilgai fields) are not good candidates for ponding due to extremely long ponding times required.
2. The clay may become so swollen after ponding that shrinkage of soil around the perimeters of slabs-on-grade may be severe in arid locations, unless a high moisture content is artificially maintained in the clay outside the slab.

In this regard, ponding in conjunction with exterior moisture barriers may prove effective.

Moisture Barriers.—Moisture barriers may be constructed around the perimeters of slabs-on-grade to minimize moisture variation beneath the slab's perimeter. Recent studies (8,10,31) suggest that vertical trenches about 15 cm wide by 1.5 m deep and filled with gravel (capillary barrier), lean concrete, or mixtures of granulated rubber, lime, and fly ash reduce the long term effects of differential movement around a slab's perimeter. In one field study in the Dallas area on residential slabs (23), perimeter movements (in this case, heave) in a residual CH soil were 5 cm–8 cm where the surrounding subgrade was flooded, but movements stabilized in 1 yr–2 yr. Movements on similar foundation without barriers were somewhat higher but were far more variable around the perimeter.

Horizontal barriers appear to be less effective than vertical barriers because they are easily punctured and, in the case of concrete, can act as wicks to draw moisture out of the soil.

Replacement.—Expansive clays can be replaced by less expansive materials when the expansive clay lies close to the surface. Low-expansion clays are preferable to granular soils in this respect because granular soils are conducive to collecting water on the surface of expansive clays that were not excavated, either from the air through hydrogenesis or through surface seeps.

Compaction.—Compaction is used in conjunction with replacement or alone to control expansion and shrinkage of near-surface soils. In general, compactive efforts should be limited in order to limit the suction but should be high enough to maintain adequate bearing pressures that develop in compacted soils. Compacted soil can be placed above moderately expansive clays to serve as a surcharge which will reduce expansion in near-surface soil. Floor slabs, and, in some cases, lightly loaded interior footings can then be placed in the fill. This process is not recommended for sites having total probable heave exceeding 4 cm. Gromko (7) recommends compaction at 2%–5% above optimum moisture content and maintenance of the following maximum compactive efforts in terms of percent of maximum standard Proctor density.

Maximum Drought Period, in weeks	$I_p = 25$	$I_p = 50$	$I_p = 75$
4	98	90	90
6	96	90	80
12	93	82	70
>12	80	70	70

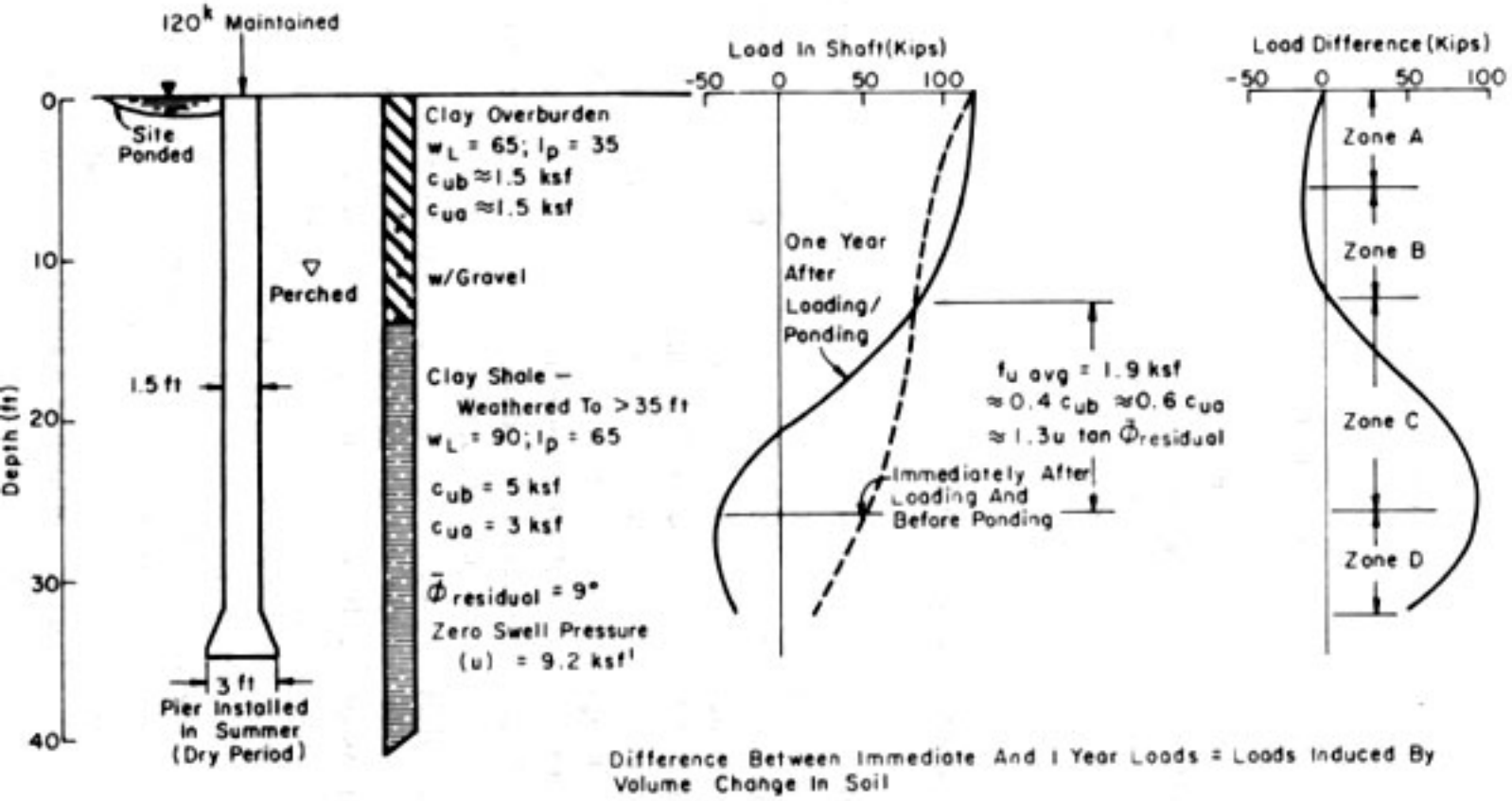
The first three rows correspond very roughly to TMI values of 25, 0, and −20, respectively.

Bypassing of Expansive Clay

As a general rule, when total probable heave exceeds about 8 cm, slab-on-grade foundations are not generally economical. In such a case, expansive clays can be bypassed by using drilled piers to support structural loads. In principle, the drilled pier is installed such that an enlarged base is constructed in stable soil below the active zone. The base, which should have a diameter at least 0.5 m greater than the shaft, acts as an anchor against upward direct shear stresses generated on the shaft due to expansion of clays in the active zone.

Many authorities suggest that the plinth and enlarged base anchor be designed to withstand an uplift load U computed by the following equation (4,14):

$$U = 2\pi r\, u\, z' \tan \bar{\phi}_{ps} \quad \ldots \ldots \ldots



c_{ub} = Undrained Cohesion Before Ponding
c_{uo} = Undrained Cohesion 1 Year After Ponding
$f_{u\,avg}$ = Unit Side Shear In Excess Of That Developed During Immediate Loading
[1] Soil Specimen Loaded To Overburden Pressure Before Adding Water
(1 k = 4.45 kN, 1 ksf = 47.9 kN/m^2, 1 ft = 0.305 m)

Zone A: Reduction In Unit Load Transfer, Possibly Due To Softening Of Soil-Pier Interface
Zone B: Minor Swell Zone
Zone C: Major Swell Zone
Zone D: No Swell

FIG. 8.—Uplift Induced by Volume Change in Expansive Clay on A Belled Drilled Shaft (34)

Johnson (10) recommends, based on empirical correlations, that the following general minimum lengths of drilled piers be considered in expansive clay environments:

$$L = 2Z' - 1.42\left(\frac{D_b}{D_s}\right)^{2.5}; \quad D_s = 1.5 \text{ ft (0.48 m)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$L = 2Z' - 1.76\left(\frac{D_b}{D_s}\right)^{3.0}; \quad D_s = 2.5 \text{ ft (0.76 m)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which D_b = base diameter; D_s = shaft (plinth) diameter; L = pier length; and f_u = shaft adhesion $\leq$ 1 ton/sq ft (95.8 kN/m^2).

When soils are severe enough to require the use of piers, grade beams should be suspended off the soil a distance greater than the total potential heave; and floor slabs should also be suspended off the soil, except where experience dictates that articulation of floor slabs formed on compacted fill having low expansion potential perform satisfactorily.

Provision of Shallow Foundation to Withstand Differential Movements

When predicted probable free-field differential movements do not exceed about 8 cm, it is often economical to found light structures on slabs-on-grade. Numerous design procedures exist for such foundations, most of which are either empirical or are modifications of rational design procedures based on local experience. Three prominent rational procedures are described in the following. The last procedure has been proposed as a minimum standard for the design of post-tensioned slabs but may also be applied to slabs reinforced with deformed bars.

Building Research Advisory Board (BRAB) Procedure (27).—This procedure, although rational, may result in overdesigned foundations because overreinforcement is implicitly required and because partial support of soil near the perimeter is excluded. As a result, the BRAB procedure is no longer accepted as a standard, but it is still an effective guide for the design of stiffened slabs on expansive clays. The basic assumptions of the BRAB method are; (1) The superstructure load is treated as a uniformly distributed load, w, in force per unit area; (2) the soil reaction pressure is uniform over the area of slab assumed to be in contact with the soil (central portion for center lift, perimeter for edge lift), with the slab acting as a cantilever elsewhere; and (3) the area of soil support is a function of a semi-empirical support index, C, which is a function of the weighted average of I_p in the top 5 m of soil (or other measures of swell potential), and a climate rating factor, C_w, which is a function of the maximum drought period in an area. The value of C varies from 1.0 for $I_p \leq 15$, and $C_w = 45$ (maximum drought period of 4 weeks) to 0.6 for $I_p \geq 50$, and $C_w = 15$ (drought period of 12 weeks or longer).

With these assumptions a rectangular slab can be designed for the following:

$$\text{Maximum Moment} = w L^2 L' \frac{(1 - C)}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$\text{Maximum Shear} = w\,L\,L'\frac{(1-C)}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

$$\text{Maximum Differential Deflection} = w\,L^4\,L'\frac{(1-C)}{48\,E\,I} \quad \ldots\ldots\ldots\ldots \quad (11)$$

in which L = slab length; and L' = width; and $E\,I$ = the product of moment of inertia of the slab cross section (cracked section assumed and considering stiffening beams), and elastic modulus. Nonrectangular slabs are designed by analyzing for moments and shears in rectangular subsections. The unsupported

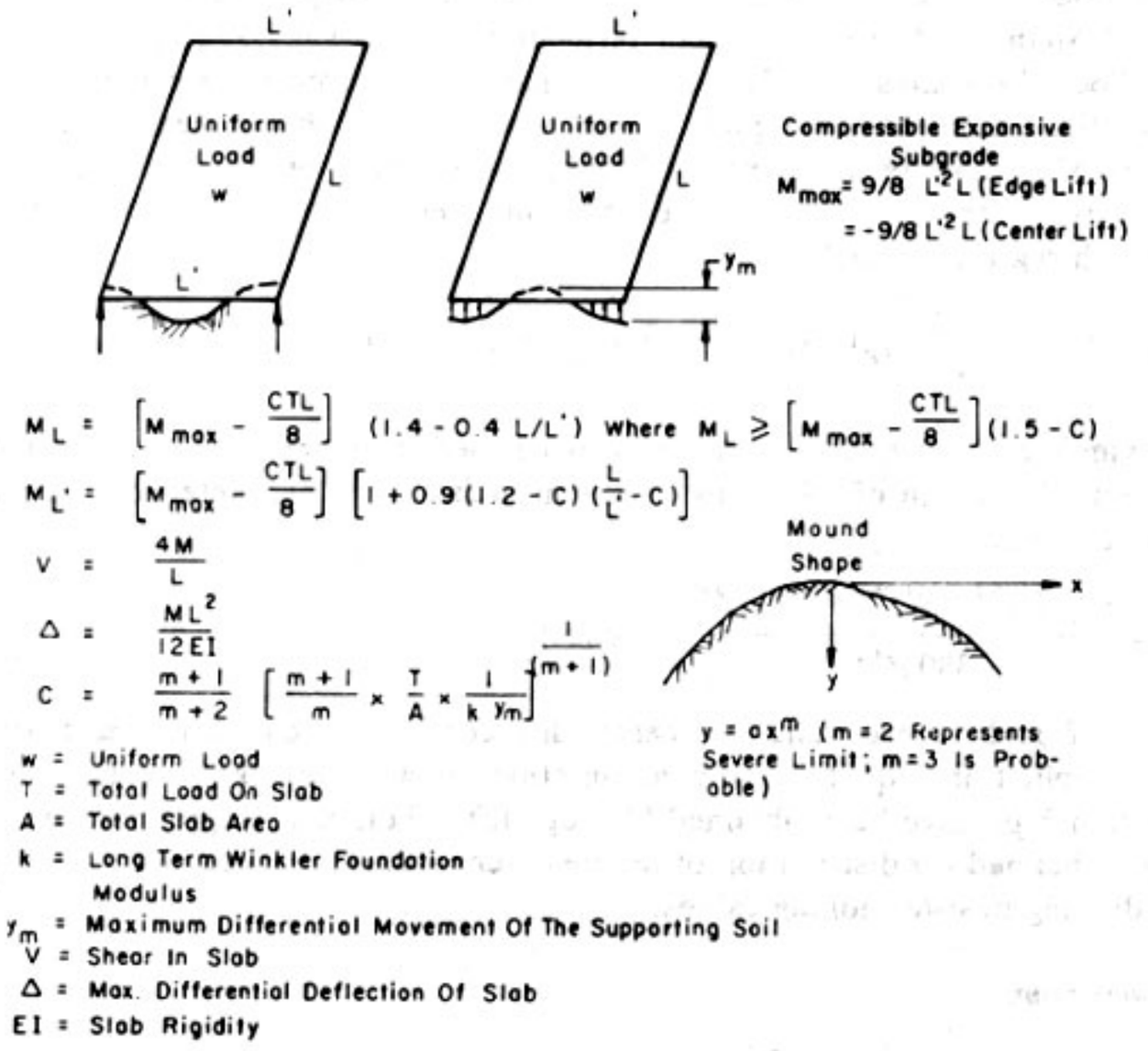


FIG. 9.—Example Results of Subgrade Reaction Procedure (18)

peripheral part of a center lift slab is 0.5 $(1 - C)L$ (or L') and is 0.5 CL (or L') for an edge lift condition in a one-way analysis.

In the Houston area in which $C_w = 25$; stiffening beams for BRAB slabs must be approx 24 in. (0.61 m)–30 in. (0.76 m) deep for I_p between 25 and 40; with center-to-center spacing of about 5 m. Actual beam depths used for slabs that perform successfully are perhaps 20% shallower than BRAB beam depths.

Subgrade Reaction Procedure (17,18,36).—In this numerical method the soil is represented by a two-way series of Winkler springs defined by the product of k from Fig. 2 (or from some other appropriate method) and the contributing area of the slab over which the spring acts. To account for differential swell near the edges, the "rigid supports" for the springs move (convex downward

toward the edge of the slab for center lift or concave upward for edge lift) in a "mound" configuration depicted in Fig. 9. The flexible slab-soil system is then analyzed by finite difference techniques to yield moments, shears, and deflections. Fig. 9 presents some solutions for a uniformly loaded rectangular slab using this approach. The factor y_m is a maximum differential soil movement in the free field (without the structure), which can be arrived at by taking a realistic percentage of total swell, based on uniformity of site conditions, or by using the procedure outlined in Fig. 7.

Permissible differential deflections, in terms of expected superstructure performance, are approx 0.005 L (or L'), 0.002 L, and 0.0007 L for wood, wood with brick veneer, and solid masonry construction, respectively (36).

PTI Method (25,36).—A recent refinement of the subgrade reaction method has been the analysis of slabs on an expansive, compressible elastic halfspace using finite element techniques, in which surface soil shapes near the perimeter of the slab have been ascertained using the concept outlined in Fig. 7. After analysis of numerous cases, the following design equations were developed through regression analysis:

$$M_{L'} \approx M_L = \frac{1}{727}(e^{1.238})(L^{0.013} S^{0.306} d^{0.688} p^{0.534} y_m^{0.183})(\text{ft-k/ft}) \quad \ldots\ldots (12)$$

in which L', L, e, and y_m have been defined previously; s = beam spacing in feet; d = depth of stiffening beams in inches; p = perimeter load in pounds per foot; and

$$\Delta = \frac{(y_m L)^{0.205} S^{1.059} p^{0.523} e^{1.296}}{380\, d^{1.214}} (\text{inches}) \quad \ldots\ldots (13)$$

Eqs. 12 and 13 are valid for center lift conditions, and for $e \leq 5$ ft (1.53 m). Implicit in Eq. 13 is the assumption of an uncracked section. Similar relationships have been obtained for edge lift conditions. Expressions have also been obtained for distribution of moment across an uncracked slab for purposes of draping post-tensioning cables.

Conclusions

1. Numerous rational methods exist for quantifying swell potential or swell probability, or both. Such methods are likely to yield varying results. In order for engineering professionals to communicate experiences properly with each other, a thorough understanding of the method used to arrive at the quantification and its limitations are essential.
2. The introduction of soil suction into quantification methods is possible with current technology. Suction is a more fundamental measure of potential expansion or shrinkage than are water content and index properties.
3. Rational methods of foundation design on expansive clays exist, and simple parametric solutions have been developed that can be applied in practice.

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Appendix II.—Notation

The following symbols are used in this paper:

C_u = undrained cohesion;
C_τ = suction index;
CF = climatic variation factor;
d_s = swell deficit;
e_0 = initial void ratio;
G_s = specific gravity of soil;
h = thickness of sublayer;

I_p = plasticity index;
K = coefficient of long-term subgrade reaction;
L = length of slab;
L' = width of slab;
M = moment;
P = surcharge pressure;
TMI = Thornthwaite Moisture Index;
U = zero horizontal swell pressure;
w = moisture content;
w_L = liquid limit;
Y = surface heave;
Z' = depth of active zone;
γ = soil total unit weight;
Δ = slab deflection;
τ = soil suction; and
$\bar{\phi}_{ps}$ = effective angle of plinth-soil friction.

15949 FOUNDATIONS METHODOLOGY ON EXPANSIVE CLAYS

KEY WORDS: Classification; **Clays;** Climate; **Expansion; Piers; Shrinkage; Slabs**

ABSTRACT: Recent research into the behavior of expansive clays has led to rational procedures for quantifying swell and skrinkage and differential movements in expensive subgrades. These procedures, used in conjuntion with modern numerical techniques for analyzing the static interaction of foundations and soils, have permitted the development of rational foundation design procedures that can be employed in a design office. This paper surveys recent advances in these areas and summarizes the salient features of some of the more prominent studies.

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