VARIOUS ASPECTS OF EXPANSIVE SOILS RELEVENT TO GEOENGINEERING PRACTICE

Simple Correlations Between Soil Plasticity and Expansion Potential

Soil Expansion Potential (ASTM D-4829)

This test was developed in Orange County, California in the mid-1960s and introduced in the 1973 Uniform Building Code as UBC Test Standard 29-2. It was re-designated as UBC Test Standard 18-1 in the 1994 code. This standard was adopted by ASTM in 1988. Soil material is disaggregated and passed through the #4 sieve and then brought to approximately the optimum moisture content (as determined by ASTM D-1557). The optimum moisture content equates to approximately 80 to 85% of saturation. After setting for 6 to 30 hours, the moisture-conditioned soil is compacted into a 4-in diameter mold. The moisture content is then adjusted, if necessary, to bring the sample to 50% saturation. A 144 psf surcharge is applied and the sample is wetted and monitored for 24 hours, measuring the volumetric swell. The Expansion Index is calculated as follows:

$$EI = 100 \times \Delta h \times F$$

Where Δh = percent swell and F = fraction passing No. 4 sieve

Section 1803.2 of the 1994 Uniform Building Code directs expansive soil tendency be graded by this method. The UBC mandates that "special [foundation] design consideration" be employed if the Expansion Index is 20, or greater (UBC Table 18-1-B). UBC Table 18-1-C may be applied to gain a "weighted index", allowing for a lessening of expansion with increasing depth (confinement).

EI	Expansion Potential
0 to 20	Very Low
21 to 50	Low
51 to 90	Medium
91 to 130	High
>130	Very High

According to ASTM, "The expansion index has been determined to have a greater range and better sensitivity of expansion potential than other indices" (such as Atterberg limits).

Example Design Input from Geotechnical Consultant

Two soil samples from a structure exhibiting distress from apparent uplift were subjected to the UBC Expansion Index test. The two samples were observed to swell 12.8 to 13.2%. These values are then multiplied by 100 to obtain the Expansion Index, in this case, 128 and 132. The UBC index test does not attempt to replicate any particular moisture or loading conditions that actually exist in the field, it is simply a relative index of swell potential. The soil specimen is arbitrarily subjected to moisture absorption at 50% saturation, under a normal load of 144 psf. An Expansion Index (EI) of 100 would correspond to a volume increase of 10%. In this example, the volume increase from 50% saturation was significant. The Uniform Building Code states that EI's between 91-130 are considered to have a "High Expansion Potential" and any values in excess of 130 are to be termed "Very High Expansion Potential". The distinctions are contained in Table 18-I-B of the UBC.

Empirical estimate of uplift pressures

Clayey materials can undergo relatively large volume changes in response to fluctuations in water content. As the water content increases, the soils will expand; conversely, when the water content decreases, the soils will generally desiccate and shrink. In the mid 1960s, the San Francisco firm of Lowney-Kaldveer Associates developed the following empirical relationship:

Soil Uplift Pressure = 100 (Plasticity Index) – 1000 in psf

A material with a PI of 45 could be expected to swell as much as 3500 psf. A PI of 18 could be expected to exert as little swell pressure as 800 psf.

These values can also be useful in making preliminary estimates of active and passive soil pressures acting against continuous strip footings of foundations.

IDENTIFICATION TESTS

Atterberg Limits and clay content can be combined into a single parameter called Activity. Skempton (1953) defined the term as follows:

$$\text{Activity}_{(Ac)} = \frac{\text{Plasticity Index}}{\%\text{ finer than } 2\mu m}$$

Skemton suggested three classes of clays according to activity:

- *inactive* for activities less than 0.75
- *normal* for activities between 0.75 and 1.25
- *active* for activities greater than 1.25

Active clays provide the most potential for expansion. Typical values of activities for the three principal clay mineral groups are as follows:

Mineral	Exchange-able Ion	LL (%)	PL (%)	PI (%)	SL (%)	Activity
Montmorillonite	Na^{+1}	710	54	*656*	*9.9*	*7.2*
	$K+^{1}$	660	98	562	9.3	-
	Ca+2	510	81	429	10.5	1.5
Illite	Na^{+1}	120	53	67	15.4	0.9
	K+1	120	60	42	17.5	-
	Ca+2	100	45	55	16.8	-
Kaolinite	Na^{+1}	53	32	21	26.8	0.33-0.46
	$K+^{1}$	49	29	20	-	-
	Ca+2	38	27	11	24.5	-

<u>WATER AND EXPANSIVE SOILS</u>

Water in the soil occurs as <u>three</u> types:

1. <u>Gravitation water:</u> Water free to move downward from the force of gravity, or water that drains from a soil.

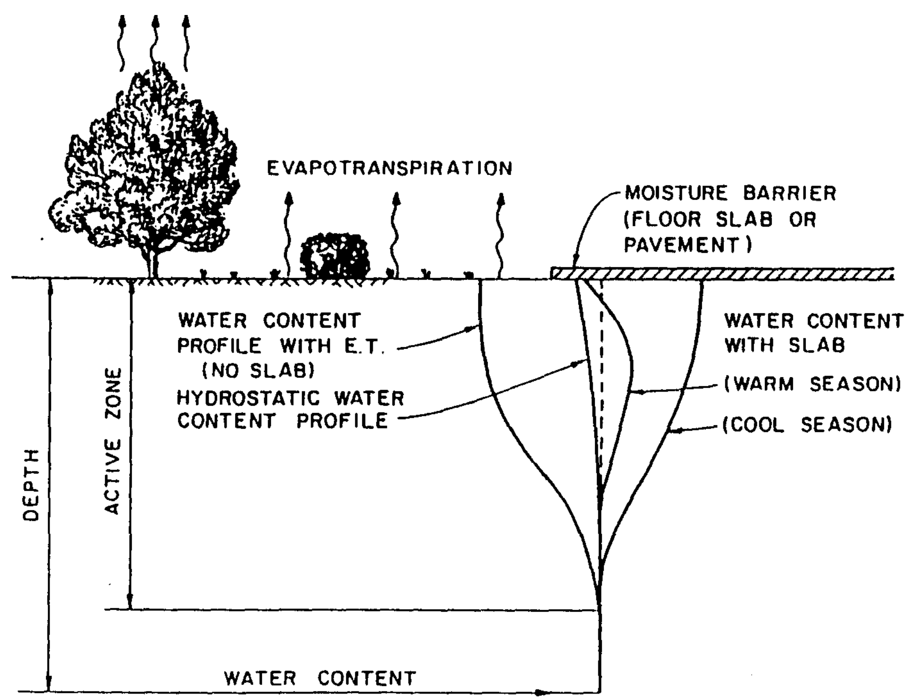
2. <u>Capillary water:</u> Water held in the capillaries or pores of the soil

3. <u>Hygroscopic water:</u> Moisture that remains after capillary and gravitational waters are removed. This moisture is in the form of a thin film held by each grain of soil. It also has a chemical affinity for the soil particle and its neighbor particle to tightly bond them together. It is also in balance with the humidity of the air.

Today, we will focus on the capillary water that heavily influences expansive spoils.

<u>Active Zone</u>

Expansive soils typically arise as a result of an increase in water content in the upper few meters from ground surface. There have been instances of deep-seated heave, but these are rare. The water contents in the upper few meters are influence by climatic conditions and environmental factor. This zone is generally termed either the *zone of seasonal fluctuation* or the *active zone.*



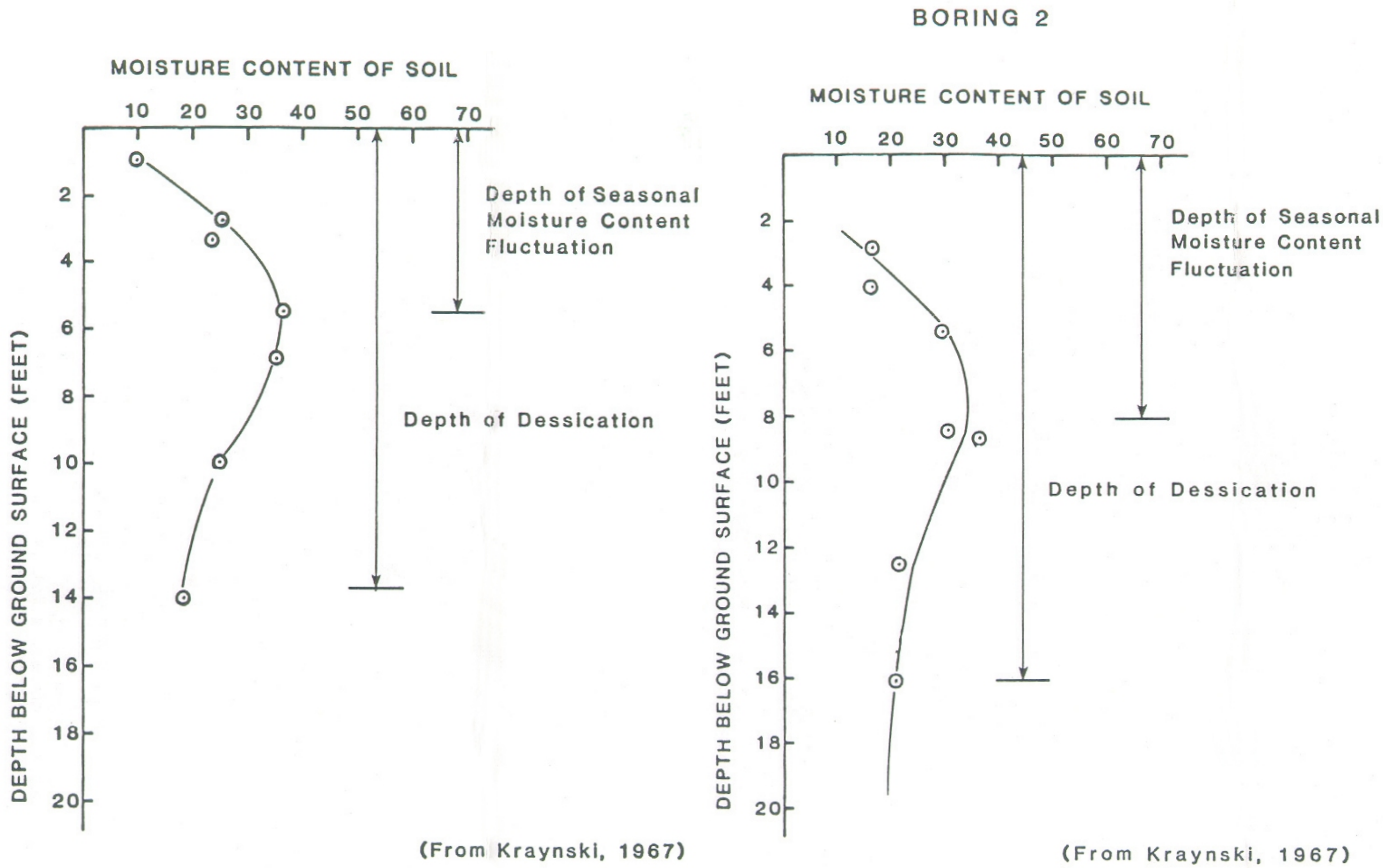


FIGURE 1 - *Typical plots of moisture content versus depth for exploratory borings in expansive soils. Note how the zone of seasonal moisture fluctuation and maximum depth of dessication can be estimated from such data.*

Laboratory tests used in identification of expansive soils

Test	Reference	Properties Investigated	Parameters Determined
Atterberg Limits	ASTM Standards 1991	Plasticity, consistency	
Liquid Limit (LL)	ASTM D-4038	Upper limit water content of plasticity	PI = LL - PL = plasticity index
Plastic Limit (PL)	ASTM D-4318	Lower limit water content of plasticity	$L1 = \frac{w-LL}{LL-PL}$ = liquidity index
Shrinkage Limit	ASTM D-427	Lower limit water content of soil shrinkage	R = shrinkage ratio L_S = linear shrinkage
Clay content	ASTM D-422	Distribution of fine-grained particles sizes	Percent finer than 2 μm
Mineralogical tests	Whiting (1964)	Mineralogy of clay particles	
X-ray diffraction	ASTM STP (1970)	Characteristic crystal dimensions	Basal spacings
Differential thermal analysis	Barshad (1965)	Characteristic reactions to heat treatments	Area and amplitude of reaction peaks and thermograms
Electron microscopy	McCrone & Delly (1973)	Size and shape of clay particles	Visual record of particles
Cation-exchange capacity	Chapman (1965)	Charge deficiency and surface activity of clay particles	CEC (meq/100 g)

Laboratory tests used in identification of expansion soils (continued)

Test	Reference	Properties Investigated	ParametersDetermined
Free swell test	I-Ioltz and Gibbs (1956)	Swell upon wetting of unconsolidated unconfined sample of air dried soil	Free swell = (V_{alet} - Vdry)fVdry *100%
Potential volume change meter	Lambe (1960)	One dimensional swell and pressure of compacted, remolded sample under semi strain controlled conditions	Swell Index (lb/ft^2) (SI) Potential volume change (PVC)
Expansion index test	Uniform Building Code	One dimensional swell under I psi surcharge of sample compacted to 50% saturation initially.	Expansion Index (EI)
California bearing ratio test	Yoder and Witczak (1975); Kassiff et al (1969)	One dimensional swell under surcharge pressure of compacted, remolded samples on partial wetting.	Percent swell CBR (%)
Coefficient of linear extensibility (COLE) test	Brasher et al. (1966)	Linear strain of a natural soil clod when dried from 5 psi (33kPa) to oven dried suction	COLE and LE (%)

Relationship between various suction units used

Height of Water Column (cm)	pF*	psi	Kgf/cm^2	kPa	Bars	Atmospheres
1	0	0.0142	0.001	0.0981	0.00098	0.00097
10	1	0.1422	0.01	0.981	0.0098	0.00968
10^2	2	1.422	0.1	9.81	0.098	0.0968
10^3	3	14.22	1	98.1	0.981	0.968
10^4	**4**	**142.2**	**10**	981	9.81	9.68
105	5	1,422	10^2	9,810	98.1	96.8
10^6	6	**14,220**	103	98,100	981	968
10^7	7	105	10^4	981,000	9,810	9,680

* The pF value is simply the logarithm to the base 10 of the capillary head (i.e., suction head) measured in centimeters of water.

Note: Typical units of insitu suction values may range from zero to over 15,000 psi (100,000 kPa). Suction values as high as 150,000 psi have been reported. These high values include a predominant component of osmotic suction.

Coefficient of Linear Extensibility (COLE)

This test is a shrinkage test used routinely by the U.S. Soil Conservation Service, National Soil Survey Laboratory, for characterizing expansive clays. The COLE test determines the linear strain of an undisturbed, unconfined sample on drying from 5 psi suction to oven dry suction.

The procedure involves coating undisturbed soil samples with a flexible plastic resin. The resin is impermeable to liquid water, but permeable to water vapor. Natural clods of soil are brought to a soil suction of 5 psi in a pressure vessel. They are weighed in air and water to obtain their volumes. The samples are then oven dried and another volume measurement is performed in the same manner.

COLE is a measure of the change in sample dimension from the moist to dry state and is estimated from the bulk densities of the clod at a suction of 5 psi and oven dry moisture conditions. The value of COLE is given by:

$$COLE = \Delta L / \Delta L_D = (\gamma_{ds}/\gamma_{dM})^{0.33} - 1$$

where $\Delta L / \Delta L_D$ = linear strain relative to dry dimensions

γ_{dB} = dry density of oven dry sample

γ_{dM} = dry density of sample at 5 psi suction

The National Soil Survey uses Linear Extensibility (LE) as an estimator of clay mineralogy. The ratio of LE to clay content is related to mineralogy as follows:

LE/Percent Clay	Mineralogy
>0.15	Smectites
0.05-0.15	Illites
<0.05	Kaolinites

Estimate of Volume Change

A constitutive relationship for volume change of an unsaturated soil may be related to the stress state variables using appropriate constitutive relationships. Because the stress state variables are independent, the stress-strain relationships must be depicted on three-axis plots, such as the one shown for void ratio below. The constitutive surfaces can be linearized by plotting the volume-weight parameters (void ratio, water content or saturation) versus the logarithm of the stress state variables. The constitutive surface shown in the figure below can be represented by an equation as follows (Fredlund, 1979):

$$\Delta e = C_t * \Delta \log(\sigma - u_a) + C_m * \Delta \log(u_a - u_w)$$

Where:

- e = void ratio
- C_t = compression index
- $(\sigma - u_a)$ = saturated effective stress state variable
- C_m = suction index in terms of void ratio and matric suction
- $(u_a - u_{,,,})$ = matric suction

The constitutive relationship for the water phase may be similarly presented:

$$\Delta w = D_t * \Delta \log(\sigma - u_a) + D_m * \Delta \log(u_a - u_w)$$

Where:

D_t = water content index with respect to saturated effective stress state variable

D_m = water content index with respect to matrix suction

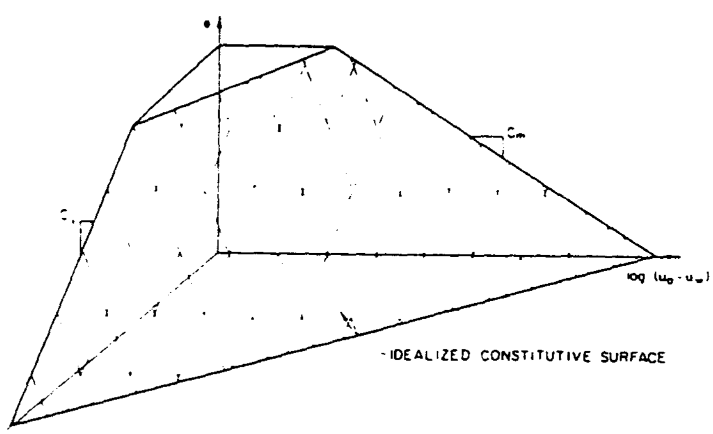


FIGURE 2 - *Idealized three-dimensional constitutive surface for unsaturated soils in terms of void ratio independent stress state variables.*