AN INVESTIGATION OF THE EFFECT OF SWELLING PRESSURE ON EARTH ANCHORED RETAINING WALL DESIGN

by
Christian H. Baxter

A thesis submitted to the Faculty and Board of Trustees of the Colorado School of Mines in partial fulfillment of the requirements for the degree of *Master of Science in Geological Engineering*.

Golden, Colorado

Date: 11/12/04

Signed: ____________________
Christian H. Baxter

Approved: ____________________
Dr. Paul M. Santi
Thesis Advisor

Golden, Colorado

Date: Nov. 15, 2004

____
Dr. Murray Hitzman
Professor and Head
Department of Geology and
Geological Engineering

ABSTRACT

Lateral pressures due to swelling soils can far exceed those expected from the weight of the soil-water system, yet there is no standard method for incorporating them into retaining wall designs. Earth anchored retaining walls (EARW), which are often utilized as low cost design alternatives to more traditional wall types, are particularly susceptible to damage from soil swell, so their use in expansive soils is avoided due to lack of an adequate design algorithm.

Unfortunately, the incorporation of swell pressure into retaining wall design is complicated by the numerous environmental and physio-chemical factors that affect the generation of swelling pressure. Direct measurement in the laboratory is notoriously inaccurate due to the inability of test procedures to adequately mimic field conditions, and in-situ testing is rarely feasible. As a result, the application of swell pressure data to retaining wall design is dependent on the method by which it was measured.

The feasibility of designing EARW's in expansive soils is based on dimensional characteristics of swell behavior. The amount of swell pressure on a structure is a function of deflection. Though in direct contact with swelling soils, EARW can be designed to be flexible, to allow swelling and the attenuation of pressure, or rigid, to restrict deflection and accommodate pressure. A design procedure was developed to utilize the elastic deflection of tiebacks and soldier beams in response to lateral loading, to reduce the design swelling pressures that expansive soil exerts on the wall due to its displacement.

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

ACKNOWLEDGEMENTS

I wish to acknowledge the support, encouragement and understanding provided by Dr. Paul Santi over the course of this project. As an educator he has few equals.

I also wish to thank John Franceski of Schnabel Foundation Company for his faith in my abilities and willingness to arrange support (which I know was not easy in tight economic times).

This research was made possible by a grant from Schnabel Foundation Company. Additional funding was provided by the Orlo Childs Fellowship, awarded by the Department of Geology and Geological Engineering, Colorado School of Mines.

INTRODUCTION

Expansion of swelling soils has been a recognized phenomenon since the 1930's (Chen, 1988). They can generate pressures strong enough to uplift foundations, crack reinforced concrete, and strain structures beyond repair. Though the estimation of damage is largely inaccurate and subjective, even conservative estimates place the cost of damage due to shrinking and swelling of expansive soils in the billions of dollars annually (Nelson and Miller, 1992).

Significant swelling soil deposits cover large regions of the United States. Eight states, including Colorado, California, Texas and many of the northern Great Plains states are recognized as having severe or moderate expansive soil problems (Chen, 1988). In Colorado, as with many of the other states, rapid population growth and the accompanied suburban expansion means that an increasing amount of development occurs each year on expansive soils.

The majority of the engineering work conducted in swelling soils deals with swelling in the vertical sense and lightly loaded structures are considered the most susceptible. Swelling, however, is three-dimensional and under certain environmental conditions, swelling in the horizontal direction can be of equal or greater magnitude to that in the vertical direction (Katti and Katti, 1994).

For earth anchored retaining walls the lateral pressures generated by swelling are an important factor (Edil and Alanazy, 1992; Trisot and Aboushook, 1983). Though vertical swell still may have unwanted consequences, lateral swelling pressures directly add to the total load carried by the wall and any associated tieback anchors. This is especially true for caisson, sheet pile, or soldier beam and lagging walls, which are installed directly into the subsurface and are in direct contact with any expansive soil. The problem is that, regardless of the fact that the damage caused by swelling soils is broadly acknowledged, the potential lateral pressures generated by swelling soils are poorly understood in reference to retaining wall design. The lack of design algorithms for lateral pressures in expansive soils may lead to uncertain designs and inefficient construction (overly- or un-conservative).

This work examines the characteristics of soil swell, the methods to quantify and predict lateral pressure effects, and the significance swelling pressures can have on earth anchored retaining walls and their design. In addition, a design method is presented for walls to be constructed in expansive materials. The need for such work is clear. In areas with considerable amounts of expansive soils, earth anchored retaining systems are being designed with uncertain regard for lateral swelling pressure. Academic and industrial literature suggests that the swelling pressures can be significant, but a suitable application of this knowledge to retaining wall design does not yet exist.

Definitions

The definition of several frequently used terms and concepts is presented to clarify their use in this work. Some of the terms examined do not have universally accepted definitions; their use in this paper, however, is consistent with the definitions presented below.

Expansive/Swelling Soils

Throughout this report, the terms expansion (or expansive) and swell (or swelling) are used interchangeably. An expansive or swelling soil is a soil that has the tendency to undergo a volume change in response to changes in moisture conditions. If in a relatively dry state, increases in moisture conditions cause them to expand, after which, any decrease in moisture content will cause them to shrink.

Swell Potential

Swell potential is a soil characteristic pertaining to the maximum amount of volumetric strain a soil sample could undergo under specific conditions. It is typically described in qualitative terms (e.g. very high, high, moderate, and low). Moisture content, density, structure, state of stress and suction all affect swell potential. Swell potential is independent of environment and is only affected by factors intrinsic to the sample.

Swell Capacity

Swell capacity is an extension of swell potential to the soil mass. It takes into account other environmental conditions such as the presence of layers of non-expansive material, fissures or shrinkage cracks, and/or applied loads.

Swell Pressure

Swell pressure has previously been defined according to the method with which it was measured. It is typically defined either as the pressure required to maintain constant volume upon an increase in moisture content, or as the pressure required to recompress a sample to its original volume after swelling. As such, its meaning is closely tied to the method used to measure it.

For this analysis, swelling pressure is defined as the external pressure exerted by an expansive soil upon wetting. Its magnitude is related to the factors that control swelling (addressed later) and the state of stress around the soil mass or sample. Swelling pressure is generated in response to any external resistance to soil expansion.

Earth Anchored Retaining Walls (EARW)

Soldier beam and lagging walls are the most common type of earth anchored wall system used in the United States and provide the design case for this study (Sabatini et al., 1999). They consist of discrete vertical elements that are typically spanned by timber lagging. The construction sequence for these walls is presented in Figure 1. Retaining support is derived from passive resistance against the soldier beam toe and lateral stress exerted by the earth anchors against horizontal pressures.

Soldier beam and lagging walls were chosen as the specific design case because the soldier beams are in intimate contact with the in-situ soil and any increases in lateral pressure are transferred to both the soldier beams and the tieback anchors. They can also be designed to be fairly flexible, a characteristic that can be utilized for dealing with soil expansion. In addition, they can provide a financially attractive alternative to other types of retaining walls built in expansive soils (if an adequate design methodology can be identified). Most of the conclusions presented in this report however, are broadly applicable to any earth-anchored retaining wall that is in direct contact with an expansive soil (e.g. sheet pile).

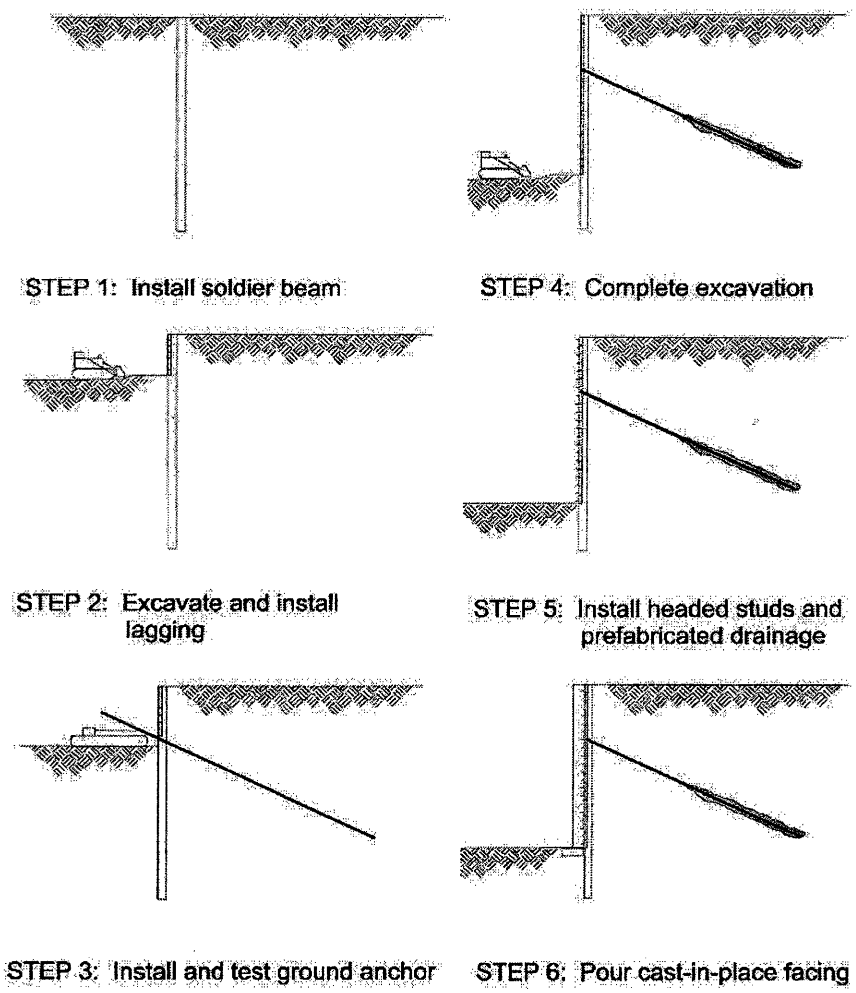

Figure 1: Soldier beam and lagging retaining wall construction sequence (Sabatini et al., 1999)

The increase in lateral earth pressures due to swelling pressures, if of sufficient magnitude, could lead to one of four types of failures (see Figure 2). The earth anchor could fail due to pullout failure, tensile failure of the tieback, or failure of the grout/tieback bond. The soldier beam or facing could fail due to bending. All of these failure mechanisms require consideration during wall design and are addressed in this research.

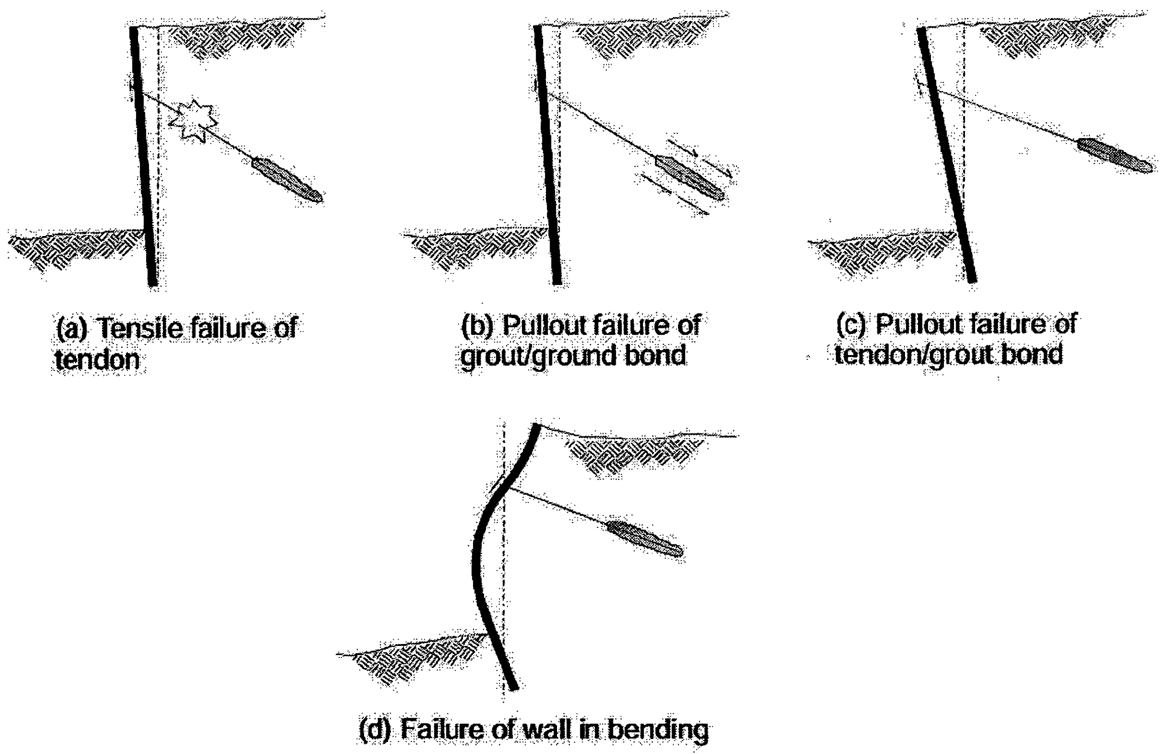

Figure 2: Examples of failure types for earth anchored retaining walls (Sabatini et al., 1999)

Design Literature Pertaining to Expansive Soils

For the design engineer, there are several publications that are available which contain design guidelines, considerations and project specifications for retaining walls. These are published by government agencies and serve to provide standards of practice for companies working on government projects. However, their usefulness and applicability extend to any project.

Several manuals deal directly with expansive soils and their affect on engineered structures including *FWHA-RD-76-82* (Patrick and Snethen, 1976), *FWHA-75-48* (Snethen, 1975), and *USACE-TM5-818-7* (USACE, 1983a). Unfortunately, they deal with vertical swelling and swell pressures and provide few guidelines for incorporating lateral swelling pressures into retaining wall design. *NAVFAC DM_7.02* notes that when working in expansive soils, "earth pressure should be calculated…with due consideration to potential poor drainage, swelling and frost action." Further explanation of "due consideration" is not provided.

The most recent work available that pertains directly to earth anchored retaining walls is *FHWA-IF-99-015: Circular No.4; Ground Anchors and Anchored Systems* (Sabatini et al., 1999). Within this volume there is no mention of expansive soils or their potential affects.

Retaining wall designers are therefore left to their own devices when working in expansive soils. There is a body of literature dealing with expansive soils but a compiled review and prescribed design methodology are not publicly available.

METHODS

This study was conducted by gathering pertinent literature concerning the behavior and characteristics of expansive soils and using it to examine design considerations and issues for EARW built in expansive soils. The literature review primarily examined journal articles, textbooks, design manuals and conference proceedings. Based on the findings of the literature review, a design procedure was developed using finite element analysis.

The objective of the literature review was to gather information pertinent to the following subjects. They represent significant factors to understanding, characterizing and anticipating the behavior of expansive soils to environmental changes. Each factor is accompanied by an explanation of its importance.

Factors affecting and mechanisms causing soil swell

This information is used to establish the environmental and physiochemical conditions needed to produce soil swell. It identifies the micro and macro scale contributors and mechanisms related to swell behavior.

The dimensional behavior of soil expansion under varying environmental conditions

Swell behavior is examined with respect to environmental conditions in order to predict the evolution of swelling pressure with changes in environment. These include state of stress and the magnitude and type of environmental change. Behavior is examined both in terms of laboratory and real world settings.

Test methods to determine or predict swell potential and pressure

Numerous tests to examine swell potential and swell pressure are available. At present there is little agreement between the academic community, who believe that many of the standard tests do not provide suitable estimations of lateral swelling (swell capacity), and industry, who are influenced heavily by cost and the unavailability of non-standard tests. Information on the advantages and disadvantages of the various methods were examined in order to delineate appropriate and practical methods for obtaining swell pressure data suitable for use in design.

Existing swell test data

This information was gathered to obtain a data set from which sample analyses could be run using the proposed procedure.

Finite element techniques were utilized to assess the feasibility of designing EARW in expansive soils and are part of the proposed design process. The reduction in swelling pressure with displacement invalidates analysis methods based on constant loadings. To calculate beam deflections/lateral stresses due to expansive soils requires one to consider the evolution of swelling pressure as a function of beam deflection. Though FEM is sometimes less accessible to practicing engineers, it provides the simplest means to accomplish this.

The finite element program utilized is p44, a publicly available free program that can be downloaded off the web (for program details and instructions on its use, the reader is referred to Smith and Griffiths, 1998). Though not intended for this type of analysis, p44 is able to calculate two-dimensional deflections of specified materials in response to user-defined loadings. In order to use p44 to analyze wall response to swelling pressure, several iterations were required to determine final deflections. Though not carried out by the program automatically, an organized spreadsheet enabled the modification of loadings for reanalysis based on previous FEM calculated deflections. The analysis is carried out using the procedure proposed in this research (details about FEM setup are available in Smith and Griffiths, 1998).

As is typical in geotechnics, several assumptions were made to simplify the conducted analysis. The following addresses the main assumptions along with discussions of their conservativism and methods by which they could be tested.

Trapezoidal loading diagram

A trapezoidal loading diagram (Figure 3) was utilized to calculate required tieback loadings prior to soil expansion. It is similar to the method described in Sabatini et al. (1999) for non-expansive soils. This is the most recent design manual for earth anchored retaining walls and many government projects specify that submitted designs follow the criteria it presents.

Soldier beam toe is fixed

Fixity is assumed to provide a reference for displacements. Due to the large embedment depths required by Sabatini et al. (1999), it is expected that displacements at the toe are minimal. This could also be the case if the soldier beam toe penetrated bedrock.

The bonded zone of the tieback is fixed

As with the soldier beam toe, the bonded zone of the tieback is assumed to be fixed. This was specified in order to minimize the number of variables. From the standpoint of tieback sizing this is a conservative assumption (any displacement would reduce the load on the anchor). However, by fixing the bonded zone of the tieback, the wall displacement may be limited (unconservative) because all anchor "pullout" is prohibited. Regardless, this assumption is made because it is considered beyond the scope of the project to model adhesion between the tieback and the soil. This is justified because displacement of the unbonded zone is necessarily minimal if the required load/displacement criterion is to be achieved. For discussion of load/displacement criteria the reader is referred to Schnabel and Schnabel (2002). Tieback testing is also discussed in Sabatini et al. (1999). The load carried by the tiebacks could be calculated based on their deflection during the analysis.

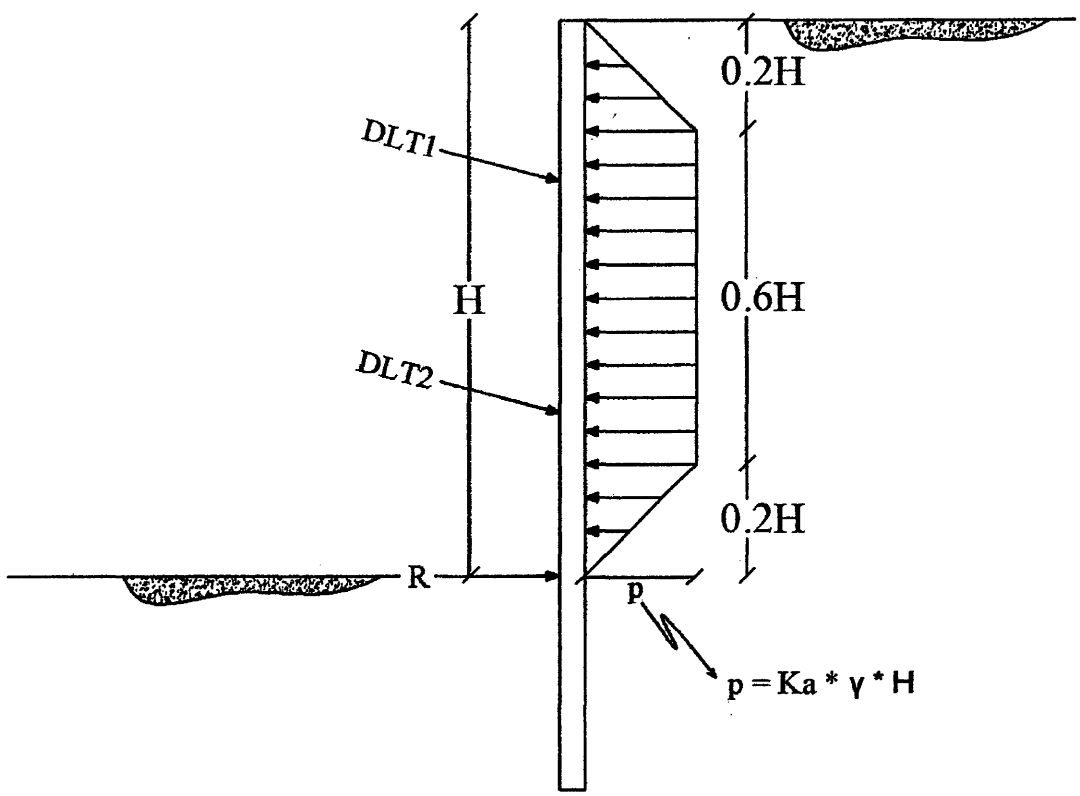

Figure 3: Trapezoidal loading diagram. DLT1 and DLT2 represent tieback loads and R represents the subgrade reaction. P is the maximum pressure ordinate.

Soil is homogeneous

While this may be an unrealistic assumption, it is conservative to assume that all of the soil is expansive. In reality the retained soils are probably stratified with non-expansive materials such as silt or sand. In addition, the effects of secondary permeability are negated because it is assumed that the soil mass over the entire width is wetted (independent of time and distribution mechanism). In practice, the relative locations of expansive layers should be noted and accounted for because an economical design requires an accurate estimate of the contact area between expansive soils and the retaining wall.

Swell pressure acts over the area consisting of the drill diameter (+6") over the entire height of the wall

Contained within the drill diameter is the soldier beam and soil mixed with minor amounts of cement. This stiffened mass is assumed to be the contact area with the expansive soil. The reason behind doing this is that the soil retained by the lagging can undergo some displacement before actually affecting the wall. During typical construction there are minor voids left between the lagging and the soil which allow space for displacement. In addition, the wood lagging is fairly compressible, also allowing for deflection. This could be a conservative assumption because the lean mix is generally weak and constructed with poor quality control. But it may also be un-conservative if large amounts of force are transferred from the lagging to the soldier beams. Measures may be taken to further validate this assumption and will be addressed later. They include the placement of a non-swelling compressible media between the lagging (or facing) and any expansive soil, or creating void space between the lagging and permanent facing.

No change in water content or soil suction occurs below subgrade

This is assumed to simplify the analysis and is considered to be justified because most drainage schemes only drain the retained soils. In addition, the gradation in suction from low to high up through the capillary fringe is ignored. This is also a conservative assumption as the swelling potential within that zone is decreased due to the presence of moisture.

Lateral swelling pressure is independent of overburden pressure

This assumption also significantly simplifies the proposed design procedure. It is conservative because at shallow depths the ability of a soil to exert lateral pressure on a structure may be significantly reduced by vertical strain.

The research paper ends with an example calculation of a retaining wall design in an expansive soil where the maximum swell pressure is 5ksf. This was accomplished using the design algorithm presented. The logic behind the analysis and use of the finite element method follow basic material mechanics principles and are explained in detail.

LITERATURE REVIEW

The factors that affect the generation of swelling pressures and soil swell are fairly well recognized and agreed upon. However, dissention exists regarding the role of each variable and an adequate means to quantify potential swelling pressures based on laboratory tests. There are simply too many dependent variables and dynamic responses that occur during swelling to anticipate swell behavior based on single variables. The following sections examine the known variables and present thoughts regarding the magnitude of their influence on swell behavior.

Dimensional Characteristics of Soil Swell

In order to characterize the three-dimensional nature of soil swell into predictive and broadly applicable terms, a large body of three-dimensional swell test data from a variety of soil types is required (advocated by McOmber and Thompson, 1992). Unfortunately, there is a lack of such data and until it is compiled, analyzed, and the relationships within it are delineated, broad extrapolations of specific observations to universal solutions are all that is possible. As such, numerous assumptions, often of questionable validity, are required in any attempt to anticipate an expansive soils in-situ behavior.

The following section examines some of the available data and observations pertinent to understanding the three-dimensional behavior of swelling soils. The topics addressed cover factors that are thought to influence or characterize the generation of lateral swelling pressure behind retaining walls.

Lateral Pressure vs. Overburden Pressure

Numerous authors note that lateral forces generated in swelling soils are much higher than active or at rest pressures. Richards and Kurzeme (1973) found that, for an instrumented excavation in expansive clay, the lateral stress values recorded down to 20 feet in depth were up to 4 times the overburden stress. Fourie (1989) notes that the lateral pressures generated may be more than twice the vertical value. Katti and Katti (1994) constructed a large scale test apparatus and found that, for the soils analyzed, the lateral pressures developed were always in excess of those expected to develop due to overburden pressures. In all cases the ratio of lateral to vertical stress was greater than one, and lateral pressure at ~3 feet depth was up to 18 times the overburden stress. In particular, a high ratio of lateral to vertical stress was observed at low surcharge pressures.

Edil and Alanazy (1992) undertook a study of three-dimensional swell behavior in a modified oedometer cell. They observed that if the vertical pressure is increased, it will restrain swelling in the vertical direction and increase the potential for volume expansion in the lateral direction. Of interest however, is that the K value (a soil characteristic defined as the ratio of vertical stress to lateral stress) changes very little upon swelling.

Dakshanamurthy (1979) also noted that for retaining walls, buried pipes, tunnels and box culverts, lateral forces due to swelling of the retained soil are much higher than active or at rest pressures. He noted that for a given mean stress ($\sigma_1+\sigma_2+\sigma_3/2$), radial strain ($\varepsilon_r$) increases with an increase in the ratio of axial stress to radial stress (σ_1/σ_3). Similarly, the swelling ratio ($\varepsilon_a/\varepsilon_r$) decreases with an increase in the ratio of axial stress to radial stress (σ_1/σ_3). The pressure exerted by the soil onto the apparatus, however, is greatest in the direction of greatest applied stress.

The Relationship between Swelling Pressure and Strain

Gromko (1974) noted that the allowance of a small amount of swell strain greatly reduces swelling pressure (only in qualitative terms and accompanied by no supporting data). Fourie (1989), while conducting three-dimensional hydraulic triaxial swell tests, similarly observed marked reductions in lateral pressure with small increases in lateral strain. He measured a 35% reduction in lateral pressure with 1% radial strain.

Katti and Katti (1994) also examined the relationship between swelling pressure and strain using their large scale testing device. They measured the affects of various wall rotations on the lateral pressure generated by an expansive soil, prior to and after saturation. From their data it is evident that the lateral pressure exerted by an expansive soil decreases with increasing wall displacement away from the soil mass (active conditions). For the data they presented, the lateral swell pressure in all cases reached a constant value by ~0.5% strain. That constant value was, on average, 45% of the lateral pressure measured prior to any wall movement.

Wallace and Lytton (1992), examined lateral pressures and swelling in a cracked expansive clay profile. They observed that initial swelling may be taken up by shrinkage cracks so that, with respect to lateral pressure development, the effective lateral swell strain is reduced. Field observations of cracks in the retained soil profile are therefore important to note because allowance of small increases in volume can significantly relieve swelling pressure (ASTM, 2003). It should be noted however that cracks in the soil profile accelerate water movement through the soil. Johnson and Snethen (1978) estimate that the presence of fissures reduces lateral pressures within the soil, limiting the amount of vertical heave to about 1/3 of the volumetric swell. Similarly, Coduto (2001) (in qualitative terms) notes that swelling pressure on a retaining wall can be reduced by allowing some expansion through wall flexibility.

Shanker et al. (1987) studied three dimensional swell behaviors by testing cubic samples. They observed that percent swell is almost proportional to the number of directions that the sample was allowed to swell (see Figure 4). This indicates that swell is not volumetrically constant, i.e. a sample allowed to swell freely undergoes a larger volume increase than one that is even partially restrained.

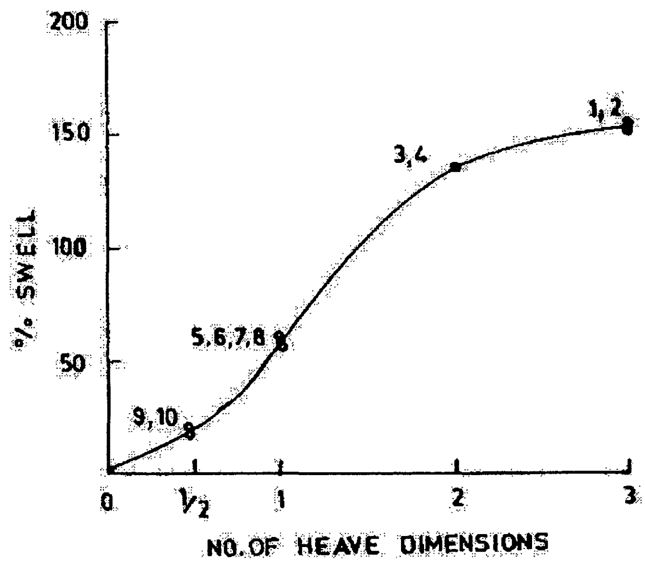

Figure 4: Shanker et al. (1987) Effect of Dimensionality of Swell on percent Swell

The Variation of Lateral Swelling Pressure with Depth

The evolution of lateral soil pressure with depth was quantified by Katti and Katti (1994) using a large scale testing device. They observed that lateral pressures generated by soil swell increased rapidly from 0 at the surface to a maximum value between 3 to 5 feet in depth (for 100% saturation). From that point, the lateral pressure due to swelling remained relatively constant. This curvilinear pressure distribution is quite different from the pseudo-linear distributions contributed by the weight of the soil mass and hydrostatic pressure. Similar trends were observed for density, moisture content and vane shear strength: until the maximum lateral pressure was reached (at relatively shallow depth), density increased, moisture content decreased and vane shear strength increased (then stabilized).

The above observations made by Katti and Katti (1994) however, contradict how one would intuitively expect lateral swelling pressure to evolve with depth. Instead of reaching a maximum and constant value at a shallow depth, one would expect lateral swelling pressure to evolve slowly with depth. The reason is that, down to depths where the overburden stress is less than the maximum swelling pressure, most of the volumetric strain would occur vertically (in the direction of minor stress). This vertical strain would reduce the swell pressure that could be generated by the soil (see the previous section). Therefore one would expect lateral swelling pressure to evolve as a function of increasing overburden stress. Only at a depth where the overburden stress exceeded the maximum swelling pressure would a constant lateral swell pressure be expected. This is inconsistent with the findings of Katti and Katti (1994) who found that swelling pressure reached a constant and maximum value at overburden stresses that were significantly lower than the maximum swelling pressure. Direct data to contradict their findings however was not discovered during this research. Friction within the soil mass may partially explain this inconsistency.

Factors Affecting Soil Swell

The factors affecting soil swell are well studied and the reader is referred to Nelson and Miller (1992) for a complete list and associated references. Factors affecting lateral swell behavior of a soil in either the field or the laboratory are presented below. Particular attention is given to those factors that may be affected by sampling and testing because many design decisions are based on laboratory test results. The commonly utilized testing methods are examined in a later section.

The following factors influencing the swelling characteristics of expansive soils was compiled from Parcher and Liu (1965), Gromko (1974), Katti and Katti (1994), Chen (1988), and Nelson and Miller (1992). In general, soil properties determine the potential for swell, while environmental conditions determine the magnitude and rate of swell (Johnson and Snethen, 1978). The soil characteristics examined include mineralogy, grain size distribution and soil structure. The environmental conditions examined are moisture condition, stress history, density, state of stress and time.

Mineralogy

The most troublesome group of clay minerals in terms of swelling is montmorillonite clays (Likos, 2000). During formation of the clay mineral, isomorphous substitution within the crystalline layers leads to a net negative charge imbalance on the interparticle surface. To maintain electroneutrality, the negative charge is offset by exchangeable cations (e.g. Na^{+} or Ca^{2+}) which are maintained in a diffuse layer above the mineral surface (Chen, 1988; Sridharan et al, 1986). The capacity to hold exchangeable cations is especially strong in montmorillonite clays, which have a very large specific surface. Swelling occurs due to repulsive forces between adjacent diffuse layers and absorption of water in polar bonds. Unlike other clay minerals, montmorillonite undergoes both interlayer and interparticle expansion, causing a greater degree of swell. Other clay minerals do not undergo appreciable interlayer expansion.

Though the swelling capacity of a soil mass depends on the amount and type of clay minerals present (Nelson and Miller, 1992), mineralogy is not typically utilized in engineering applications. Mineralogical identification requires specialized equipment (e.g. x-ray diffraction or electron microscope resolution), and does not provide an adequate indication of macroscopic engineering behavior.

This is because soils never consist purely of homogeneous clay minerals: numerous other factors affect swell behavior that are better represented by larger scale tests.

Soil Suction

Moisture retention force or soil suction is a measure of the tendency of the soil to undergo a change in moisture content and is directly related to the volume change characteristics of expansive soils. As the moisture content increases, the soil suction decreases (an action) and the soil swells (a reaction) (Snethen and Huang, 1992; Mawire and Senneset, 2000).

For characterization of swell behavior soil suction tests are simple, economical, expedient and at least as capable as consolidometer methods at simulating field conditions. Also only one test is required to delineate a single suction/water content relationship for the range of water contents in the field (Johnson and Snethen, 1978). Soil suction however, is not directly related to swell pressure and results in qualitative classification with respect to swell behavior.

Grain Size Distribution

El Sohby and Rabbaa (1981) examined the effect of grain size distribution on swell behavior. As reason would conclude, they found that as the percent of clay increases, swell potential increases. However, they noted that for a given percent of swelling clay, silty soils swelled more than clayey soils. This is postulated as being due to the cohesive strength of clayey soil which resists relative displacement within the soil mass.

Soil structure

The importance of particle alignment to swell behavior is noted by several authors (Seed et al., 1962; Parcher and Liu, 1965; Mitchell, 1993; Gromko, 1974;

Komornik and David 1969, Nelson and Miller, 1992; Al Shamarani and Dhowwian, 2003). Dakshanamurthy (1979) demonstrated that, in an isotropic stress field, swelling behavior is anisotropic due to the anisotropic nature of the clay fabric. If anisotropic stress systems have been applied to the soil in the past, then anisotropic swelling characteristics usually result (Mitchell, 1993). The platy geometry of clay particles causes them to preferentially align with their interlayer surfaces perpendicular to any applied pressure. As a result, repulsive forces between adjacent diffuse layers occur in the same orientation as the applied stress. Mitchell (1993) demonstrated that compacted expansive soils with flocculent structures are likely to be more expansive than those with dispersed structures. Experimental work carried out by El-Ala-Habib (1995) also clearly indicates that the soil structure has a significant contribution to the development lateral pressure in expansive clay. He attributed changes in soil behavior with different stress paths to alteration of soil structure.

It should be noted that remolding during sampling and testing alters soil structure, often resulting in a more dispersed structure (Nelson and Miller, 1992). ASTM D4546-03 notes that compaction methods such as kneading or static compaction may influence the volume change behavior when prepared wet of optimum. Yevnin and Zaslavshy (1970) demonstrated that higher moisture content causes more structural changes during compaction, reducing the swelling tendency. Gromko (1974) also noted similar trends. He observed that compaction methods that induce shear strains (dispersed structures) reduce heave upon wetting. Kneading, versus static compaction, was indicated as producing a more dispersed structure. Edil and Alanazy (1992) also examined the effect of soil structure on swelling characteristics. They noted that samples compacted to the same density and moisture content, by different compaction methods, exhibited different swelling characteristics. In general, static compaction was shown to produce higher lateral swelling pressure than kneading owing to different particle arrangements (contradictory to the findings of Gromko, 1974).

There is general agreement that swell behavior is influenced by soil structure and that in-situ soil structure is altered by sampling and test preparation. Unfortunately however, there is little data regarding the magnitude of swell behavior change due to remolding. Therefore samples should be kept in as close to natural state as is possible prior to and during testing for the most representative results.

Moisture condition

Parcher and Liu (1965) showed that the magnitude of swelling is proportional to the amount of water imbibed, so the capability for swelling is related to the in-situ moisture content (see Figure 5). Several authors have noticed that, intuitively, as the initial water content increases, the magnitude of swell decreases (Al-Shamarani and Al-Mhaidib, 2000; Petry et al., 1992; El Sohby and Rabbaa, 1981; Shanker et al., 1987; Parcher and Liu, 1965).

For field conditions, the realization of swell potential, and the accumulation of significant swell pressure, requires that in-situ soil starts in a relatively dry state (<15% moisture content (Chen, 1988)), followed by exposure to moisture. Chen (1988) has observed cases where detrimental swelling occurred with only a 1-2% increase in moisture content. This condition is often realized in arid regions when people irrigate their lawns by an amount larger than the average precipitation rate or otherwise introduce water into the relatively dry subsurface. Similarly, for swelling soils to affect a retaining wall, the moisture regime in the soil must undergo a significant moisture change. This can occur due to improper or inadequate drainage, or from other water sources like leaking utility lines or upslope irrigation.

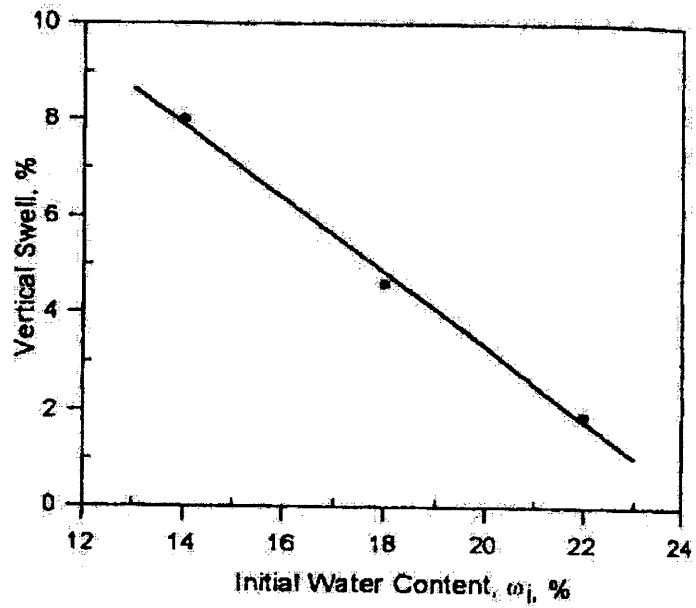

Figure 5: Vertical Swell vs. Initial Water Content (Al-Shamarani and Al-Mhaidib, 2000a)

The amount of swell that occurs in a given period depends on the quantity of water that enters the soil; thus, the rate of swell is dependent on the hydraulic gradient and permeability of the soil (Edil and Alanazy, 1992). Even small increases in water content may cause significant amounts of heave. In order to predict heave at any time it is necessary to define the zone of soil that has experienced an increase in water content and what the swell potential of that zone is. In order to compute the maximum heave it is necessary to assume that the depth of wetting will proceed throughout the depth of potential heave. Conservative design must consider the maximum amount of heave that can occur in the lifetime of the structure. The depth of transient wetting, or active zone, commonly extends to ~20 feet in arid climates (Nelson et al., 2001). For retaining wall design it is necessary to consider an unlimited width of wetting behind the wall to consider to "worst" case. For areas with deep permanent water tables the active zone may be alternatively defined as the depth at which the overburden vertical stress equals or exceeds the swelling pressure of the soil (Nelson et al., 2001). Below that depth, the generation of swelling pressures results in no relative soil displacement (in a vertical sense).

Stress history

Pre swell-test stresses change the physical properties of soils, drastically affecting measured swell. Petry et al. (1992) tested samples under varying three dimensional stress environments. They demonstrated, through the use of a triaxial device, that when higher pre-test stresses were applied, higher surcharge pressures were needed to prevent swelling. Komornik and David (1969) similarly found that when a sample was subjected to a higher preload, it demonstrated a higher swelling pressure.

Density

Soil density is closely tied to stress history. Katti and Katti (1994) observed that samples tested with lower void ratios (higher density) resulted in higher swelling pressures for given clay contents (see Figure 6). In general, higher densities indicate closer particle spacing and therefore greater repulsive forces. This leads to an increase in swell potential and higher magnitude swell pressures (El Sohby and Rabbaa, 1981). Komornik and David (1969) demonstrated this by testing undisturbed and artificially consolidated samples from the same expansive soil. The swell pressure generated by the consolidated sample exceeded the swell pressure generated by the undisturbed sample by 34 times. Parcher and Liu (1965) also noted a direct relationship between compactive effort and magnitude of swell.

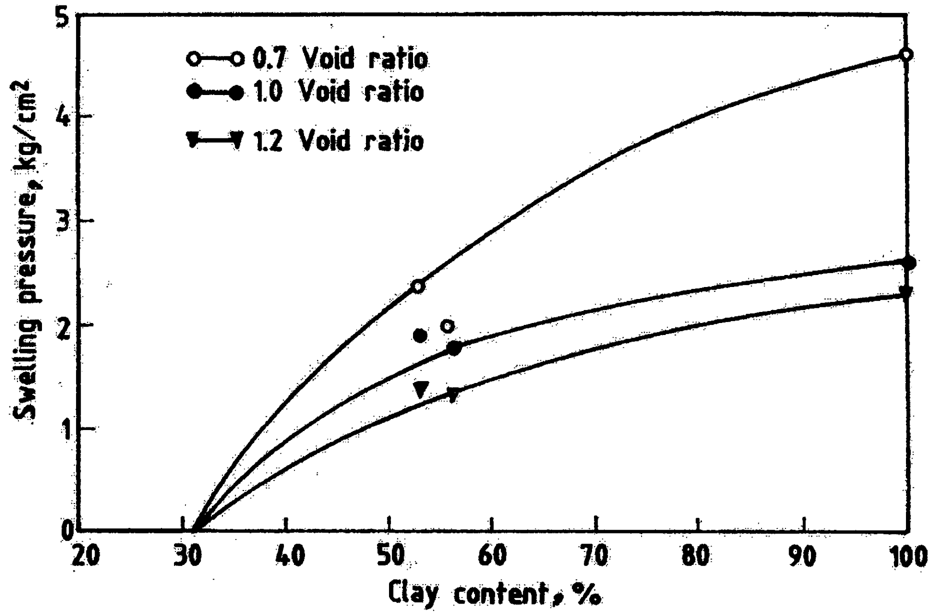

Figure 6: Swelling Pressure vs. Clay Content for Soils with Various Void Ratios (Katti and Katti, 1994)

State of stress

It should be noted that, while strain is significantly affected by state of stress, pressure generation is not. If the horizontal and vertical confining stresses exceed the swell pressure, no sample strain will be observed; however, swelling pressure is still generated. The state of stress may affect sample density though, altering the potential magnitude of swelling pressure that can be generated (see above).

Time

The time required for a soil to reach its maximum swell potential varies considerably and depends on the initial density, permeability and thickness. For remolded laboratory samples, generally 24 hours is required to realize 95% of potential (Chen, 1988). Katti and Katti (1994) noted that 45 days was required to saturate their large scale testing apparatus (~4.5 ft. x 3 ft. x 10 ft.). Primary and secondary permeability significantly effect the movement of water through a soil mass or sample, and therefore the time required to reach full swell potential or capacity. For typical triaxial devices ~15 days is frequently required for accurate analysis (Fourie, 1989).

Qualification of Swell Potential

There are at least 40 schemes presented in the literature to predict swelling pressure and heave in qualitative terms based on varying soil properties. These have met with varying degrees of success depending on their level of rationality (Mawire and Senneset, 2000) and no scheme has been accepted as a standard. Regardless of their shortcomings though, qualitative schemes provide early indications of swell potential based on typically obtained soil parameters (e.g. % fines, Atterberg limits). By examining their swell potential classification, select soils can be chosen for more rigorous and expensive quantitative tests. Qualitative classifications at this point are not suitable for entry as numerical design parameters.

Nusier and Alawneh (2002) attribute serious problems at a site in Irbid City, Jordan, to a lack of recognition of high shrink/swell potential prior to the design stage. They stress the importance of early recognition of expansive soils so appropriate mitigation measures can be taken from the design phase of any project. Qualitative schemes often allow this to be carried out without tests.

Table 1 shows several of the more common schemes utilized today, and the classification criteria. Unfortunately, a standardized classification procedure has not evolved, and the universal application of any scheme outside of the soil type, or test condition, for which it was developed may be misleading and of little value (Nelson and

Table 1: Qualitative classification schemes for expansive soils (modified from Chen, 1988; and Nelson and Miller, 1992).

REFERENCE	BASIS			PREDICTIVE RESULT	
A. Altmeyer, 1955	**Linear Shrinkage**	**Shrinkage Limit (%)**		**Probable Swell (%)**	**Degree of Expansion**
	>8	<10		>1.5	Critical
	5 to 8	10 to 12		0.5 to 1.5	Marginal
	<5	>12		<0.5	Noncritical
B. Holtz and Gibbs, 1956	**Colloid Content**	**Plasticity Index**	**Shrinkage Limit (%)**	**Probable Expansion (%)**	**Degree of Expansion**
	>28	>35	<11	>30	Very High
	20 to 31	25 to 41	7 to 12	20 to 30	High
	13 to 23	15 to 28	10 to 16	10 to 20	Medium
	<15	<18	>15	<10	Low
C. Chen, 1965	**% fines**	**Liquid Limit**	**SPT**	**Probable Expansion**	**Degree of Expansion**
	>95	>60	>30	>10	Very High
	60 to 95	40 to 60	20 to 30	3 to 10	High
	30 to 60	30 to 40	10 to 20	1 to 5	Medium
	<30	<30	<10	<1	Low
D. Raman, 1967	**Plasticity Index**	**Shrinkage Index (%)**			**Degree of Expansion**
	>32	>40			Very High
	23 to 32	30 to 40			High
	12 to 23	15 to 30			Medium
	<12	<15			Low
E. Snethen et al., 1977	**Plasticity Index**	**Liquid Limit**	**In-situ Soil Suction**	**Potential Swell (%)**	**Swell Classification**
	>35	>60	>4 tsf	>1.5	High
	25 to 35	50 to 60	1.5 to 4	0.5 to 1.5	Marginal
	<25	<50	<1.5	<0.5	Low
F. Chen, 1988	**Plastcity Index**				**Swell Potential**
	>35				Very High
	20 to 55				High
	10 to 35				Medium
	0 to 15				Low
G. McKeen, 1992	**Suction Compression Index**	**Change in Suction/ Change in Moisture**		**Vertical Strain (%)**	**Potential for Field Heave**
	>-6	-0.227		>10	Special Case
	-6 to -10	-0.227 to -0.120		10 to 5.3	High
	-10 to -13	-0.120 to -0.040		5.3 to 1.8	Moderate
	-13 to -20	-0.040 to nonexp.		1.8 to 0	Low
	<-20	nonexpansive		0	Nonexpansive

Miller, 1992). Fityus and Smith (1998) similarly noted that the style and magnitude of swell movement can vary widely between expansive soil sites.

The purpose of this section is to present some of the available schemes that can be used for early prediction of swell potential. Depending on the geotechnical parameters obtained during early investigations these parameters can be used to identify samples or units for which potential swelling pressure should be quantified for use in design.

The variation between the swell potentials (% probable expansion) predicted by the various schemes is evident and was studied by Chen (1988). He found that the variability is a reflection of the different conditions under which the schemes were developed. As such, application of any scheme to prediction of swell potential requires the user to consider its sphere of validity and if it is accurate for the soils on which it is being used (Rao and Smart, 1980). For several of the tests in Table 1 and many others, Chen (1988) noted that the index values, though always characteristic of expansive soils, are not unique to soils that swell. For example, all soils with high swell potential have a high plasticity index, but not all soils with a high plasticity index are expansive.

Bandyopadhyay (1981) proposes another scheme from 259 soil samples collected in Kansas. He estimates swell pressure based on % clay fraction, activity and % free swell under a 1-psi surcharge. The author claims its universal applicability, yet it has not gained widespread acceptance. It should be noted that this scheme, along with many of those that follow, is self evaluated (by those with the greatest interest in demonstrating their universal applicability). This introduces the possibility of bias, even subconsciously, into the evaluation, perhaps influencing the findings.

Rao and Smart (1980) placed soils in closely defined "families" based on particle size similarity. Data regression equations were calculated for each family that enabled prediction of both swell pressure and potential "within 10% error." Unfortunately, the relations were based on only 10 soil samples from the same geologic unit so the applicability of the findings elsewhere seems dubious.

Snethen (1984) evaluated 17 qualitative classification schemes. His findings indicate that LL, PI, and natural soil suction provided the most accurate estimation of swell potential and classification.

Dakshanamurthy and Raman (1973) proposed a classification based on LL, PI and SL from a specific soil unit but also compared to data taken from other research publications. Their scheme has not gathered widespread acceptance.

Covar and Lytton (2001) propose estimating soil suction compression index from LL, PI and fine clay fraction, and then using that information for classification using previously published methods like that proposed by Lytton (1994). Soil suction is also used by Garbulewski and Zakowicz (1995) to assign fine grained soils into swelling groups. As a classification parameter soil suction is fairly promising though the test is time consuming and not universally available.

Vijayvergiya and Ghazzaly (1973) also propose classification methods using LL and dry unit weight or LL and water content. The applicability of this method is dubious because the variability of the soils from which the correlations were derived is not known. They simply indicate that the soils came from a "wide geographical coverage".

For retaining wall design, qualitative schemes can be used to identify expansive soils that may be problematic and require more rigorous quantitative testing for use in design. In areas where highly expansive soils have been previously identified, such as along Colorado's Front Range, "local experience" may also provide an indication of expansive soils that should undergo direct swell pressure measurement. At this time qualitative schemes are not specific enough for suitable use in retaining wall design. For early classification however, the method proposed by Chen (1988), based on PI, is probably suitable for identifying samples that should undergo testing specific to quantifying swell behavior.

Quantification of Swell Pressure

The results of all laboratory swelling tests are at best rough approximations, partly because of inevitable changes in water content and structure of the soil during drilling, sampling and handling in the laboratory (Terzaghi et al. 1996). In addition, some factors which govern behavior in the field are difficult to simulate in the lab. These include stress path, drainage, water infiltration, confinement, etc. (Edil and Alanazy, 1992). Reliable estimate of swell pressure however, represents the most important factor influencing the selection of treatment alternatives or design preparations to accommodate soil expansion (Al-Shamrani and Al-Mhaidib, 2000b).

Porter and Nelson (1980) advocated the use of strain controlled testing for characterizing vertical swell behavior for foundation design. Their tests were carried out using strain consolidometers modified to allow for control of strain and measurement of the resulting loads. A similar concept is applicable to retaining wall design.

The swell potential of a soil sample is intrinsic to the sample analyzed. It represents the maximum pressure a sample could generate under ideal conditions. Ideal conditions include zero displacement, and the maximum change in water content possible (dry to saturated). Swell potential is independent of the size of the sample analyzed and therefore represents the upper bound to swell pressures acting on a wall.

The magnitude of the swelling pressure that a soil can exert on a structure is difficult to quantify accurately due to the large number of variables that affect a soil's swell behavior. Al-Shamarani and Dhowian (2003) considered the multi-dimensional study of expansive soils to be one of the "most complex and analytically intractable problems in geotechnical engineering." The difficulty arises due to differences between in-situ conditions and those in the laboratory. Several factors that affect swell pressure generation are the same for both; these include mineralogy, percent clay fraction, cation exchange capacity and the exchangeable cation. Other in-situ conditions can be approximated in the lab, such as soil density, initial moisture content and confining stress.

However the loss of soil structure due to remolding of samples, and the affects of testing methodology on swell pressure generation, are two factors that cause discrepancies between potential in-situ swell pressures and those measured in the lab (El Sayed and Rabbaa, 1986). In addition, tests requiring the application of an external load introduce mechanical effects to the swelling process, altering the results (Mawire and Sennesest, 2000).

There are four principle methods of quantifying lateral swelling pressure: one-dimensional oedometer tests, triaxial tests, large scale tests, and predictive equations based on various soil properties. Of these, oedometer tests are the most commonly employed to provide swelling pressures as design parameters (Trisot and Aboushook, 1983; Nelson and Miller, 1992). For lateral pressure prediction it is assumed that the one-dimensional (vertical) swell pressure in the laboratory is comparable to the lateral swell pressure a soil could develop in the field. Triaxial tests provide information on both lateral and vertical swell pressure while allowing for more environmental control. As such they are more complicated, require specialized equipment and are typically more expensive. Large scale tests are difficult to conduct and are typically only employed in a research environment (Katti and Katti, 1994). Instrumentation of constructed projects has also been utilized to provide quantitative values of swelling pressure (Richards and Kurzeme, 1973). Each of these methods are discussed below.

Oedometer Swell Tests

Numerous procedures are used to measure swell pressure with oedometers. For a complete list of swell prediction methods the reader is referred to Nelson and Miller (1992) and Chen (1988). Though most fit roughly into one of the following categories, none are accepted as the universal standard. All of the typically utilized oedometer tests however, fall into one of three general categories (see Figure 7)(Sridharan et al, 1986, El Sayed and Rabbaa, 1986; Nelson and Miller, 1992):

For retaining wall design in expansive soils there are two parameters which are required for a reasonable design. The first is the maximum swelling pressure that the retained soil can generate, and the second is the evolution, or dissipation, of that swelling pressure with wall deflection.

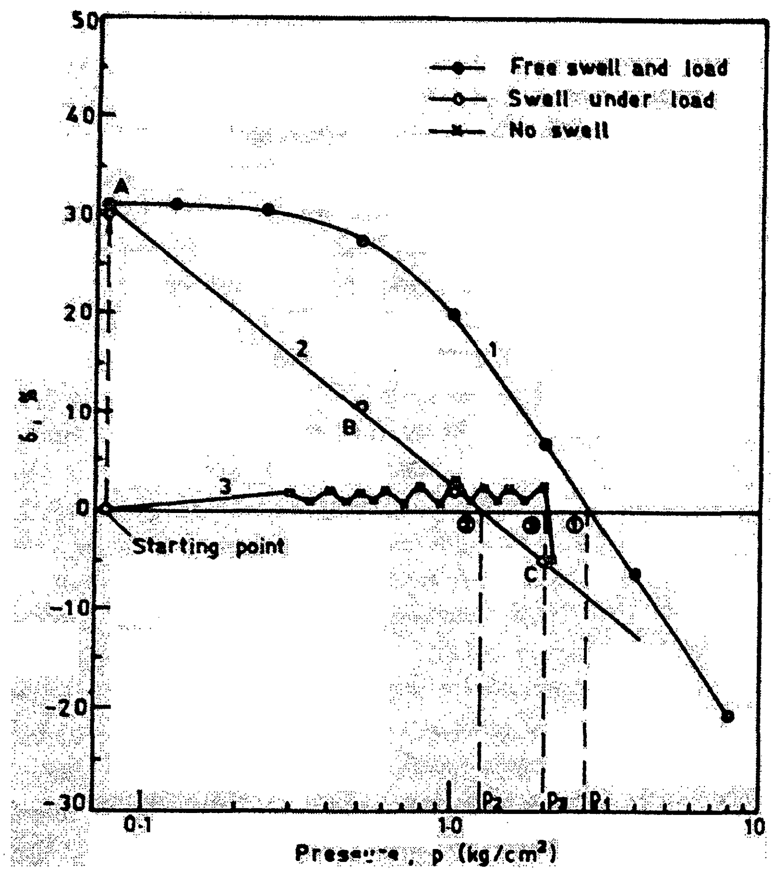

Figure 7: Results from various methods of swell pressure tests. The different stress paths taken by samples during constant volume (3), constant pressure (2) and swell consolidation methods (1) (Sridharan et al., 1986).

Constant Pressure

Three or more specimens are placed in oedometers with identical initial conditions. Different loads are applied and the samples are allowed to reach dry-state equilibrium. The samples are then saturated and allowed to swell until equilibrium. Vertical movements are then plotted against the applied pressure and a best fit line is drawn. The x-intercept corresponds to the swelling pressure at zero volume change (El Sayed and Rabbaa, 1986).

A variation of this test is to calculate the pressure at which the void ratios in the dry and saturated condition are equal. The premise is that void ratio increases as swelling occurs, so the initial void ratio is analogous to the zero volume change condition. As such the swelling pressure is the pressure at which the void ratio remains constant upon wetting. This again is the x-intercept, on a plot of vertical displacement as a percent vs. log applied pressure (Sridharan et al, 1986).

Swell-Consolidation

A sample is seated in the oedometer under a relatively minor load under dry conditions. It is then saturated and allowed to swell freely. Once equilibrium is reached, the sample is incrementally loaded until the initial volume condition is reached. The pressure required to accomplish this is taken as the swelling pressure (Holtz and Gibbs, 1956; El Sayed and Rabbaa, 1986; Sridharan et al, 1986).

Sikh (1993) proposed a method with which swelling pressure could be estimated if free swell were known. By testing numerous samples from various different soils using swell-consolidation procedures an equation was derived relating free swell to swell pressure that is thought to be broadly applicable. The applicability of the relationship however is questionable because of a relatively large scatter in the data, and reconsolidation is required to obtain swelling pressure for the tested soil.

Constant Volume

In this method a sample is seated in the oedometer and allowed to imbibe water under a zero volume change state. The constant volume condition is maintained by applying pressure to the sample in response to observed strain. In practice it is noted that maintaining zero strain is difficult and that the test requires uninterrupted human control for often long periods of time. The pressure at which a zero strain condition is maintained is regarded as the swelling pressure (Holtz and Gibbs, 1956; El Sayed and Rabbaa, 1986; Sridharan et al, 1986).

Double-Oedometer Test

In the double oedometer test, two identical samples are compressed under different pressures. The first sample remains in a dry state for the duration of the test, and the second is allowed to swell and is then compressed. The results are then plotted (as void ratio vs. log pressure) and the intersection of the two curves is taken as the swelling pressure (El Sayed and Rabbaa, 1986). It is noted that questions exist regarding the corrections needed to appropriately correlate the two curves (Nelson and Miller, 1992).

Sridharan et al (1986) compared results from the *constant volume, swell consolidation* and *constant pressure* tests. They noted a general lack of correlation between the test results. The *swell-consolidation* test generally yielded the highest value, while the *constant pressure* test was generally the lowest.

Edil and Alanazy (1992) also noted that different test methods yield different results. According to their analysis, more pressure was required to prevent vertical expansion (*constant-volume* method), than to recompress the sample to its original volume after some degree of swelling (*swell-consolidation*). They also observed that the magnitude of lateral pressure generated with each test method was different. During constant volume tests the magnitude of lateral pressure increase was almost three times greater than that generated during swell-consolidation tests. They also showed that in swell-consolidation tests, the increase in vertical pressure did not add appreciably to the lateral pressure measured. These findings stress the importance of stress path in swell pressure testing.

Khaddaj et al. (1992) found that the constant pressure type tests yielded significantly different results from the constant volume or swell consolidation methods. They attributed this difference to the fact that the test requires three different samples and may therefore be influenced by inhomogeneity of the deposit. The reason that representation of the inhomogeneity in the soil is sited as a negative attribute is not clear. It seems that the more of the soil mass that is represented the broader the applicability of the test. Testing multiple samples also limits the possibility of testing a sample with anomalous properties compared with the rest of the soil mass. This is a concern due to the small number of samples on which swelling tests are usually conducted.

Regardless of the specific category of test employed, the one-dimensional oedometer tests for determining potential swell pressures are generally considered to be somewhat inaccurate (El Sayed and Rabbaa, 1986; Fourie, 1989; Katti and Katti, 1994; Al-Shamarani and Al-Mhaidib, 2000a; Al-Shamarani and Dhowian, 2003). Some authors have proposed a correction factor, but none of these have gained widespread acceptance (e.g. Al-Shamarani and Al-Mhaidib, 2000(a & b)). In general, the one-dimensional approach overestimates swelling pressure and heave considerably. The following are the reasons most frequently cited for the often large discrepancies between laboratory results and field conditions:

Skin friction between the sample and the apparatus

Numerous authors site skin friction as a major contributor to inaccurate test results (El Sayed and Rabbaa, 1986; Fourie, 1989; Katti and Katti, 1994; Al-Shamarani and Al-Mhaidib, 2000; Al-Shamarani and Dhowwian, 2003; Trisot and Aboushook, 1983 etc.). In general the affect of skin friction is to inhibit swell by some undetermined amount. Due to the nature of the test, the *swell-*

consolidation method is significantly affected by skin friction, and therefore provides highly questionable results (Trisot and Aboushook, 1983). This leads to an underestimate of swelling potential, even if the walls are lubricated (Khaddaj et al., 1992).

Lateral confinement

The overestimation of swell pressure and swell potential by one-dimensional oedometer tests is largely the result of rigid lateral confinement. Lateral swell and lateral confining pressure are not simulated with oedometer tests (ASTM, 2003). McKeen (1981) noted that, particularly in the early stages of expansion, there is potential for lateral expansion that is not reflected to vertical swell. Al-Shamarani and Dhowian (2003) observed that the entire observed swell is in the vertical direction, while in the field, vertical swell is only one component of the total volume change. As such the one-dimensional test essentially measures volumetric strain and hence is an overestimate of in-situ one dimensional movement (El Sayed and Rabbaa, 1986; Fourie, 1989; Dakshanamurthy, 1979; Shanker et al, 1987). The necessity of a lateral restraint reduction factor is noted. Several authors suggest using a correction factor of 1/3 for oedometer tests to account for the effects of lateral confinement (Al-Shamarani and Al-Mhaidib, 2000a; Fityus and Smith, 1998; Dhowian, 1990).

The constant-volume technique addresses the problem of one dimensional movement to some degree by not allowing any strain to occur. However, overall specimen confinement may be an issue (Khaddaj et al., 1992). Under such conditions the accumulation of swelling pressure means that the sample is subject to an increasing state of stress. This may or may not be consistent with swelling conditions in the field, but the phenomena does affect measured swelling pressure. In the field, overburden stresses are generally constant (not increasing as in the constant volume swell test). Unfortunately, Khaddaj et al. (1992) pose this as a theoretical question, and do not investigate it further, though it seems consistent with the notion that there is a proportional relationship between pre-test pressure and measured swell pressure.

Sample disturbance

As noted earlier, sample disturbance of soil samples greatly diminishes the meaningfulness of results and should be minimized (ASTM, 2003).When dealing with remolded samples, some of the effects can be mitigated through careful sample preparation (obtaining in-situ density, moisture content etc.) Others, like soil structure, cannot be mitigated. Mitchell (1993), Nelson and Miller (1992) and Seed et al (1962) noted that soils with a flocculent structure swell more than those with a dispersed structure; concluding that soil structure has an affect on swell characteristics. Sampling may also disturb soil samples and alter their swell behavior. El Sayed and Rabbaa (1986) noted that samplers which exert significant pressures on soils may over-compact them, yielding higher observed swell pressures during testing. The ASTM does not recommend the free swell method for finding swell pressure because sorption of water under no restraint may disturb the soil structure and give erroneous results (ASTM, 2003).

Stress path during testing

El Sayed and Rabbaa (1986) and Fredlund and Rahardjo (1993) have noted that the stress path followed by a sample during testing may have an effect on the test results. As a result all attempts should be made to follow a similar stress path to that expected in the field. In order to accomplish this, collected samples must be approximately oriented with respect to the vertical axis. For example, the *swell-consolidation* test allows the sample to swell under little confining pressure, and then loads it to the initial zero strain condition. In the field, the sample will imbibe water under the in-situ confining pressure, and may therefore exhibit different swelling characteristics.

For one-dimensional oedometer tests the constant volume method generally provides the best results (Trisot and Aboushook, 1983; Nelson and Miller, 1992). Some possible reasons for this are that the stress path followed during the test are similar to those expected in the field, and that the zero strain condition minimizes the affects of side friction. For retaining wall design though, the constant volume method provides no indication of the relationship between swelling pressure and strain, which is required information for designing EARW in expansive soils.

Besides on a theoretical basis, the usefulness of comparing oedometer swell test methods to each other is of questionable value. With one dimensional swell pressure testing, given the inconsistent findings of numerous researchers, concluding that one method is "better" or "worse" than the others based on comparison with the other two is dubious at best; and certainly statistically invalid. Without a baseline data set, like one compiled from direct field monitoring, it is impossible to tell which method is the "best".

Standard one dimensional oedometer tests are not capable of quantifying lateral pressures, but lateral pressure measurements have been carried out using modified oedometers as a substitute for running triaxial tests (e.g. Komornik and Zeitlin, 1965, 1973; Edil and Alanazy, 1992; El-Ala-Habib, 1995). Komornik and Zeitlin (1965) attached strain gauges to an oedometer ring. They did this because triaxial apparatus are elaborate, require constant attention and control, require special procedures and are thus limited in the number of samples that can be tested.

Triaxial Swell Tests

The triaxial testing apparatus provides more test flexibility and a greater capacity to mimic in-situ conditions during testing. Where lateral pressures could affect a retaining structure, several authors feel it is necessary to carry out two- or three-dimensional studies of swelling and swell pressures. These represent some of the most complex and analytically intractable problems in geotechnical engineering (Wallace and Lytton, 1992). Gromko (1974) postulated that the large disagreement between laboratory and field results is due to the failure and inability to exactly duplicate field conditions in the lab. For the estimation of lateral swell pressures, triaxial cells have the unique ability to hold the sample under a constant vertical stress (like field conditions) and measure lateral swell pressures, without significant friction between the apparatus and sample. Triaxial devices theoretically provide more accurate swell potentials for EARW design because the confining stress (parallel to the direction of consequence) can be approximated and swell can be measured in the desired direction (normal to the back of the wall).

Triaxial swell tests, like oedometer tests, utilize *constant pressure*, *constant volume*, or *swell-consolidation* test procedures. Trisot and Aboushook (1983) demonstrated that similar results are obtained by any of the methods; largely attributed to the absence of rigid lateral confinement and skin friction. Fourie (1989), Al-Shamarani and Dhowwian (2003), and Shanker et al (1987) all indicate that the multi-dimensional testing approach is better than the one-dimensional approach.

As with any laboratory testing, samples are typically remolded or disturbed either during sampling or preparation. This no doubt leads to some differences between swell behavior in the lab and that found in the field. The lack of skin friction and lateral confinement result in triaxial test conditions that more accurately reflect in-situ conditions.

A triaxial method was described by Fourie (1989), yet it has not found widespread use. His laboratory method utilizes a hydraulic triaxial apparatus which was originally described by Bishop and Wesley (1975). The device allows stress-controlled testing of both lateral and axial pressures. The advantage of the method is its capability to measure lateral pressure under prescribed values of finite lateral strain which is information directly applicable to retaining wall design. The primary disadvantage of the method is the time required: the method requires six tests to be carried out under different initial pressures which, with one device operating, take an average of four to six weeks to run. In addition, triaxial devices are still primarily used as research tools, and are not ubiquitous to soil mechanics laboratories.

Al-Shamarani and Dhowwian (2003) directly compared the results of tests run in both triaxial and oedometer devices (see Figure 8). They showed that swell values obtained in triaxial devices are considerably lower than those measured in oedometers (which typically overestimate swell pressure). According to Khaddaj et al. (1992), triaxial devices allow better control of environmental conditions, enabling control of the stress-strain path to mimic that expected in the field.

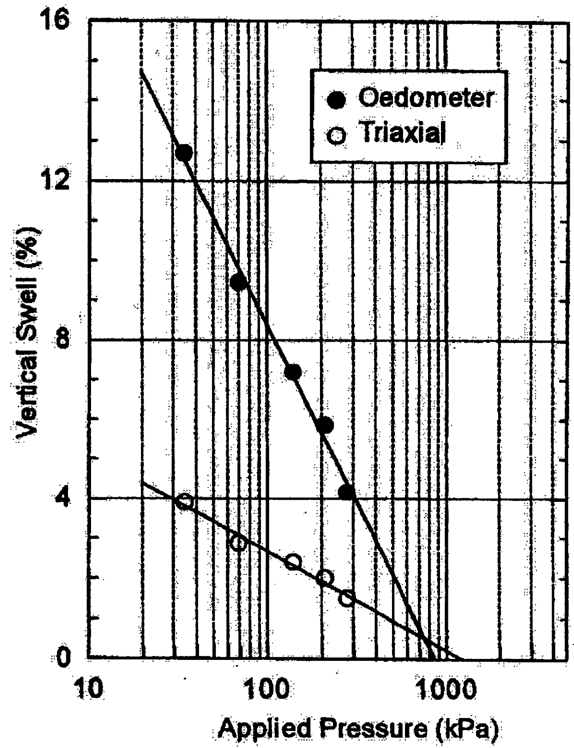

Figure 8: Differences between triaxial and oedometer swell test results for similar samples (Al-Shamrani and Dhowian, 2003).

Sabbagh (2000) demonstrated that measurement of lateral pressures is required to obtain an indication of the complete stress state within the soil. This is because, due to restricted lateral movement, horizontal pressure could be greater than vertical pressure. The horizontal stress would therefore be the major principle stress (in the maximum stress direction) and the vertical stress would be the minor (perpendicular to the maximum stress).

Trisot and Aboushook (1983) also advocate the use of triaxial devices because they allow the tester to trace the stress path. They recommend the constant pressure method because it provides swelling percentage at a given set of stresses. Preswelling (*swell-consolidation*) they felt, regardless of the device used, provided results that were virtually useless.

Triaxial swell testing for lateral earth pressures unfortunately lacks a standardized method and at present there is little data on the engineering behavior of swelling samples under triaxial conditions (Al-Shamarani and Dhowwian, 2000). Fourie (1989) advocates an equilibrium void-ratios procedure (*constant pressure*), while others describe their own procedures (e.g. Al-Shamarani and Dhowwian, 2000). It is perhaps due to the lack of a standard procedure, the need for specialized equipment, and the complexity of the tests, that triaxial swell tests are seldom used to quantify lateral swelling pressures outside of a research environment. Even the triaxial cells in some soils labs are inadequate because the lateral strain vs. swell pressure measurement requires a cell with the ability to measure lateral displacement, not just volumetric changes.

Recommendations for Laboratory Testing of Expansive Soils

In an ideal situation a triaxial device would be employed to test undisturbed samples from a variety of elevations behind the wall. The evolution of lateral pressure could be measured under various amounts of confining pressure. This however would require multiple triaxial tests that would be accompanied by additional expense. The cost benefit ratio of triaxial testing and wall design is presently not known and beyond the scope of this work. At present triaxial tests are not typically carried out for retaining wall design and it is not felt that their application here is necessary.

Though more accurate than oedometer tests, triaxial methods are generally more complicated, time consuming and expensive. Constant pressure oedometer tests are therefore considered sufficient to characterize a soils swell behavior, given their lower cost and greater availability. The tests are standardized, their deficiencies are well studied, and they are readily available and comparatively inexpensive. In addition, the values they provide are generally overestimates and therefore conservative (see Figure 8).

The swell consolidation method and the constant pressure method give displacements for various swelling pressures. The stress path followed by the swell consolidation method is significantly different from that in the field and its applicability is somewhat questionable.

The constant pressure method, for one dimensional tests, seems to provide the most accurate information regarding maximum swelling pressure and the relationship between swelling pressure and displacement. Its use therefore, is recommended for retaining wall design in expansive soils. The main drawbacks are that this is a one-dimensional test, with unknown influence from the apparatus (in addition to all of the other criticisms). Advantages are standardized methodology, more widespread availability, relative cost and time required.

Sample preparation is also important. Sample disturbance should be minimized for more accurate testing results (ASTM, 2003). Thin walled samplers should therefore be used (e.g. piston tube, pitcher tube or Denison corer). In this way, the appropriate relationship between the soil fabric and stress regime can be analyzed. Remolding, as discussed earlier, changes the swell behavior and therefore reduces the correlation between the testing program and in-situ conditions.

Large Scale Tests

Large scale tests on lateral soil swell were conducted by Katti and Katti (1994). The shear size and complexity of large scale testing makes it impractical as a commonly employed test technique, but large scale tests have provided valuable information regarding in-situ behavior behind retaining walls. For the soils examined they have provided swell pressure, density, moisture content and void ratio as functions of depth (discussed throughout this text). Unfortunately, the application of those results to localities with different soils and environmental conditions is considered dubious.

Perhaps when more large scale test data is available, conclusions can be made that are applicable to all expansive soils.

Monitoring of structures in expansive soils is also used as a research tool for investigating the in-situ behavior of swelling soils. Richards and Kurzeme (1973) made instrumented observations of a retaining wall built in expansive soils. However, the contribution of swelling pressure to wall deflection was not distinguishable from those caused by settlement, backfill compaction, adjacent construction activity, and re-equilibration of the water table.

A controlled experiment monitoring a full scale retaining wall in expansive soils could provide important information on the affects swelling soils have on retaining structures. Due to inherent complexity efforts to limit outside influences would be required.

Predictive Equations

Predictive equations for swelling pressure are frequently developed for examined soil units. The broad use of these equations is somewhat problematic because their applicability outside of the regions for which they were developed is questionable. Rao and Smart (1980) developed equations to predict swell pressure and swell potential based on particle size similarity (accurate within 10%). They caution that the relation is based on testing of a specific geologic unit, and is probably not applicable elsewhere. Katti and Katti (1994), Bandypadhaya (1981) and El Sohby and Rabbaa (1986) have all undertaken similar work. They all established correlation formulas for specific soils, which, while reasonably accurate for the unit examined, are probably not universally applicable. For example, Katti and Katti (1994) correlated swelling pressure to undrained shear strength (S_u) and found that, for the soils examined, swelling pressure is equal to 2.6 x S_u.

Nelson and Miller (1992) note that qualitative predictions may be successfully utilized in design only if they were developed for the specific area under study. If not, then their application is of little value and possibly misleading (Al-Shamarani and Dohwian, 2003).

The calibration of empirical and semi-empirical formulas is also of concern. Typically they are calibrated against oedometer tests, which as mentioned above, often provide misleading results. It is therefore necessary, prior to the use of any correlation formula, to research its applicability and development.

DESIGN METHODOLOGY

The following discussion outlines the proposed design methodology. The logic behind each step and progression between steps is described in detail to facilitate its use and remove any "black box" concerns. Unfortunately, despite the relatively simple steps, the process is cumbersome. In the future a computer program could be written to do the post-processing work and iterations, thereby simplifying the calculations.

A soil with a maximum swell pressure of 5ksf was chosen to demonstrate the proposed design procedure. The majority of expansive soils exhibit maximum swelling pressures within the range of 1 to 10 ksf, but values approaching 20 ksf are not uncommon. Values in excess of 50 ksf have been reported but are rarely encountered.

The equations below were derived from experimental data presented in Fourie (1989)(Eq. 1) and Al-Samarani and Dhowian (2000b)(Eq. 2). Fourie's data was gathered using a triaxial device following the constant pressures methodology. For a complete description of the method the reader is referred to Fourie (1989). The equation relating swell pressure to lateral strain (%) was obtained by fitting a trendline to the swell pressure vs. strain data presented in Fourie (1989) and then scaling it to reflect a maximum swelling pressure of 5ksf. In general the type of trendline equation chosen should intuitively result in the maximum R^2 value (at a reasonable complexity). For the lateral strain percentage vs. cell pressure data presented in Fourie (1989), it is an exponential decay function.

$$P_s=P_{smax}*C_1^{\varepsilon} \qquad \text{Eq. 1}$$

where...

P_s= swelling pressure

P_{smax}= maximum swelling pressure

C_x= a constant

ε = strain

It should be noted however that, for the swell consolidation data presented in Al-Shamarani and Dhowian (2000b) the trendline equation with the highest R^2 is a polynomial.

$$P_s=C_2*\delta^2-C_3*\delta+P_{smax} \qquad \text{Eq. 2}$$

where...

P_s= swelling pressure

P_{smax}= maximum swelling pressure

C_x= constants

δ= displacement [L]

The difference in the predictive equations is attributed to the different test methods utilized, a dependency that is not encountered in the field. Further research is required to delineate the form of the actual relationship between swell pressure and displacement. At present, the best trendline equation (high R^2 value and reasonable behavior between data points, i.e. smooth curve) should be utilized.

One of the more intractable issues for retaining wall analysis in expansive soils is estimating the volume of soil that is expected to undergo a significant moisture change. Natural variability within the soil, secondary permeability, and local gradient all affect the movement of moisture through a soil and lead to a non-uniform moisture distribution. In addition, transient conditions are likely due to seasonal moisture variation and provisions for drainage. In the worst case, all of the soil behind the wall goes from a dry state to being saturated. This is the scenario that should be anticipated during wall design because it is the most conservative. In addition, accurate estimation of the magnitude of potential moisture change is dependent on many variables (time, permeability, nature of source etc.) and is difficult to estimate.

Critical Width

The relationship between swelling pressure and lateral strain for the retained soil is derived from the laboratory test data and then modified for application to retaining wall design (so lateral strain can be reasonably correlated to deflection). The modification is necessary because it is not realistic to apply lateral strain vs. swell pressure data to increasing thicknesses of soil because of passive resistance to deflection within the soil mass. As such lateral strain data is applied to the wall design as deflection using the concept of "critical width." The critical width represents a fixed retained soil width from which the displacement is calculated using the laboratory strain data vs. swell pressure data. It exhibits identical behavior at the wall face as does an almost infinite retained soil width where passive pressure is accounted for.

The purpose of this section is to present the steps taken to derive the swell pressure vs. displacement equations used in the finite element analysis. They are presented symbolically, and can be followed for analysis of any expansive soil. The following steps are taken.

1. Determine the maximum deflection the soldier beam would experience under the maximum swell pressure.
2. Determine the elastic modulus of the retained soil.
3. Determine the equivalent retained soil width needed to result in the maximum beam deflection (h_O). This step is taken to avoid a composite material analysis. h_o is defined as the width of soil needed to result in the same moment of inertia as the soldier beam.

4. Derive the equation for the swell pressure experienced by the wall (w_{wall}) as a function of retained soil width (h). Intuitively, each successive increment of retained soil width away from the wall exerts less swell pressure on the wall during swelling. This is because the passive pressure of soil increments closer to the wall resists swell pressure from increments further from the wall.
5. Integrate to determine the area under the function $w_{wall}=f(h)$ and then calculate the "critical width" (h_c). The "critical width" is the uniform thickness of soil (measured on the x-axis of Figures 9) that results in the same area under the curve (in ksf/ft) as the function $w_{wall}=f(h)$.
6. Convert lateral strain percent, in the laboratory derived swell pressure as a function of lateral strain percent data, to displacement using h_c as the retained soil width. The final design analysis is completed using the swell pressure vs. displacement equation.

A staged examination of these steps follows.

STEP 1: Soldier beam deflection

The maximum deflection, δ_{max}, of a cantilevered beam with a distributed load (unit width analysis) is given by Eq. 3.

$$\delta_{max} := \frac{w_o \cdot H^4}{8 \cdot E \cdot I}$$

where
w_o = distributed load (swelling pressur
H= height of the wall above subgrade (Eq. 3)
E= Young's modulus of the beam
I= Moment of inertia of the beam

To convert to a unit width configuration (from full width) the swell pressure was converted into a distributed load. This was accomplished by multiplying the swell pressure (w_o) by the soldier beam area and then recalculating the swell pressure for a unit width configuration (w_u) (or as a distributed load)(Eq. 4). For the initial analysis the width of the wall (B) was considered to be the diameter of the drill hole (~30") plus 6" for conservatism (Eq. 5).

$$w_o \cdot H \cdot B := w_u \cdot H \cdot (1ft) \qquad \text{Eq. 4}$$

where

$$B := b + 6in \qquad \text{Eq. 5}$$

b = diameter of the drill hole into which the soldier beam was installed (filled with lean mix concrete)

STEP 2: Elastic modulus

To get the deflection, Eq. 3 was utilized ($w=w_u$). That deflection was then used to determine the equivalent retained soil width (h_o) required to result in the same magnitude deflection. This essentially replaces the beam with a soil mass that exhibits an equivalent elastic response to the swelling load. To accomplish this, the Young's modulus for the retained clay soil (E_s) was estimated by averaging values presented in the literature from a variety of sources (Table 2).

Table 2: Typical elastic moduli of clay soils

Soil Type	Elastic Modulus (ksi)	Reference
Soft Clay	0.6 - 2.9	Das (1994)
Medium Clay	2.9 - 5.8	"
Hard Clay	5.8 - 14.5	"
Very Soft Clay	0.1 - 0.7	USACE (1990)
Soft Clay	0.7 - 2.8	"
Medium Clay	2.8 - 6.9	"
Hard Clay	6.9 - 13.8	"
Clay Shale	13.8 - 27.8	"

For typical studies it is more appropriate to directly measure the elastic modulus from the results of field tests or laboratory tests on undisturbed samples. It is also can be estimated from empirical correlations. For further discussion of elastic parameters for soil the reader is referred to USACE (1990).

STEP 3: Determination of h_o

The equivalent retained soil width was calculated by setting deflections (Eq. 3, for soil and for soldier beams) equal to each other, Eq. 6, and solving for h_o, Eq. 8. The moment of inertia for the soil mass was estimated according to (Eq. 7) which is the moment of inertia for a volume with a rectangular cross-section.

$$\frac{w_u \cdot H^4}{8 \cdot E \cdot I} := \frac{w_o \cdot H^4}{8 . E_s \cdot I_s} \qquad \text{Eq. 6}$$

where

$$I_s := \frac{1}{3} \cdot \left(b_{u.} \cdot h_o^{\,3} \right) \qquad b_u = 1 \text{ ft.} \qquad \text{Eq. 7}$$

Solving Eq. 6 for h_o...

$$h_o = \frac{\left(3 \cdot E \cdot I \cdot w_o\right)^{\frac{1}{3}}}{\left(E_s \cdot w_u\right)^{\frac{1}{3}}} \qquad \text{Eq. 8}$$

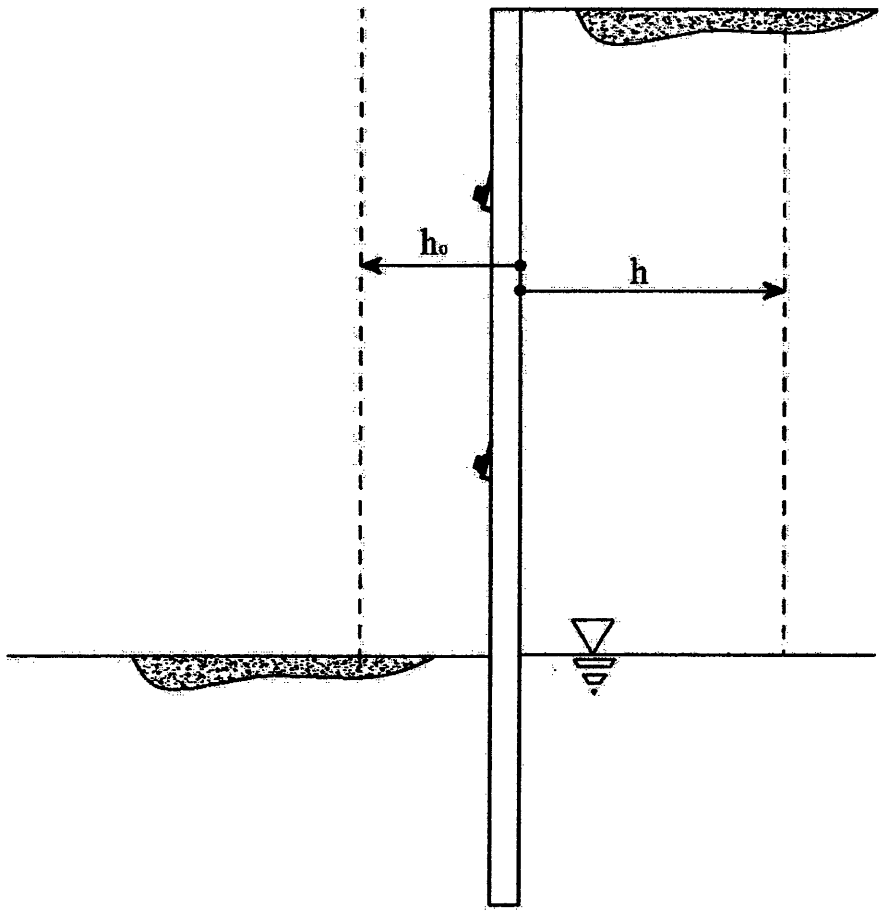

Figure 9: Soil width parameters ho and h. h is distance behind the wall. ho is the width of soil required to result in the same elastic properties as the soldier beam.

STEP 4: Swell Pressure as a function of retained soil width

Once the value of h_o (width of a soil column with the same elastic properties as the soldier beam) is calculated, the swelling pressure exerted on the wall by soil at any distance (h) behind the wall is determined. The deflection due to soil swell at specific distances behind the wall (h) is given by Eq. 9, a combination of Eq. 3 and Eq. 7.

$$\delta := \frac{3 \cdot w_o \cdot H^4}{8 \cdot E_s \cdot b_u \cdot (h_o + h)^3} \qquad h = \text{distance behind the wall} \qquad \text{Eq. 9}$$

The swelling pressure experienced by, or exerted on the wall (w_{wall}) by soil at any specific value of h (distance behind the wall) is then given by Eq.10 which is the swell pressure required to obtain the deflection calculated in Eq. 9 without the additional soil width h.

$$w_{wall} := \delta \frac{8 \cdot b_u \cdot h_o^3 \cdot E_s}{3 \cdot H^4} \qquad \text{Eq. 10}$$

Combining Eq. 9 with Eq. 10 and simplifying yields Eq. 11 which is the general form of the equation relating swelling pressure exerted on the wall by soil at any specific distance behind the wall.

$$w_{wall} = \frac{8 \cdot b_u \cdot h_o^3 \cdot E_s}{3 \cdot H^4} \cdot \frac{3 \cdot w_o \cdot H^4}{8 \cdot E_s \cdot b_u \cdot (h_o + h)^3} \rightarrow h_o^3 \cdot \frac{w_o}{(h_o + h)^3} \qquad \text{Eq. 11}$$

Two plots are presented to illustrate the graphical form of this equation. Figure 10 is a plot of wall vs. retained soil width for a 20 ksf and 1 ksf maximum swelling pressure. Figure 11 is a plot of w_{wall} as a fraction of maximum swelling pressure vs. retained soil width. This is the general form of the curve.

STEP 5: Integration to find critical retained soil width

The purpose of integrating Eq. 11 is to determine the area under the curve to delineate the critical width. The first step is to determine integration limits. For this analysis the limits were calculated as the retained soil width corresponding to the maximum swelling pressure (h = 0 ft.) and a width corresponding to a negligible pressure value that was arbitrarily chosen as 1psf. Using Eq. 12 the retained soil width corresponding to 1psf swelling pressure exerted on the wall were determined. This value is the upper integration limit (h_u).

$$h_u := \frac{h_o \cdot w_o^{\frac{1}{3}}}{w_{wall}^{\frac{1}{3}}} - h_o \qquad \text{where } w_{wall} = 1 \text{ psf} \qquad \text{Eq. 12}$$

Eq. 11 is then integrated, Eq. 13, to determine the total area under the curve between the integration limits (0 ft., h_u ft.).

$$\text{Area} = \int_0^{h_u} \left[w_o \cdot \left[\frac{h_o^3}{(h_o + h)^3} \right] \right] dh \qquad \text{Eq. 13}$$

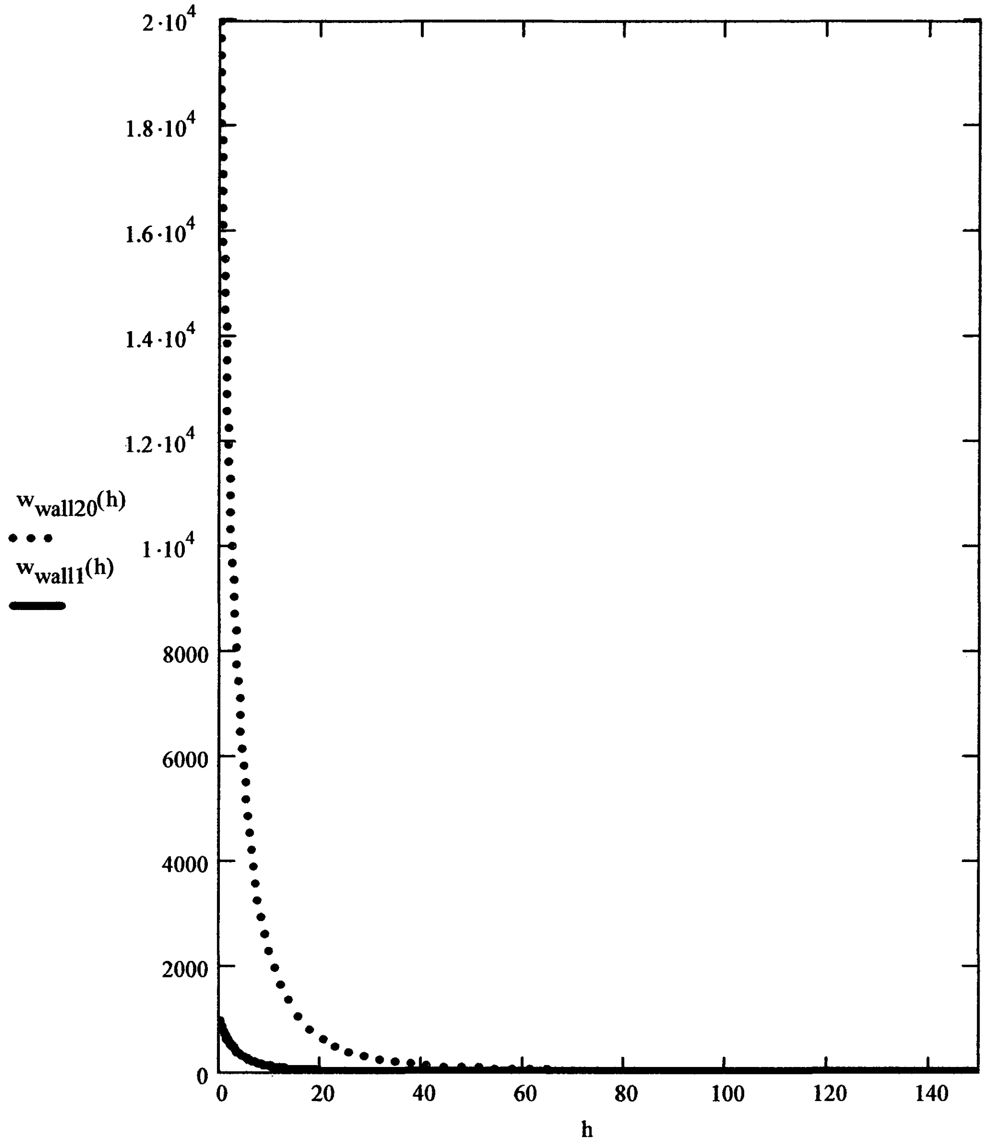

Figure 10- Plot of w_{wall} vs. retained soil width for maximum swelling pressures of 1 and 20 ksf. Retained soil width is measured in feet.

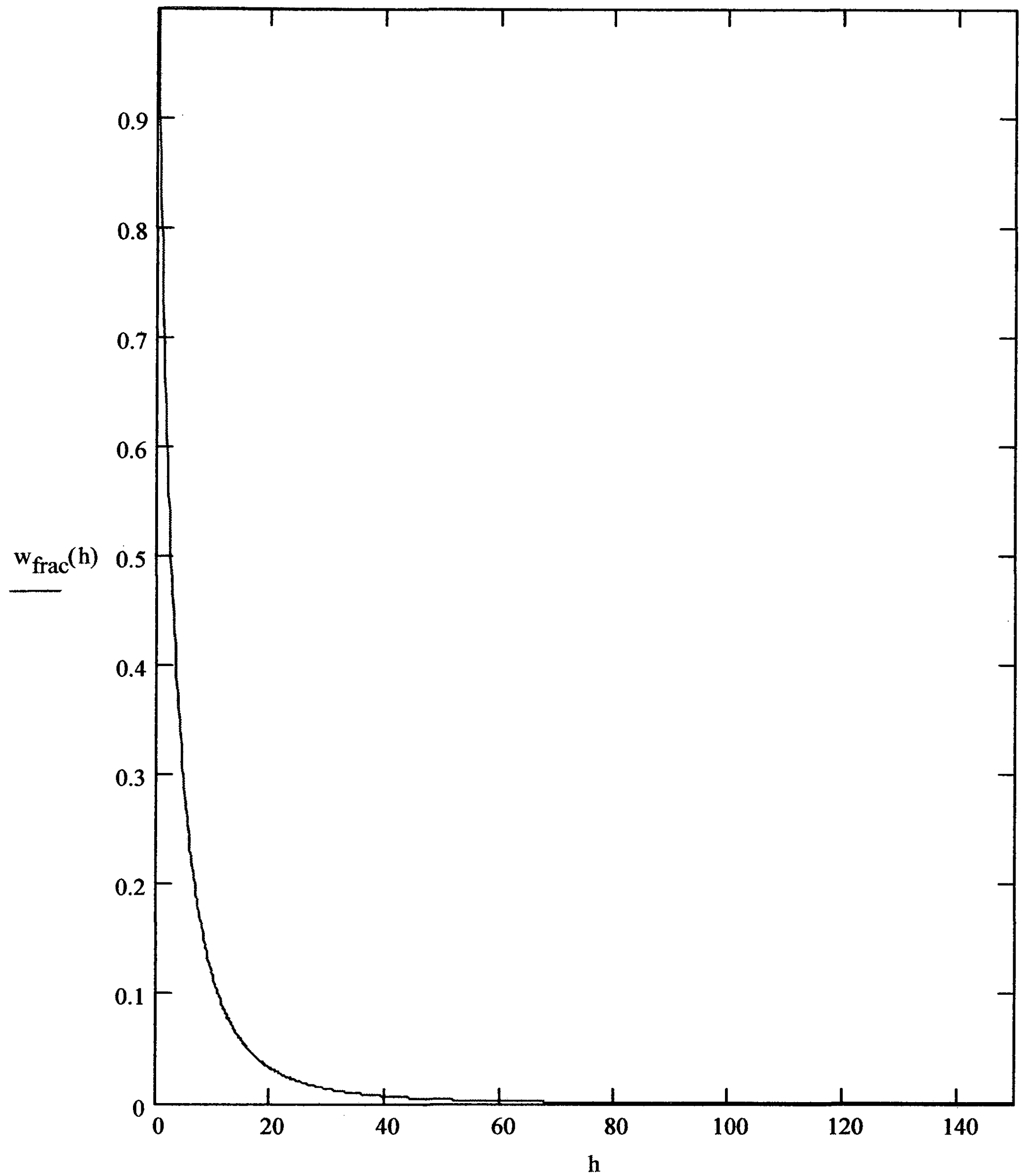

Figure 11- Plot of w_{wall}/w_o vs. retained soil width. Retained soil width is measured in feet.

The critical retained soil width (h_c) is taken as the width that, when multiplied by the maximum swelling pressure analyzed (w_o), results in the same area as was calculated with Eq. 13. It is given by Eq. 14.

$$h_c := \frac{\text{Area}}{w_o} \qquad \text{Eq. 14}$$

STEP 6: Incorporation of swell pressure vs. deflection

The final design analysis is conducted by using the swell pressure vs. lateral strain data obtained through laboratory tests. The lateral strains are converted to actual wall scale displacement by multiplying the lateral strain percentages by h_c ("critical width"). A best fit line is then fit to the data yielding an equation of swell pressure as a function of displacement. This equation then provides to basis for the finite element analysis; an example of such an equation and its use follows.

Use of the Finite Element Method

After obtaining the appropriate trendline equation, relating swell pressure to displacement for the retained soil, the initial analysis is conducted using the finite element method. The FEM mesh (Figure 12) consists of structural elements and soil elements onto which loads are applied. Soil elements are placed in front of the wall below subgrade, following the geometry of a passive soil wedge to approximate the subgrade reaction. Young's Modulus for the soil is estimated using ranges provided in Table 2. No other soil elements are utilized, the retained soils are modeled using nodal loads.

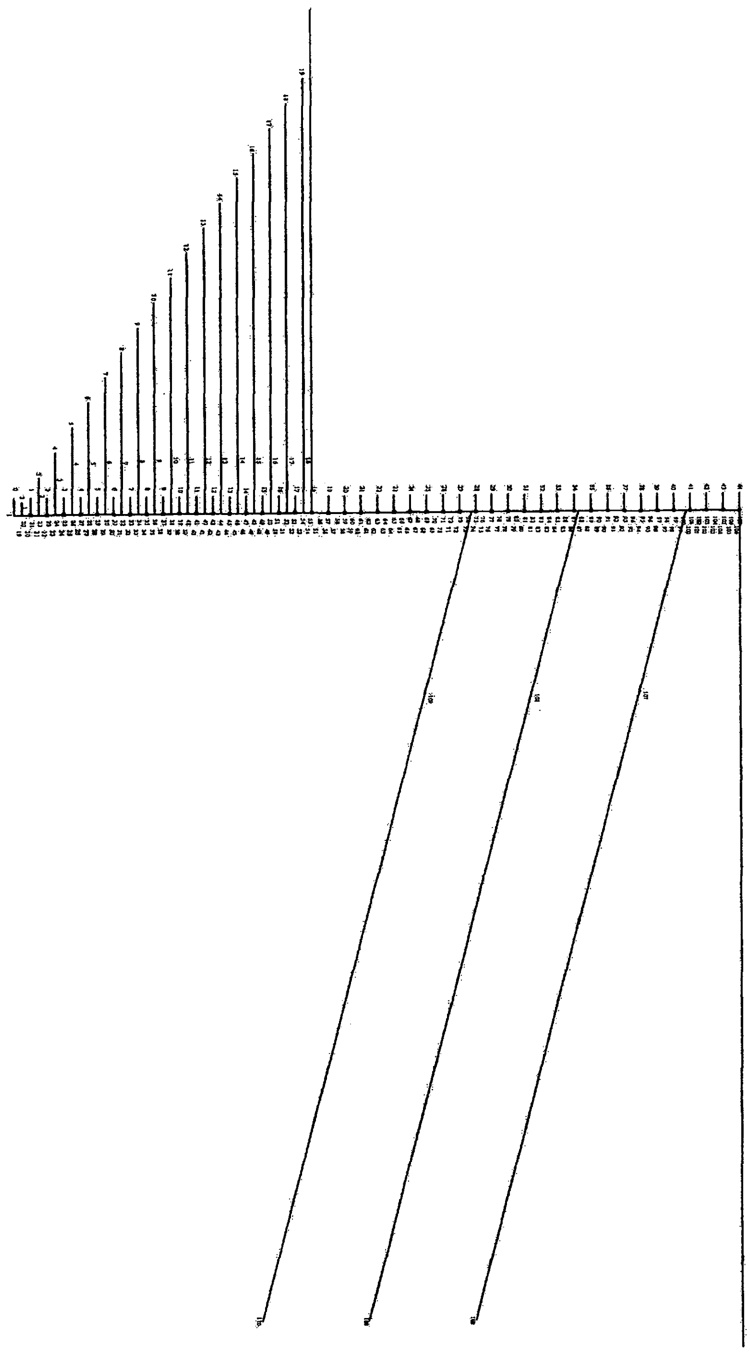

Figure 12: The finite element mesh established for this analysis.

Structural elements are added into the mesh according to the geometry of the wall design. Wall height for the trial analysis was arbitrarily discretized into 6" elements. Though the final design configuration is yet to be determined, an initial estimate is made based on experience. The accuracy of the initial guess does not influence the final design parameters, but decreases the analysis time by limiting the number of design cases checked. Steel strength parameters and geometries are entered into the program as designed. Adjustments are made based on the results of the initial analysis and initial results (it is envisioned that this could be automated). The bonded zone (Figure 13) at this point is modeled as a fixed node at length Lu (it is envisioned that the bonded zone could be directly modeled in the future, but La will be easily determined by the load imposed on the anchors and the soil strength).

Once the mesh is established, a series of iterations yield nodal displacements and moments from soil loads and soil expansion. The soil loads (modeled using a 0.2, 0.6, 0.2H trapezoidal loading diagram, Figure 3) remain fixed throughout the analysis. The initial swelling load imposed is that of the maximum swelling pressure (no displacement). For each subsequent iteration, the resulting displacement is used to recalculate the swelling load at each node (from the trendline equation). The new nodal loadings are then re-entered for the next iteration. The iterations are ideally run until stable. (For the trial analysis the new loadings were entered by hand, after twenty iterations the final result was extrapolated.)

The output data from the finite element analysis yields moments at each node (after post processing). From the maximum nodal moment, the required section is calculated and appropriate beams are chosen. The analysis is then re-run with the new structural properties to ensure it is an adequate section. In general, the more rigid the beam, the greater the proportion of the maximum swelling load is resisted (less deflection).

Similarly, tieback loads are calculated using the deflection outputs of the finite element program. Because the elastic and geometric properties of the tiebacks are known, deflection at the nodes corresponding to tieback locations can be used to calculate the load imposed by swelling pressures (d=PL/AE). As with the soldier beams, tiebacks can then be appropriately sized for adequate section and the analysis re-run for conformation. The remainder of the wall design (components etc.) can then be calculated using established procedures.

DESIGN EXAMPLE

The proposed design algorithm for a specific design case was run as an analysis example. The wall geometry, component sizing and soil parameters were as follows, a diagram of the wall is presented in Figure 13:

<u>Geometry</u>

H = 26 ft. (H1 = 3.25 ft., H2 = 6.5 ft., H3 = 6.5 ft.)

Spacing = 10 ft. (space between soldier beams)

$\alpha_{1,2,3}$ = 15° (tieback angle from horizontal)

Lu = 50 ft. (tieback unbonded length)

<u>Soldier Beam</u>

2-W14 x 61

<u>Tiebacks</u>

T1- 4-0.6"*

T2- 4-0.6"

T3- 4-0.6"

* denotes that each tieback consists of four strands that are 0.6" in diameter and made of 270ksi steel.

<u>Soil Properties</u>

γ = 115 pcf

φ = 22°

<u>Swell pressure vs. displacement equation</u>

$P_{swell} = 5*e^{-22.42*\delta}$ (δ in feet) (Eq. 16)

$P_{swellmax}$ = 5 ksf

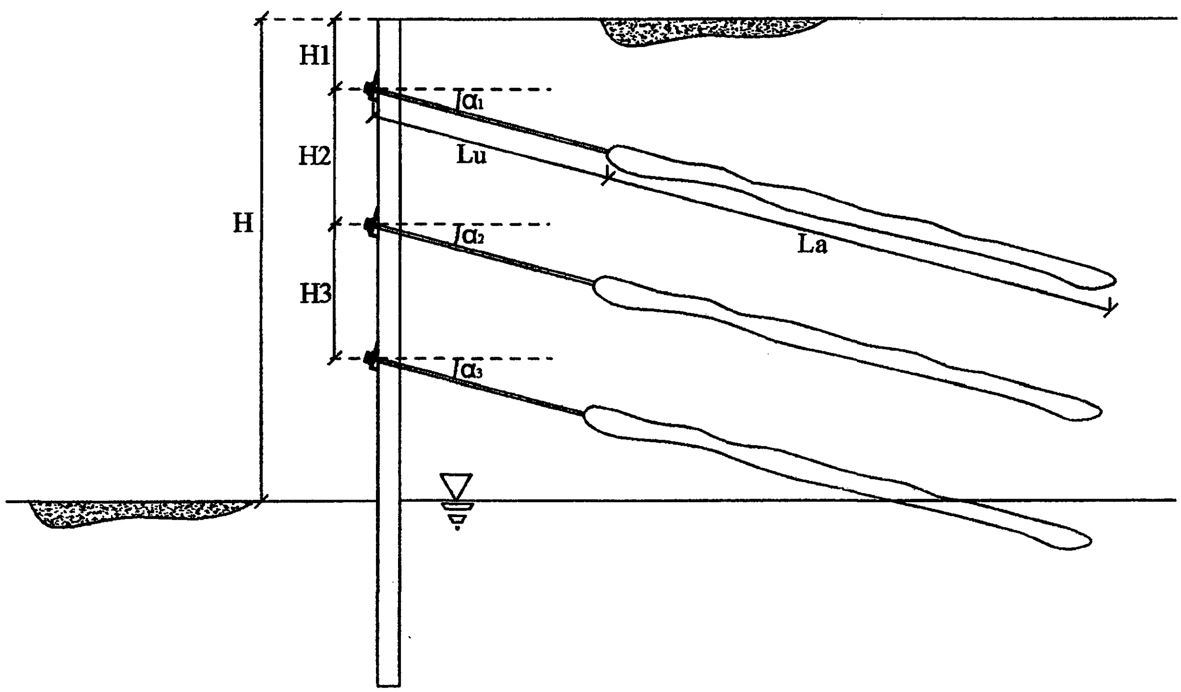

Figure 13: Diagram of three tier earth anchored retaining wall.

The above parameters characterize the final design configuration run for the analysis. Obviously tieback anchors of that capacity and a soldier beam with that section modulus are overly conservative for a 26 foot high wall, if it were in non-expansive soils. The choice of those sections was from initial and partial runs of the finite element analysis where it was possible to predict early in the analysis if the sections would be adequate.

The finite element analysis was run through a series of iterations. The initial run was conducted at the maximum swelling pressure (the zero displacement condition), providing an overestimate of displacements. For the second iteration, those displacements were used to update the swell pressure at each node (using the modified swell pressure vs. displacement equation whose derivation was discussed earlier). Because the displacements from the initial analysis were overestimates, the swell pressures calculated from them are underestimates, and the resulting displacements are also underestimates. The third iteration therefore starts with overestimated swell pressure and results in overestimated displacements. The fourth provides and underestimate of displacement, the fifth an overestimate and so on, until each node converges. It should be noted that each run took approximately 8 hours of operator time (for only 20 iterations). When extrapolated to the final configuration (see Figure 14) it is estimated that upwards of 3000 iterations would be required (for +/-0.1%). The final configuration for this analysis was extrapolated by estimating the convergence from each side (overestimates and underestimates) with best fit equations, and finding their intercept. The validity of this extrapolation procedure is questionable and was based on minimum R^2 values approaching 92%.

The swell pressure vs. displacement equation used was based on laboratory data from triaxial testing of an expansive soil using the constant pressure method by Fourie (1989). A trendline was fit to his modified data (fraction of maximum swell pressure vs. displacement from h_c) and then modified to reflect a maximum swelling pressure of 5 ksf.

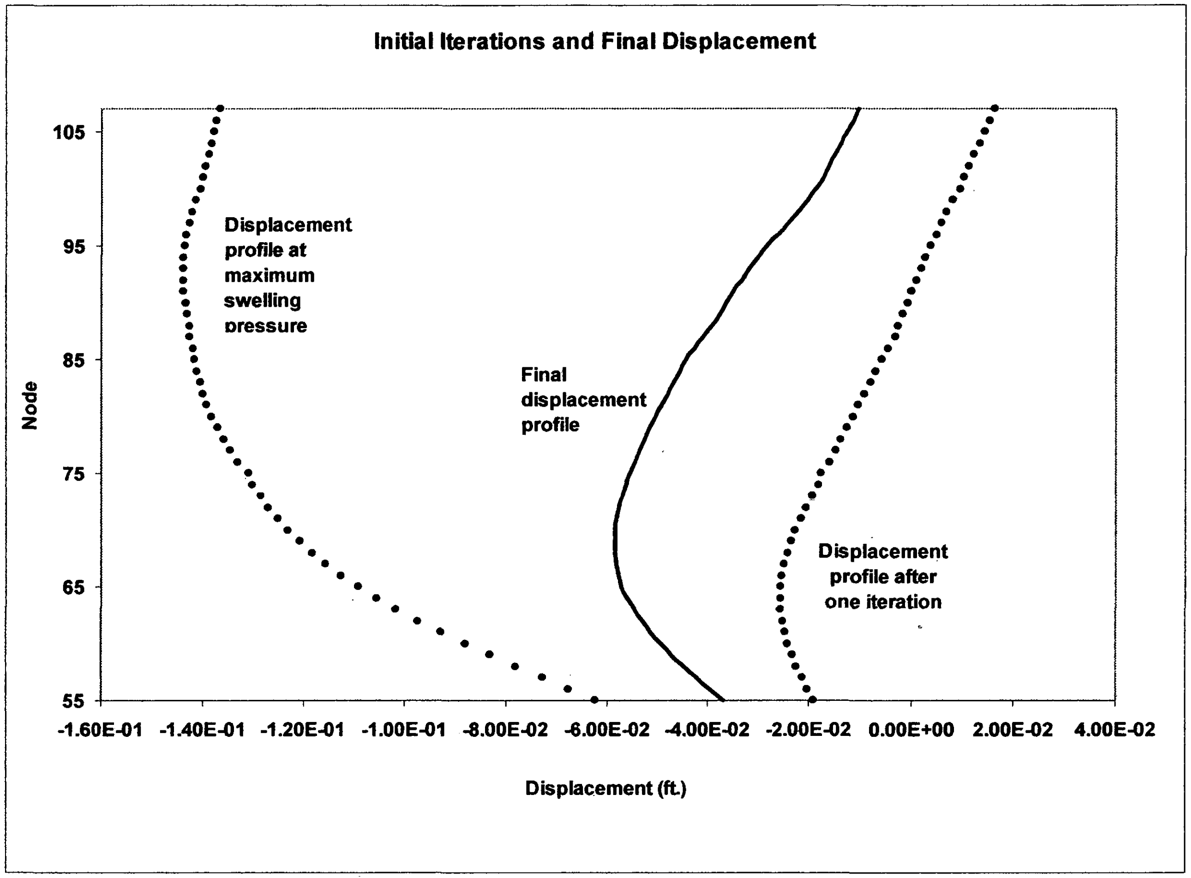

Figure 14: Node vs. Displacement at three stages during the FEM analysis (max pressure, at one iteration and final configuration). Nodes 106 and 55 represent the original ground surface and sub-grade respectively.

For the data presented in Fourie (1989) the equation was calculated to be (R^2 = 98.5%)...

$$P_{swell} = 5*e^{-22.42*\delta} \quad (\delta \text{ in feet}) \qquad \text{Eq. 16}$$

The general form of the equation is...

$$P_{swell} = C_1 * e^{C_2 * \delta} \quad (\delta \text{ in feet}) \qquad \text{Eq. 17}$$

where

P_{swell} = swell pressure

δ = displacement (ft.)

C_1 = the maximum swelling pressure

C_2 = a best fit constant

Obviously each soil will have different values for C_1 and C_2. As noted earlier the form of the best fit equation may also be different depending on the nature of the laboratory test data (it may be a polynomial for example).

It should be noted that the relationship between swell pressure and displacement used in the design calculation reflects the behavior of this soil only. Extrapolation to all expansive soils cannot be reasonably accomplished until some understanding exists as to the natural variability of this relationship (not only from soil to soil, but within a single soil mass). If it turns out that the attenuation of swell pressure with displacement is strongly correlated with the maximum swelling pressure then it may be possible to standardize the relationship based on maximum swelling pressure. At this point however, the relationship between swell pressure and displacement must be obtained for each soil analyzed.

The results of the analysis are presented in Table 3. An analysis was run for zero swelling pressure, the maximum swelling pressure with no attenuation with displacement, and for swell pressure attenuated with displacement.

Table 3: Results of design calculations for no swell pressure, attenuated swell pressure with displacement, and maximum swell pressure (DLT terms represent respective tieback loads).

	M_{max}	S_{req}
No swell pressure	**6.3 kips*ft**	**22.7 in^3**
Calculated	**38.9**	**139.7**
Max	**46.7**	**184.7**

There is a ~7.4x increase in the maximum soldier beam moment when examining a wall designed for maximum swelling pressure versus a wall designed for no swell pressure. When the swelling pressure is attenuated with displacement the increase is ~6.1x. That means that, for this soil and wall configuration, including the attenuation of swell pressure with displacement decreases the forces and moments acting on the wall by ~17%.

CONSTRUCTION RECOMMENDATIONS

For this analysis, the width over which the swell pressure was applied was 3.5 feet. This accounts for full swell pressure over the width of soldier beam drilling (30") plus 6 inches for conservatism. The remaining span can be designed to accommodate large displacements in response to swelling pressure with little transfer of force to the wall. This could be accomplished by lagging the inner flange of the back of the beam with a relatively flexible material (wood), then lagging the front of the beam with regular lagging, leaving a void in between (~8 inches typ.). The actual effectiveness of this mitigation scheme needs to be tested in the field for proper assessment.

In addition, other existing methods to mitigating soil expansion could be employed. They include soil treatment and pre-wetting. A full discussion of treatment procedures is considered beyond the scope of this work but there is a voluminous amount of literature available on the subject.

The analysis presented in this paper assumed that all of the retained soil was both expansive and dry. In reality though, the actual area over which swelling may be problematic, and deserving of the proposed analysis procedure, may be much smaller than the full retained soil height. Retained layers that will not exert additional pressures due to soil swell are saturated soils, and non-expansive materials. Layers that will exert little swell pressure are those expansive soils with relatively high moisture contents, such as those within the capillary fringe.

It is therefore necessary to construct a careful profile of the retained soil to determine the extent of any expansive layers. Once they are identified, each can be assessed in terms of swell capacity and mitigation alternatives can be examined (particularly for shallow depths). The proposed procedure can then be run on only those layers identified as having a significant swell capacity. The pressure that they can exert on the wall will then be a function of their location within the retained soil profile, and relation to the structural elements of the wall.

CONCLUSIONS

Wall flexure is desirable in expansive soils because deflection decreases the amount of swell pressure a soil can exert on a wall. For the soil and wall configuration analyzed, a 17% reduction in forces acting on the wall theoretically occurs due to wall deflection.

The primary conclusion from this work is that the design algorithm presented is a useful tool when designing EARW's in expansive soils. The advantage it has over traditional design algorithms is that it takes into account the attenuation of swelling pressure with wall displacement, resulting in a more efficient design.

When designing in expansive soils the following recommendations are made:

1. Carefully identify retained soils layers that have a significant swell capacity so that pressures on the wall can be realistically modeled and an efficient laboratory testing program can be implemented.
2. Examine swell mitigation options for the soils identified in #1.
3. Run constant pressure oedometer tests on those layers identified in #1 to determine the relationship between swelling pressure and deflection for that particular soil (if mitigation is not feasible).
4. Run the proposed design procedure for those layers identified in #1.
5. Design the wall for deflection. For soldier beam walls, the transfer of swell pressure to the soldier beams from the lagging (the space between beams) can be minimized by creating space for deflection.

SOURCES OF ERROR

There are several sources of error unique to an analysis run in this manner. Besides errors associated with standard design practice (e.g. soil pressure approximation using a trapezoidal loading diagram), the laboratory work and use of the finite element method introduce additional errors.

The most significant source of error is the trendline equation established between soil swell pressure and displacement. At this point the methods used in the laboratory exert significant influence on the result, as previously discussed. In addition, there is little indication in the literature about the scale over which the relationship between swell pressure and displacement is significantly variable. Statisticians frequently balk at the "validity" of geotechnical data but it would be interesting to see how representative one or a few samples are of the greater "population" of soil retained behind a wall. It is not known how much testing and sampling would be required to explain a high percentage of the variability behind the wall.

The use of the finite element method requires simplifying assumptions such as a homogeneous retained soil mass, which is also assumed for standard practice in non-expansive soils. Though thought to be a conservative assumption (variability means that some of the soil is probably not expansive) the true affects of variability in the soil mass are not known.

FEM also requires estimation of the moment of inertia and Young's Modulus for soil. The validity of this estimation is questionable and ultimately affects the outcome of the analysis. At this time the accuracy and significance to the analysis of this assumption are not known.

FURTHER WORK

The usefulness of this design method is presently hampered by post-processing and a simple yet lengthy and repetitive finite element analysis. A computer program written to automatically run the iterations and do the post-processing would save immense time during wall design. In addition, several design operations could be automated, including choosing appropriate tieback and soldier beam sections, toe depth and bonded length (from the tieback load and wall geometry). Additional improvements such as a simple to use Windows interface should also be employed to increase the accessibility of the design method.

A more serious and difficult research pursuit is to obtain and examine a statistically significant data set of swell pressure versus deflection data using a triaxial testing apparatus for a wide variety of swelling soils. The motive for this work is to attempt to delineate any behavior characteristics that are shared by all expansive soils (perhaps from statistical analysis). Is the relationship between swell pressure and deflection the same for all expansive soils? Are there any patterns otherwise that may be used to predict swell behavior? Is it truly required that each soil be tested on an individual basis? How much variation is there within single soil units? Depending on the answer to these questions it may be possible to generalize the relationship between swell pressure and deflection to all expansive soils (within a reasonable range).

For this analysis it was assumed that lateral swelling pressure was not affected by overburden stress. This is no doubt conservative, and somewhat consistent with the findings of Katti and Katti (1994), but intuitively questionable. One would expect lateral swelling pressure to evolve with depth as a function of overburden stress (as previously mentioned). Future work should include a triaxial study to determine how lateral swelling pressure changes with overburden stress. The results could be used to modify the design procedure to more accurately reflect field conditions.

Another worthy pursuit would be to instrument a wall and measure deflections from soil swell. The obtained data however would probably only be useful if the environment were strongly controlled so that deflections could be specifically attributed to specific circumstances. Richards and Kurzeme (1973) instrumented a wall in expansive soils but the contribution of swelling pressure was very difficult to quantify due to changes in surcharge load and a fluctuating water table. The instrumentation may have to be done in a research setting rather than on a job site.

REFERENCES

Al-Shamarani, M.A. and Dhowian, A.W., 2003, *Experimental Study of Lateral Restraint Effects on the Potential Heave of Expansive Soils*, Engineering Geology, Vol. 69, pp 63-81.

Al-Shamrani, M.A. & Al-Mhaidib, A.I., 2000a, *Vertical Swelling of Expansive Soils Under Fully and Partially Lateral Restraint Conditions*, Unsaturated Soils for Asia, Proc. 1st Asian Conf. on Unsaturated Soils (UNSAT-ASIA 2000) Singapore (ed. Rahardjo, H., Toll, D.G. & Leong, E.C.), Rotterdam: Balkema, pp. 627-632.

Al-Shamarani, M.A. and Al-Mhaidib, A.I., 2000b, *Swelling Behavior Under Oedometric and Triaxial Loading Conditions*, Advances In Unsaturated Geotechnics, , Shakelford, C.D., Houston, S.L. and Chang, N.Y, eds Geotechnical Special Publication, Vol. 99, ASCE, pp 344-360.

ASTM, 2003, *Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils D4546-03,* ASTM International, West Conshohocken, Pa., 7p.

Bandyopadhyay, S.S., 1981, *Prediction of Swelling Potential for Natural Soils,* Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. 5, pp 658-661.

Bishop, A.W. and Wesley, L.D., 1975, *A Hydraulic Triaxial Apparatus for Controlled Stress Path Testing*, Geotechnique, Vol. 25, No. 4, pp 657-670.

Chen, F.N., 1988, Foundations on Expansive Soils, Developments in Geotechnical Engineering Vol. 54, Elsevier, New York, N.Y., 463p.

Coduto, D.P., 2001, Foundation Design: Principles and Practices, 2nd Ed., Prentice Hall, Upper Saddle River, N.J., 883p.

Covar, A.P. and Lytton, R.L., 2001, *Estimating Soil Swelling Behavior Using Soil Classification Properties*, in Expansive Clay Soils and Vegetative Influence on Shallow Foundations, Geotechnical Special Publication No. 115 (ed. C.

Vipulanandan, M. B. Addison and M. Hasen), Reston: American Society of Civil Engineers, pp. 44-63.

Dakshanamurthy, V., 1979, A Stress Controlled Study of Swelling Characteristics of Compacted Expansive Clays, Geotechnical Testing Journal, Vol. 2, No. 1, pp 57-60.

Dakshanamurthy, V. and Raman, V., 1973, *A Simple Method For Identifying Expansive Soil,* Soil and Foundations, Vol. 13, No. 1, pp 97-104.

Das, B.M., 1999, Fundamentals of Geotechnical Engineering, Brooks/Cole, Pacific Grove, CA., 593p.

Dhowian, A.W., 1990, *Simpified Heave Prediction Model for Expansive Shale,* Geotechnical Testing Journal, Vol 13, pp 323-333.

Edil, T.B. and Alanazy, A.S., 1992, *Lateral Swelling Pressures*, Proc. 7th Int. Conf. on Expansive Soils, Dallas, Vol. 1, pp. 227-232.

El Sayed, S.T. and Rabbaa, S.A., 1986, *Factors Affecting the Behavior of Expansive Soils in the Laboratory and Field- A Review*, Geotechnical Engineering, Vol. 17, pp 89-107.

El Sohby, M.A., and Rabbaa, S.A., 1981, *Some Factors Affecting Swelling of Clayey Soils,* Geotechnical Engineering, Vol. 12, pp 19-39.

El-Ala-Habib, S.A., 1995, *Lateral Pressures of Unsaturated Expansive Clay in Looped Stress Path*, Unsaturated Soils. Proc. 1st Int. Conf. on Unsaturated Soils (UNSAT 95), Paris, France (ed. Alonzo, E.E. and Delage, P.), Rotterdam: Balkema, Vol. 2, pp. 602-608.

Fityus, S. and Smith, D.W., 1998, *A Simple Model for the Prediction of Free Surface Movements in Swelling Clay Profiles,* Proceedings, 2nd International Conference on Unsaturated Soils, Bejing, China, pp 473-478.

Fourie, A.B., 1989, *Laboratory Evaluation of Lateral Swelling Pressure*, Journal of Geotechnical Engineering, Vol. 115, No. 10, pp 1481-1486.

Garbulewski, K. and Zakowicz, S. (1995) *Suction as an Indicator of Soil Expansive*

Potential, Unsaturated Soils. Proc. 1st Int. Conf. on Unsaturated Soils (UNSAT 95), Paris, France (ed. Alonzo, E.E. and Delage, P.), Rotterdam: Balkema, Vol. 2, pp. 593-599.

Gromko, G.J., 1974, *Review of Expansive Soils,* Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. 6, pp 667-687.

Holtz, W.G., and Gibbs, H.J., 1956, *Engineering Properties of Expansive Clays,* Transactions of the American Society of Civil Engineers, Vol. 121, pp. 641-677.

Johnson, L.D. and Snethen, D.R., 1978, *Prediction of Potential Heave of Swelling Soil,* Journal of the American Society for Testing and Materials, pp 117-124.

Katti, R.K., Katti, A.R., 1994, Behavior of Saturated Expansive Soil and Control Methods, A.A. Balkema Publishing, Rotterdam, Netherlands, 1267p.

Khaddaj, S., Lancelot, L. and Shahrour, I., 1992 *Experimental-Study of the Swelling Behavior of Heavily Overconsolidated Flandres Clays*, Proc. 7th Int. Conf. on Expansive Soils, Dallas, Vol. 1, pp. 239-244.

Komornik, A. and David, D., 1969, *Prediction of Swelling Pressure of Clays*, Journal of the Soil Mechanics and Foundations Division, Procedings of the American Society of Civil Engineers, Vol. 95, No. SM1, pp 209-225.

Komornik, A. and Zeitlen, J.G., 1965, *An Apparatus for Measuring Lateral Soil Swelling Pressure in the Laboratory,* , Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. 1, pp 278-281.

Likos, W.J., 2000, Total Suction-Moisture Content Characteristics for Expansive Soils, Ph.D. Thesis, Colorado School of Mines, 182p.

Lytton, R.L., 1994, *Prediction of Movement in Expansive Clays,* in Vertical and Horizontal Deformations of Foundations and Embankments, A.T. Yeung and G.Y. Felio, eds. ASCE, New York, pp 1827-1845.

Mawire, K. and Senneset, K., 2000, *Analysis of Swelling Pressure for Expansive Soils*

Using the Resistance Concept, in Unsaturated Soils for Asia, Proc. 1st Asian Conf. on Unsaturated Soils (UNSAT-ASIA 2000) Singapore (ed. Rahardjo, H., Toll, D.G. & Leong, E.C.), Rotterdam: Balkema, pp. 699-702.

McKeen, 1981, *Field Studies of Airport Pavements On Expansive Clay,* Proceedings of the 4th International Conference on Expansive Soils, Vol. 1, pp 124-129.

McOmber, R.M. and Thompson, R.W., 2000, *Verification of Depth of Wetting for Potential Heave Calculations.* Advances In Unsaturated Geotechnics, , Shakelford, C.D., Houston, S.L. and Chang, N.Y, eds Geotechnical Special Publication, Vol. 99, Am. Soc. Civil Eng., pp 409-422.

Mitchell, J.K., 1993, Fundamentals of Soil Behavior, John Wiley & Sons, Inc., New York, N.Y., p ???.

NAVFAC, 1986, *Foundations and Earth Structures, Design Manual 7.02*, Naval Facilities Engineering Command, Alexandria, Virginia, 279p.

Nelson, J.D. and Miller, D.J., 1992, Expansive Soils: Problems and Practice in Foundation and Pavement Engineering, John Wiley & Sons, Inc., New York, N.Y., 259p.

Nelson, J.D., Overton, D.D. and Durkee, D.B., 2001, *Depth of Wetting and the Active Zone*, in Expansive Clay Soils and Vegetative Influence on Shallow Foundations, Geotechnical Special Publication No. 115 (ed. C. Vipulanandan, M. B. Addison and M. Hasen), Reston: Am. Soc. Civil Eng., pp. 95-109.

Nusier, O.K., and Alaweneh, A.S., 2002, *Damage of Reinforced Concrete Structure due to Severe Soil Expansion*, Journal of Performance of Constructed Facilities, Vol. 19, No.1, pp 33-41.

Parcher, J.V. and Liu, T.C., 1965, *Some Swelling Characteristics of Compacted Clays,* Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, SM3, Part 1, pp 1-17.

Patrick, D.M. and Snethen, D.R., 1976, *An Occurrence and Distribution Survey of*

Expansive Materials in the United States by Physiographic Areas, FHWA-RD-76-82, United States Department of Transportation, Federal Highway Administration, Washington, D.C., 77p.

Petry, T.M., Sheen, J.-S. and Armstrong, J.C., 1992, *Effects of Pre-test Stress Environments on Swell*, Proceedings of the 7th International Conference on Expansive Soils, pp 39-44.

Porter, A.A. and Nelson, J.D., 1980, *Strain Controlled Testing of Expansive Soils*, Fourth International Conference on Expansive Soils , Denver Co., Vol. 1., pp 34-44.

Rao, R.R. and Smart, P., 1980, *Significance of Particle Size Distribution Similarity in Prediction of Swelling Properties,* Fourth International Conference on Expansive Soils, Vol. 1, Denver Co., pp 97-105.

Richards, B.G. and Kurzeme, M., 1973, *Observations of Earth Pressures on a Retaining Wall at Gouger Street Mail Exchange, Adelaide*, Australian Geomechanics Journal, Vol. 3, No. 1, pp 21-26.

Sabatini, P.J., Pass, D.G. and Bachus, R.C., 1999, *Geotechnical Engineering Circular No.4: Ground Anchors and Anchored Systems, FHWA-IF-99-015*, United States Department of Transportation, Federal Highway Administration, Washington, D.C., 304p.

Sabbagh, A., 2000, *The Lateral Swelling Pressure Role on the Volumetric Behavior of Natural Expansive Soil Deposits*, in Unsaturated Soils for Asia, Proc. 1st Asian Conf. on Unsaturated Soils (UNSAT-ASIA 2000) Singapore (ed. Rahardjo, H., Toll, D.G. & Leong, E.C.), Rotterdam: Balkema, pp. 709-714.

Schnabel, H. and Schnabel, H.W., 2002, Tiebacks in Foundation Engineering and Construction, 2nd Ed., A.A. Balkema Publishers, Lisse, Netherlands, 142p.

Seed, H. B., Woodward, R.J., Jr., and Lundgren, R., 1962, *Prediction Of Swelling Potential For Compacted Clays*, Journal of The American Society of Civil Engineers, Soil Mechanics and Foundations Division, Vol. 88, No. SM3, pp. 53-87.

Shanker, N.B., Ratman, M.V. and Rao, A.S., 1987, *Multidimensional Swell Behavior of Expansive Clays,* Proceedings of the 6th International Conference on Expansive Soils, New Dehli, India, pp 143-147.

Sikh, T.S., 1993, *Swelling Soils,* Journal of Geotechnical Engineering, Vol. 119, No. 4, pp 791-792.

Smith, I.M. and Griffiths, D.V., 1998, Programming the Finite Element Method, John Wiley and Sons, New York, N.Y.

Snethen, D.R., 1984, *Evaluation of Expedient Methods for Identification and Classification of Potentially Expansive Soils, ,* Proceedings of the 5th International Conference on Expansive Soils, pp 12-17.

Snethen, D.R., 1975, *A Review of Engineering Experiences With Expansive Soils In Highway Subgrades, FHWA-RD-75-48,* United States Department of Transportation, Federal Highway Administration, Washington, D.C., 135p.

Snethen, D.R. and Huang, G., 1992, *Evaluation of Soil Suction Heave Prediction Methods,* Proceedings of the 7th International Conference on Expansive Soils, pp 12-17.

Sridharan, A., Rao, A.S. and Sivapullaiah, P.V., 1986, *Swelling Pressure of Clays,* Geotechnical Testing Journal, Vol. 9, No. 1, pp 24-33.

Trisot, J.P. and Aboushook, M., 1983, *Triaxial Study of Swelling Characteristics,* Proceedings of the 7th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Haifa, Vol. 1, pp 94-97.

Terzaghi, K., Peck, R.B. and Mesri, G., 1996, Soil Mechanics in Engineering Practice, 3rd Ed., John Wiley and Sons, Inc. New York, N.Y., 549p.

USACE, 1983a, *Foundations in Expansive Soils, Technical Manual TM 5-818-7,* United States Army Corps of Engineers, Department of the Army, Washington, D.C., 95p.

USACE, 1983b, *Soils and Geology Procedures for Foundation Design of Buildings and*

Other Structures (Except Hydraulic Structures), Technical Manual TM 5-818-1, United States Army Corps of Engineers, Department of the Army and the Air Force, Washington, D.C., 204p.

USACE, 1980, *Swell and Swell Pressure Tests, Appendix VIIA, Engineer Manual EM 1110-2-1906*, United States Army Corps of Engineers, Department of the Army, Washington, D.C., 11p.

USACE, 1990, *Appendix D- Elastic Parameters, Engineer Manual EM 1110-1-1904*, United States Army Corps of Engineers, Department of the Army, Washington, D.C., 12p.

Vijayverigiya, V.N. and Ghazzaly, O.I., 1973, *Prediction of Swell Potentail for Natural Clays,* Proceedings of the 3rd International Conference on Expansive Soils, pp 227-236.

Wallace, K.B. and Lytton, R.L. (1992) *Lateral Pressures and Swelling in a Cracked Expansive Clay Profile*, Proc. 7th Int. Conf. on Expansive Soils, Dallas, Vol. 1, pp. 245-250.

Yevnin, A. and Zaslavsky, 1970, *Some Factors Affecting Compacted Clay Swelling,* Canadian Geotechnical Journal, Vol. 7, No. 1, pp 79-89.

857019