GEOTECH ENGINEERING AND TESTING

AND

FOUNDATION PERFORMANCE ASSOCIATION

PRESENTS

SEMINAR – FOUNDATIONS ON EXPANSIVE SOILS

November 18, 2005
~~SEPTEMBER 23, 2005~~

7:00 A.M. – 6 P.M.

AT

SHERATON NORTH HOUSTON HOTEL
17500 JFK BOULEVARD
HOUSTON, TEXAS

GEOTECH ENGINEERING AND TESTING
800 VICTORIA DRIVE
HOUSTON, TEXAS
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Geotech Engineering and Testing (GET) is a Texas owned, multi-disciplined organization. Our team of licensed engineers, geologists, field and laboratory technicians, and clerical personnel combine their technical capabilities, past experience, dedication, and enthusiasm to offer the finest services available. The firm offers a wide range of services for public, commercial, and industrial clients in Texas, Louisiana, New Mexico, and Oklahoma.

GET has a staff of about fifty-five (55) engineers, technicians, and support staff. All of our employees are located in our Houston office. The firm, which was established in 1985, provides the following services:

- **Forensic engineering**, *developing causations and remedial measures for distress in foundations, retaining walls, slopes, pavements and parking lots.*
- **Geotechnical engineering**, *including soil borings, laboratory testing, engineering analyses and recommendations regarding foundations, pavements, slope stability, retaining walls, ground improvement, construction considerations, etc.*
- **Construction materials engineering**, *including earthwork, asphalt, steel, and concrete testing*
- **Environmental engineering**, *including site assessments, monitor well installations, fault studies, and underground storage tank contamination studies.*

GET employees provide services on a vast number of diverse projects and clientele ranging from small architectural firms to large architectural/engineering companies, developers, contractors, and chain stores. The primary purposes of the firm are to promptly, accurately and comprehensively provide geotechnical reports, environmental studies and materials observations through our reasonably budgeted engineering services.

Geotech Engineering and Testing and its staff members have been involved in the following types of projects:

Commercial: Shopping Centers, Industrial Buildings, Chain Stores, Office Buildings, Hospitals, Churches, Retaining Walls, Service Stations, Fast Food Restaurants, etc.

Residential: Subdivisions, Residences, Apartment Complexes, etc.

Industrial: Industrial Sites, Petrochemical Complexes, Towers, Marine Terminals, Sea Walls, Electrical Substations, Power Plants, Tank Farms, Flare Stacks, Machine Foundations, Bulkheads, Erosion Protection Systems, etc.

Public: Wastewater projects, Roads, Bridges, Prisons, Parks, Airports, Storm Sewers, Pavement Repair, Educational Facilities, Libraries, Water, Dams, Slope Stabilization, Buildings, Fire Stations, Waste Disposal Facilities, Environmental Site Assessments - Phases I, II, and III; Underground Storage Tanks, Tunnels, Railroad Design, Ground Storage Tanks, Instrumentation, Drilling and Sampling, Fault Studies, Subsurface Studies, etc.

FOUNDATION PERFORMANCE ASSOCIATION

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MISSION

The mission of the Foundation Performance Association is to improve the performance of foundations for residential and other low-rise buildings.

The targeted membership of the Foundation Performance Association includes structural and geotechnical engineers, consultants, architects, builders, inspectors, and repair contractors actively engaged in the design, engineering, construction, inspection, assessment, and repair of lightly loaded foundations in Texas. Also targeted as associate or corporate members are product manufacturers, vendors, developers, attorneys, warrantors, realtors, lenders, appraisers, insurers, and other professionals involved in foundation maintenance, materials, litigation, warranty, finance, insurance, and other aspects of the residential and nonresidential low-rise building industry.

To accomplish our mission we will:

- Provide and maintain a nonprofit technical organization with appropriate bylaws for the targeted membership.
- Regularly hold open technical meetings, seminars, and other events in order to educate our targeted membership and the public, to promote the improvement in foundation performance, and to elevate the standards and ethics of those engaged in the foundation industry for residential and other low-rise buildings.
- Organize committees with appropriate rules and peer review, with the goal of researching and writing documents such as guides and recommended practices that are beneficial to our targeted membership and the public.
- Publish our documents through a website, making them freely available to our targeted membership and the public.

SEMINAR - FOUNDATIONS ON EXPANSIVE SOILS
PROGRAM AGENDA
FRIDAY, SEPTEMBER 23, 2005

Time	Topics
6:45 am	Registration
7:15 am	Seminar Opening by David Eastwood, P.E.
7:25 am	Tribute to Professor Michael O'Neill - Mr. ~~Kenneth Tand~~ RAY MEYER, P.E. – Kenneth Tand and Associates, Inc.
7:35 am	Introduction of Unsaturated Soils by Dr. Lytton, P.E.
8:00 am	Computations of Swell and Shrinkage In Expansive Soils – Dr. Lytton Development of Volume Change Parameters Use of Soil Suction Concepts Depth of the Moisture Active Zone Depth of the Movement Active Zone Comment on PVR
8:45 am	Break
8:55 am	Continuation – Lytton
9:30 am	Field Exploration and Site Conditions - Meyer
10:00 am	Laboratory Testing - Meyer Swell Tests Procedures Soil Suction Tests
10:30 am	Break
10:40 am	Geotechnical and Structural Design of Post-Tensioned Slabs-on-Ground using PTI 2004 Manual and Computer Programs VOLFLO 1.5 and PTISLAB 3.0 - Meyer, Read
12:00 – 1:00 pm	Lunch
1:00 pm	Continuation - Meyer, Read
1:10 pm	Design Concepts of Various Foundation Systems – Dr. Lytton Drilled Footings Floating Slabs Moisture Barrier Root Barrier Pavements Slopes
2:20 pm	Break
2:40 pm	Construction Maintenance and Inspection – Price
3:40 pm	Break
3:50 pm	Forensic Evaluation of Foundations – Mr. Eastwood, P.E.
4:50 pm	Legal Issues – Mr. David Dorr, P.E., Esquire
5:20 pm	Panel Discussion Questions and Answers
6:00 pm	Adjorn

Seminar – Foundations on Expansive Soils
Course Notes
September 23,2005
Table of Contents

Mr. Kirby Meyer, P.E. and Mr. Dean Read, P.E. – Resumes

Field Exploration and Site Conditions

Application of Geotechnical and Structural Procedures for Expansive Soil Using PTI 3rd Edition Manual

Mr. Russel Price, P.E. – Biography

Post-Tensioned Prestressed Concrete

Mr. David Dorr, P.E., Esquire

Liability Management in Foundation Engineering

Foundation Performance Association Documents

Post Foundation Repair Performance of Residential and Other Low-Rise Buildings on Expansive Soils

Foundation Design Options for Residential and Other Low-Rise Buildings on Expansive Soils

Distress Phenomena Often Mistakenly Attributed to Foundation Movement

Recommend Practice for Geotechnical Explorations and Reports

Foundation Maintenance and Inspection Guide for Residential and Other Low-Rise Buildings

Design, Manufacture, and Installation Guidelines of Precast Concrete Segmented Piles for Foundation Underpinning

Quality Control Checklists for Foundation Inspection of Residential and Other Low Rise Buildings

ASCE Papers

Recommended Practice for the Design of Residential Foundations

Guidelines for the Evaluation and Repair of Residential Foundations

DAVID A. EASTWOOD, P. E., C.A.P.M.
PRESIDENT

SUMMARY

Mr. Eastwood is the President and Chief Engineer with Geotech Engineering and Testing (GET). He has practiced forensic engineering for about 26 years, serving in key technical project management and administrative roles. His experience in these functions includes a wide range of project types and large capital investments, ranging from commercial, industrial, residential to infrastructure projects. Mr. Eastwood's extensive experience is to provide clients with causations of distress on distress projects. Mr. Eastwood's experience has been in the areas of buildings, roads, parking lots, slopes, retaining walls, sewer leaks, pool leaks, etc.

EDUCATION

1977 Bachelor of Science in Civil Engineering,
University of Houston

1978 Master of Science in Civil Engineering,
University of Houston

1978 Present
Post Graduate studies at Princeton, Rice University, and the University of Houston

LICENSES

Licensed Professional Engineer - Texas No. 51419
Licensed Professional Engineer - Louisiana No. 25966
Licensed Professional Engineer - New Mexico No. 12576
Licensed Professional Engineer - Oklahoma No. 17513
Corrective Action Project Manager - Texas C.A.P.M. No. 01181

EXPERIENCE

1985 - Present

Geotech Engineering and Testing, Houston, Texas - President

1978 - 1985

Various Companies, including McClelland Engineers, Inc., Terra-Mar, Inc. and National Soil Services, Inc.

OVERALL EXPERIENCE

1. Forensic (Foundation) Engineering and expert testimony for residential, commercial and road projects. Mr. Eastwood is the founder and former President of the Foundation Performance Association, an association of engineers specializing in the evaluation of distress. Mr. Eastwood is on the Design Committee of Texas Board of Professional Engineers, Residential Foundation Committee. In addition, Mr. Eastwood is the Chairman of the Post-Tensioning Institute Slab-on-Grade Geotechnical Subcommittee. This committee develops geotechnical design guidelines for design of post-tensioned slabs-on-grade throughout the United States.

2. Mr. Eastwood is on the American Society of Civil Engineers (ASCE), Texas Section, Committee that developed the document "Recommended Practices for the Design of Residential Foundations."

3. Soils and foundation studies for design and construction of buildings, chain stores, subdivisions, high rises, parks, schools, shopping centers, apartment complexes, prisons, petrochemical complexes, highways, bridges, water, wastewater, ports, airports, rail projects, and waterfront structures.

4. Analysis of experimental test data and correlation of data with respect to swelling characteristics of expansive soils as they relate to design of residential and commercial structures.

5. Extensive computer programming and analyses capabilities with respect to:

   (a) heave
   (b) slope stability of embankments
   (c) pile foundations
   (d) settlement
   (e) dynamics of foundations
   (f) seepage
   (g) expansive soils

6. Environmental site assessment studies, waste management, field studies, monitor well installations, laboratory testing, recommendations regarding contaminations of landfills, underground storage tanks, remediations, and permitting. Mr. Eastwood is also a Corrective Action Project Manager (C.A.P.M.). He is also a Certified Environmental Inspector (C.E.I.).

7. Geologic fault studies.

PUBLICATIONS

"State of Art on Expansive Clays", report submitted to the American Society of Civil Engineers Shallow Foundation Committee on Expansive Clays, 1978.

"Hazards of Expansive Clays", Presented before the ASCE Convention in Portland, Oregon, April, 1980.

"Methodology for Foundations on Expansive Clays", published in December, 1980 edition of ASCE Journal of Geotechnical Engineering Division.

"Geotechnical Considerations in Design of Hazardous Waste Impoundments", presented before the ASCE Texas Section Spring Meeting in Fort Worth, Texas, March 1982.

"Recommended Homeowner Foundation Maintenance Program For Residential Projects In The Houston Area", published in April, 1990 Edition of Houston Builder.

D. Eastwood "Geotechnical Guidelines For Design of Residential Projects In The Houston Area", presented in the Soil-Structure Interaction Seminar, July 1994.

D. Eastwood and others "Reasons for Foundation Failure", presented in the Soil- Structure Interaction Seminar, Houston, June 1996.

D. Eastwood and others "Design of Foundations with Trees in Mind", presented before the ASCE, Texas Section, Spring Meeting in Houston, April 1997.

D. Eastwood and others "Design of Residential Foundations on Expansive Soils in Texas." Report developed for the Texas Board of Professional Engineers, March 1998.

D. Eastwood and others "State of Practice for Geotechnical Engineering for Design of Custom Homes in the Houston Area between 1990 to 2001" Presented before ASCE, Texas Section, Spring Meeting in Arlington, April 2002.

D. Eastwood and others "Application of the New e_m, y_m Soil Parameters" Presented before PTI Conference and Exhibition May, 2002.

H. Stephen Tien, Ph.D and D. Eastwood, P.E. "Case Study of the Pavement Distress at a Service Station" Presented before ASCE, Texas Section, Fall Meeting, Dallas, September 2003.

H. Stephen Tien, Ph.D, P.E. and D. Eastwood, P.E. "Case Studies of Residential Foundation Movements in Southern Houston Area" Presented before ASCE, Texas Section, Fall Meeting, Houston, September 2004.

DEVELOPMENT OF DESIGN AND REMEDIAL MEASURES FOR LIGHTLY-LOADED STRUCTURES FOUNDED ON EXPANSIVE SOILS WITH TREES IN MIND

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ABSTRACT

A review of the current literature in the United States shows there may be an absence of practical approaches for the design of lightly-loaded structures founded on expansive clay soils where trees are involved. As a result, many such structures which have experienced distress as a result of the lack of the consideration of the effect of trees in the design of these structures. The authors of this paper are a part of a Houston organization called the Foundation Performance Committee. One of the activities of this Committee is the investigation of the adverse effect that the presence and/or the removal of trees can have on lightly-loaded structures. This paper includes the results of this activity, and also includes the results of a literature survey, a review of the failures of some foundations whose cause has been attributed to the presence of and/or the removal of trees, design recommendations, and repairs where trees have been identified as the primary cause.

INTRODUCTION

A significant number of residential buildings were constructed in the greater Houston are in the 1950 through 1970 time periods. Many of these buildings were founded on expansive soils and on building lots, which were void of vegetation. Trees were then planted after these building were sold and eventually the trees matured and caused foundation failure to occur. Corrective measures generally included the underpinning of the foundation perimeter beam using drilled piers and later pressed piles. When piers were used, this approach had, at best, a limited degree of success. The type of failure mode, whereby trees cause the settlement of the perimeter of residential, and other low-rise buildings, has received a significant amount of publicity in recent times and, as a result, home owners, building remodeling contractors, home builders, etc. have become aware of this problem. From this, an entire industry has grown which provides measures to control this problem.

Some notice of the manner in which trees can produce downward deflections in residential slab-on-ground foundations began to appear in the early 1970's. For example, in 1972, Davis[1] summarized existing papers which indicated that near-by trees could adversely affect foundation performance. Davis and Tucker[2] published a Technical Note which provided data which showed that Post Oak trees located South of Arlington, Texas, incurred vertical movements varying from 1.2 inches to 3.4 inches between the end of the summer months and the end of the winter months. The University of Texas at Arlington conducted an investigation of 69 abandoned residential buildings which were founded on clay soils and reported, among other things, that the extraction of moisture from the soil through the roots of trees, caused moderate to severe deflections in foundations.[3] In 1974, Buckley [4] presented data which showed damage to foundations caused by trees. Kramer and Kozlowski[5] identified the comparatively high transpiration rates of some trees in 1960. In 1987, Peverley and Hanys[6] provided measured deflections in a residential foundation which had been produced by near-by trees.

The manner in which trees can cause downward deflections in residential slab-on-ground foundations has, therefore, been well documented. Simply stated, in order to satisfy their need for soil water, trees can desiccate the soil upon which the outer edge of a foundation rests, resulting in the shrinkage of the soil with the attendant loss of soil support. Foundation distress attributable to such causes have occurred with such regularity in the greater Houston area as to have caused a major alteration to the fundamental foundation design and construction concepts

Presented at the Spring Session of the Texas Section of ASCE

with were in effect for years. Not as well understood, however, are the adverse effects that the removal of large trees can have on reconstructed foundations, even where they are resting on drilled piers. Equally misunderstood is the relationship between tree root growth and under-slab sewer leaks.

The purpose of this paper is to explore the adverse effects that trees can have on residential foundation performance based on the experience of others as documented in the literature, based on the personal experience of the authors, and based on an accumulation of information form the Foundation Performance Committee. This paper will be presented in the three following basic parts; foundation edge settlement produced by soil shrinkage, foundation edge heaving caused by soil swelling, and foundation center settlement caused by the interaction between tree roots and under-slab sewer leaks. The mechanics of such conditions along with proposed corrective measures will be discussed. Examples will be presented.

SOIL MECHANICS AS AFFECTED BY TREES

THE PHYSIOLOGY OF TREES

Trees have long been considered to be a benefit to mankind. Trees have been written about as many as 4000 years ago.[7] Trees absorb heat as they transporate, provide shade, and reduce solar radiation. They enhance air purification, aid in the control of erosion, and can, to a limited degree, provide some noise reduction benefits. Perhaps their most appreciated benefit is their ability to enhance the beauty of the surrounding landscape. One can appreciate the beauty of old oaks whose branches provide an umbrella for many of the roads in the old South or whose sculpture enhances the skyline of the Pacific Coast.

Trees do have their downside. Their limbs fall injuring property and people. In the Gulf coast, people have been injured or killed by falling trees. Tree roots clog sewers and break sidewalks. Trees can also increase the ozone content of the air, damage electrical power lines, and interfere with UHF reception. Perhaps the highest cost of trees is in their damage to residential foundations. In 1973, Jones and Holtz [8] estimated the annual cost of expansive soils in the US to be 2.2 billion dollars. In 1982, Peverley & Hanys [6] estimated the cost to repair only those residential foundations in the greater Houston, Texas real estate market for a 6-month period of time to be in excess of 28.5 million dollars. If one were to conservatively estimate that only 50% of these foundation failures were caused by trees, the costs would be obviously enormous.

Trees are the largest plants in the world.[9] Trees can generally be classified as needle-leaf or broad leaf (deciduous). The essential parts of a tree are the crown, the trunk, and the roots. The crown contains the leaves, which essentially absorbs sunlight and converts it into food. The roots are the fastest growing part of a tree. They collect water and transport it through the trunk to the leaves in the form of sap. The trunk provides the transporting mechanism between the leaves and the roots and is made up of the heartwood in the center, the cambium layer at the outer edge of the trunk, and the bark, which provides the primary protection. Roots grow only as fast as they are provided energy from the leaves. The tree system consists of the circulation of water from the roots upward through their trunk in the form of sap. When the sap reaches a leaf, the water evaporates into the air. The sap brings mineral salts from the earth to the leaves. The chlorophyll in the leaves acts with sunlight to convert the salts into food through photosynthesis. This food then flows back into the tree system through paths just below the bark. It is this system which makes a tree live.

In engineering terms, we are primarily concerned with the term evapotransporation; i.e., the withdrawal of moisture from the soil and its eventual transporation into the atmosphere. Attempts have been made to quantify this term; however, the results have not always been uniformly accepted in the engineering/ arborist communities, primarily because of the inability to accurately measure the moisture loss/replacements in the soil under most trees. Driscoll[10] presented a ranking of trees in terms of their damage potential. A modified copy of this ranking is contained in Table 1.[11]

In the greater Houston area, we do not have an abundance of Poplar trees; however, there well may be more Oak trees than any others. Also contained in this document is an example of seasonal moisture content variations, which is shown in Figure 1. Of interest is the identification of a zone of permanent moisture deficiency. This concept was further explored by Biddle[12] who measured the soil moisture content in the close proximity of various kinds of trees which were growing in a variety of clay soils, all of which were in England. A combined moisture reduction/moisture deficit curve for a Poplar tree growing

Table 1. Risk of damage by different varieties of tree

Ranking	Species	Maximum height of tree (H): metres	Separation between tree and building for 75% of cases: metres	Minimum recommended separation in shrinkable clay: metres
1	Oak	16-23	13	1H
2	Poplar	24	15	1H
3	Lime	16-24	8	0.5H
4	Common Ash	23	10	0.5H
5	Plane	25-30	7.5	0.5H
6	Willow	15	11	1H
7	Elm	20-25	12	0.5H
8	Hawthorn	10	7	0.5H
9	Maple/ Sycamore	17-24	9	0.5H
10	Cherry/Plum	8	6	1H
11	Beech	20	9	0.5H
12	Birch	12-14	7	0.5H
13	White Beam/ Rowan	8-12	7	1H
14	Cypress	18-25	3.5	0.5H

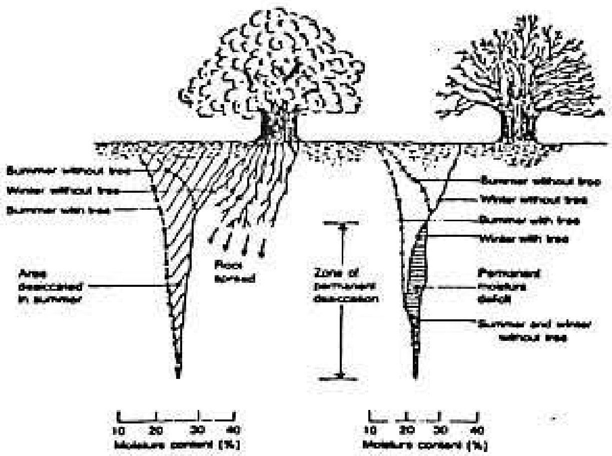

Figure 1. Seasonal variation in moisture content with and without trees

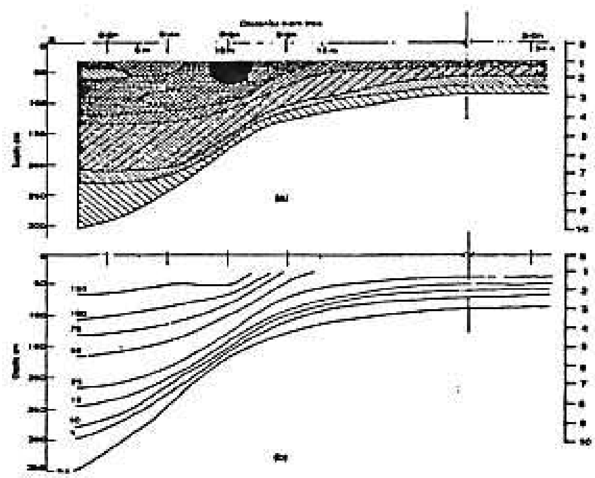

Figure 2. An example of soil moisture and soil moisture deficit reduction curves for popular trees on a bolder clay soil in London, England

in a Bolder Clay (PI = 29%) is shown in Figure 2. The moisture deficit curves are calculated by multiplying the change in moisture content by the appropriate layer thickness. In reviewing this curve, it is significant that it does represent the most severe condition for a tree which has been judged to be of a lesser threat than would be an Oak tree growing in a soil whose Plasticity Index is less than some of the major areas in the greater Houston, Texas area. The availability of such data in England has had significant impacts on the construction business. Whereas it was considered to be impractical to plant any tree closer to a foundation than its ultimate height, these data do provide some bases for the planting of certain trees closer to buildings, assuming the data provided for these curves are considered. They also demonstrate the folly of simply removing existing trees in expansive soils for the construction of a new residential structure.

WHAT CAUSES EXPANSIVE SOILS TO SHRINK OR SWELL

As clay particles are formed, there are usually several points in the particle arrangement where there is an electrical imbalance; the electrical imbalance is increased whenever a "string" of clay particles is broken apart. Thus, the result is that a clay particle typically has a negative net electrical charge on its surface. Since nature likes all things to be balanced, whenever a water molecule drifts close enough to the surface of a clay particle, the negatively charged surface of the clay particle causes the positive end of the water molecule to turn toward the particle. If it is close enough to the particle, the water molecule is attracted to the clay particle surface sufficiently strongly that the water molecule becomes trapped. Also, unattached or "free" positively charged particles, called "cations", tend to acquire a spherical-shaped arrangement of water molecules which have their negative ends directed toward the positively charged cation (and their positive ends directed away from the cation). When the free cation is "captured", water molecules approach a clay particle. The attraction between the negatively charged clay particle surface and the positively charged outside of the cation sphere of water molecules causes the cation to be "captured" by the clay particle, thus increasing the amount of water associated with the clay particle.

Clay particles are very small. A typical kaolinite particle might have a total surface area (top, bottom and edges) of approximately 1 x 10-5 mm^2 (1 x 10-10 ft^2, or 0.0000000001 ft^2). As areas go, this is very small. Smectite particles have a diameter that is 100 to 1,000 times smaller than kaolinite particles and a thickness that is 10 to 400 times thinner than kaolinite[13] and, consequently, typically have a larger surface area per particle. Thus, a single pound of montmorillonite particles would have an incredible total surface area of approximately 800 acres (325 hectares) [13] with which to attract water.

Thus, expansive soils are very small in size and have a large surface area that attracts free water. Because of these characteristics, it is easy to see why it is said that expansive soils are those clays that exhibit an extreme change in volume.

Soil suction is a measure of free energy of the pore-water or tension stress exerted on the pore-water by soil matrix. Soil suction is, in practical terms, a measure of the affinity of the soils to retain water and can provide information on soil parameters that are influenced by soil water; e.g., volume change, deformation, and strength characteristics of soil. The soil suction is measured using the filter paper method in accordance with ASTM D-5298.

The soil suction is divided into two components; Matrix suction and osmotic suction. The matrix suction is the negative pressure (expressed as a positive value) relative to ambient atmospheric pressure on soil water, to which solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with soil water; pressure equivalent to permeable wall with the soil water; and pressure equivalent to that measured by test methods D2325 and D3152.

The osmotic suction is the negative pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with a pool containing a solution identical in composition with soil water; decrease in relative humidity due to the pressure of dissolved salts in pore-water.

FOUNDATIONS AND RISKS

Many lightly loaded foundations are designed and constructed on the basis of economics, risks, soil type, foundation shape and structural loading. Many times, due to economic considerations, higher risks are accepted in foundation design. Most of the time, the foundation types are selected by the owner/builder, etc. It should be noted that some levels of risk are associated with all types of foundations and there is no such thing as a zero risk foundation. All of these foundations must be stiffened in the areas where expansive soils are present and trees have been removed prior to construction. The foundation types typically used in the area with increasing levels of risk and decreasing levels of cost are discussed in Table II.

The above recommendations, with respect to the best foundation types and risks, are very general. The best type of foundation may vary as a function of structural loading and soil types. For example, in some cases, a floating slab foundation may perform better than a drilled footing type foundation.

FOUNDATION PROBLEMS CAUSED BY TREES

FOUNDATION SETTLEMENT PRODUCED BY SOIL SHRINKAGE

Several authors as far back as 1960 have documented this type of distress.[8,14] Buckley[3] proposed that trees be placed no closer to a residential foundation than its ultimate height. The basis for this recommendation is contained in Figure 3. It was not, however, widely recognized by the designers and constructors of the millions of

Table II.

Structural Slab with Piers	This type of foundation (which also includes a pier and beam foundation with a crawl space) is considered to be a minimum risk foundation. A minimum crawl space of six-inches or larger is required. Using this foundation, the floor slabs are not in contact with the subgrade soils. This type of foundation is particularly suited for the area where expansive soils are present and where trees have been removed prior to construction. The drilled footings must be placed below the potential active zone to minimize potential drilled footing upheaval due to expansive clays. In the areas where non-expansive soils are present, spread footings can be used instead of drilled footings.
Slab-On-Fill-Foundation Supported on Piers	This foundation system is also suited for the area where expansive soils are present. This system has some risks with respect to foundation distress and movements, where expansive soils are present. However, if positive drainage and vegetation control are provided, this type of foundation should perform satisfactorily. The fill thickness is evaluated such that once it is combined environmental conditions (positive drainage, vegetation control) the potential vertical rise will be minimum. The structural loads can also be supported on spread footings, if expansive soils are not present.
Floating (Stiffened) Slab Supported on Piers. The Slab can either be Conventionalty-Reinforced or Post-Tensioned.	The risk on this type of foundation system can be reduced sizably if it is built and maintained with positive drainage and vegetation control. Due to presence of piers, the slab can move up if expansive soils are present, but not down. In this case, the steel from the drilled piers should not be doweled into the grade beams. The structural loads can also be supported on spread footings if expansive soils are not present.
Floating Slab Foundation (Conventially-Reinforced or Post-Tensioned Slab)	The risk on this type of foundation can be reduced significantly if it is built and maintained with positive drainage and vegetation control. No piers are used in this type of foundation. Many of the lightly-loaded structures in the state of Texas are built on this type of foundation and are performing satisfactorily. In the area where trees have been removed prior to construction and where expansive clays exist, these foundations must be significantly stiffened to minimize the potential differential movements as a result of subsoil heave due to tree removal.

homes built on concrete slab-on-ground foundations in the 1950 time period. In Houston, Texas, as an example, the post World War II building boom included upwards of 100,000 homes constructed in the Southwest part of the City, where the soils were typically very expansive. In most cases, these homes were initially constructed in subdivisions outside of the City limits, but were later annexed by the City. Thus, no building codes were applied. Since this real estate was largely farm land which was barren of trees, one of the things that were done by individual homeowners (and even some subdivision developers) was to plant trees in the yard close to the foundation. Trees such as Oaks, China Berry, and Pecan were popular because they were hardy. When the trees entered into their period of major growth, their water demands steadily increased and foundation problems began.

Studies[15] have shown that when a slab is placed on ground, evaporation of soil moisture is retarded. If the

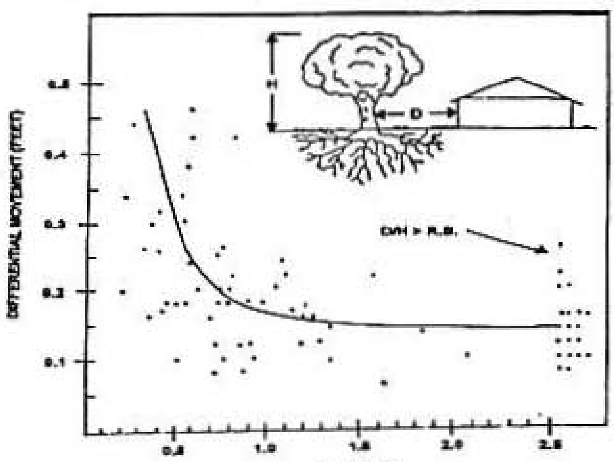

Figure 3. A curve of foundation vertical movement versus tree height

soil is desiccated at the time of construction, moisture will move toward the center of the slab and is going to be higher than at the edge. Trees use soil moisture for growth. Therefore, during wet periods, sufficient supply of moisture is available for tree growth and soil moisture contents may not be substantially affected by vegetation. During dry summer months, when evaporation rates are high, the trees will obtain large quantities of moisture from already dry soils. If these trees are located in close proximity to lightly-loaded structures such as houses, their root system will move toward the structures, in an attempt to find a moisture supply. If trees are too close to a house, desiccation of the soils below the slab may occur, causing settlement of the slab.

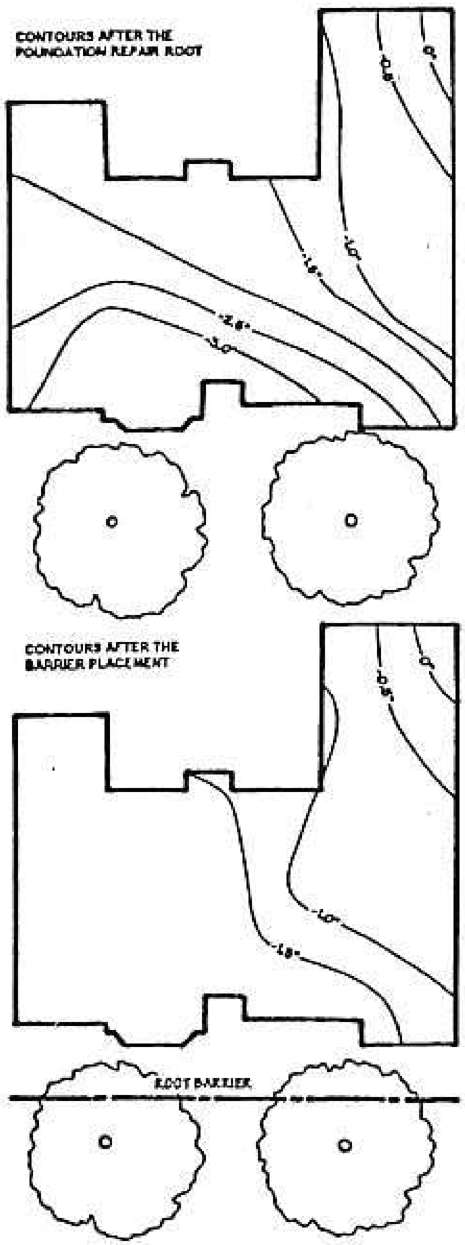

Figure 4. An example of failure of a foundation as a result of the depletion of moisture at the edges because of trees. Measurements were taken after pier installation and after root barrier installation

Figure 4 shows an example of a home located in the Southwest part of Houston, Texas, whose slab-on-ground foundation was underpinned using drilled piers. During the 1988 to 1990 drought, the foundation incurred additional deflections. At our suggestion, a root barrier, combined with an automatically actuated soaker system was applied, and the foundation not only stabilized, but some rebound occurred. The example contained in Figure 4 is not, unfortunately, an isolated example. Better results have been obtained in the recent past using pressed segmented piles and helical piers.[16] Vertical moisture barriers have enjoyed a very limited use.

FOUNDATION EDGE HEAVING CAUSED BY SOIL SWELLING

To compensate for the settlement of the outer edge of a foundation due to the extraction of moisture from the soil through the roots of nearby trees, an accepted preventative measure was to place the foundation on top of drilled piers. Most recently, however, a condition has occurred where the piers became a detriment. In the inter part of the City of Houston, Texas, there are subdivisions with comparatively small building lots, which often contained small, older homes, became very desirable because of their location. In many cases, the lots contained large, prolific trees, which were removed to make way for the construction of larger homes. In many such cases, nothing was done to compensate for the inevitable swelling of the soils which would occur when the tree, which had desiccated the soil in its near vicinity, was gone and soil suction forces moved soil water into the desiccated areas. It does not take much imagination to invision the mood of a homeowner who, in many cases, paid extra moneys to have a sturdy foundation constructed only to have it begin to move soon after the owner moved in. An example of such a condition is shown in Figure 5. The foundation, in this example, was founded on 10-foot deep drilled piers, which had 42-inch diameter bells that were inspected during construction. The pier shafts were tied to the concrete perimeter beam. The soils had a plasticity index in the 60% range. A Pecan tree was remove during the construction process, or shortly thereafter. Signs of foundation induced damage became manifested within the first year of construction and have, in the interim, worsened steadily. A literature search failed to reveal any documented discussion of this phenomenon, not only in the state of Texas, but in the United States, as well. Such discussions were, however, found in literature from outside of the United States. A listing of such sources is

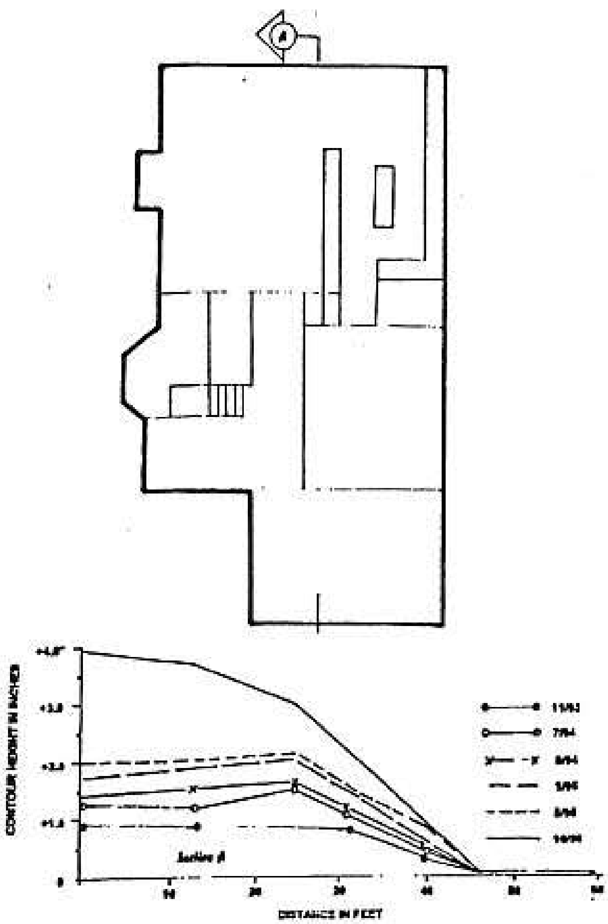

Figure 5. An example of a home which had been adversely effected by the removal of a Pecan tree

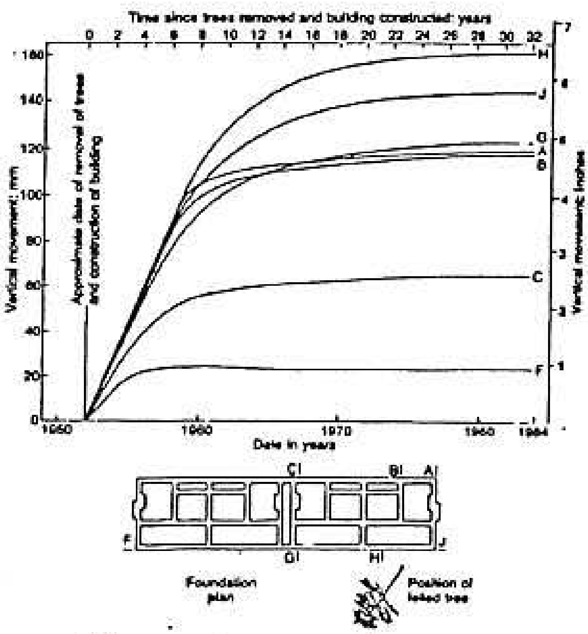

Figure 6. A copy of a vertical movement that occurred as a result of the removal of a Poplar tree in London, England

contained in the Bibliography Section of this paper. This significance of the tree removal situation is perhaps best illustrated in Figure 6[17]. In this case, the removal of a Poplar tree caused over 6 inches of heaving which was monitored over a 30-year period of time.

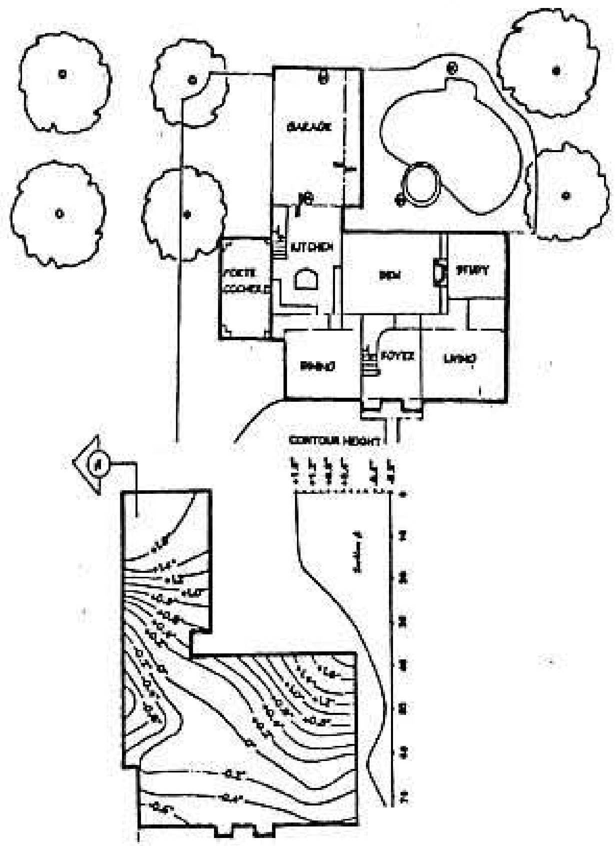

Figure 7. An example of a home in Houston, Texas, which had been constructed on a roadway that was lined with Post Oak trees

Figure 7 shows how a residential building was placed on the edge of a roadway, which, before the time of construction, had Post Oak trees on either side. The removal of the trees caused significant heaving of the foundation at the rear of this, and several other buildings, which were constructed along this roadway. We have been able to measure heaving in a limited number of cases and the results are contained in Figure 8. In comparing our data with that shown in Figure 9, we can see that the slope of our data is steeper, even though one of the curves seemed to level out after 7 years.

As mentioned earlier, tree roots tend to desiccate the soils. In the event that the tree has been removed prior to building construction, during the useful life of the structure, or if a tree dies, subsoil swelling can occur in the expansive

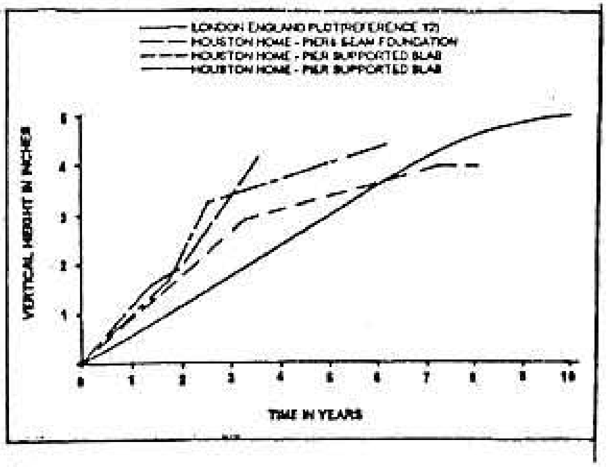

Figure 8.

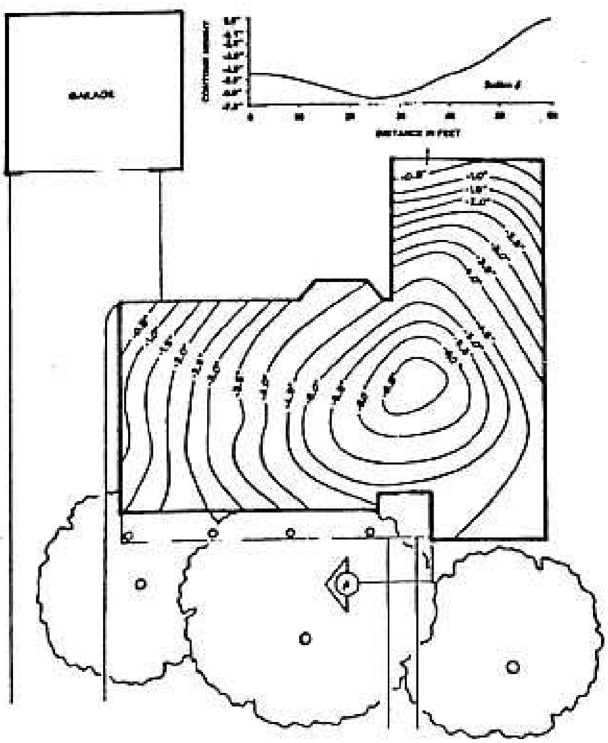

Figure 9.

soil areas for several years. Studies have shown that this process can take several years in the area where highly expansive clays are present. In this case, the foundation for the structure should be designed for the anticipated maximum heave. Furthermore, the drilled footings, if used, must be placed below the zone of influence of tree roots. This depth should be evaluated as follows:

a) The pier should be placed below the depth of constant suction or the zero movement line.

b) The pier depth should be such that it could resist the uplift loads due to expansive soils that extend along the shaft perimeter.

c) More extensive soil tests are required. Soil borings near a tree must be, as a minimum, 25 to 30 feet deep. The depth to which tree root fibers exist must be determined since they are a basis of identifying the depth of constant soil suction. Potential Vertical Movement values should also be calculated.

In the event that a floating slab foundation is used, we recommend the slab be stiffened to resist the subsoil movements due to the presence of trees. In addition, the area within the tree root zone may have to be chemically stabilized to reduce the potential movements. Alternatively, the site should be left alone for several years so that the moisture regime in the desiccated areas of the soils (where tree roots used to be) become equalized/stabilized to the surrounding subsoil moisture conditions. The length of time required for subsoils to regain their moisture is dependent on the tree species, soil type and the amount of rainfall. For most trees, one wet season may be enough for the subsoils to regain their moisture; however, removal of trees such as Live Oak, Poplar, etc. may result in moisture deficits in the soil profile that may require several years to stabilize.

Remedial measures to correct the adverse effects of this type of soil heaving are somewhat limited. One method is to raise the entire foundation out of the potential vertical rise of the soil using underpinning techniques. Soil testing will generally show the PVM (Potential Vertical Movement) values in the soil where the trees were removed. It may then become necessary to raise the foundation out of this zone. An alternative is to use a vertical moisture barrier. A combination of partial underpinning and the use of moisture barriers may also have to be used to stabilize the foundation system.

FOUNDATION CENTER SETTLEMENT

There has been an ever-increasing problem with regard to the interaction between trees and foundation performance; i.e., foundation performance induced by under-slab sewer leaks. Many of the homes constructed in the 1950 time period had cast iron, under-slab sewer pipes buried in clay soil. Over the past 40 (+) years the effect of this unfortunate marriage has produced a proliferation of under-slab sewer leaks. Since most (if not all) of the home insurance policies allow a homeowner to collect on damages caused by such leaks, there has been an attendant number of such claims filed.

Typically, the insurance carrier hires a plumbing testing company and an Engineer to determine if the leak has caused any foundation related damage. It is likewise typical that this same Engineer will observe that the foundation has deflected downward in its center section, which is the opposite of what one might anticipate if water were to be induced into expansive soils. Other contradictions may be observed which included the following:

- o The timing of the damage appeared coincidental to the occurrence of the sewer leak.
- o There were plumbing leaks; yet, where soil tests were conducted, the soils were comparatively dry.
- o There was always a reasonable degree of correlation between the presence of the sewer leaks and the points of deflection.
- o In a majority of cases, the soils were expansive, the foundation was constructed on drilled piers or was underpinned using drilled piers subsequent to the time of original construction, and there were trees growing near the foundation which were almost always mature. It is a known fact that the water demands of mature trees tend to stabilize. Could these trees then suddenly become the source of additional foundation deflections?
- o In the 20(+) cases which we examined, the foregoing conditions existed and the foundation settled in the center instead of heaving, as was anticipated. An example is shown in Figure 9. In this case, the sewer system could not be tested since it would hold no water.

We are of the opinion that the introduction of the sewer water spurred the growth of the tree roots to grow towards the source.[18, 19] The tree roots then extracted not only the moisture provided from any sewer leaks which occurred, but also any moisture which was in the soil before the leak occurred. The presence of an under-slab sewer leak then resulted in a net soil moisture loss where large trees were growing adjacent to the foundation with the ultimate result that the foundation subsided instead of heaving, as one might anticipate.

This opinion does, of course, involve a number of assumptions for which no proof exists. In fact, there is little of no real proof that any sewer leak did, or did not, cause foundation deflections to occur. More testing and study is required.

CONCLUSIONS AND RECOMMENDATIONS

The design and manufacture of most of the material things we use in our lives is based, to at least some degree, on some type of research. Automobiles are designed and extensively tested before they reach the market, manufacturers of appliances subject them to extensive testing before new models are put up for sale, new food products must be given extensive testing, etc. It is then somewhat ignominious that the design and construction of the fundamental part of what is perhaps the most expensive investment for most of us is based on little, if any, current research; at least in the United States. Instead, we tend to learn in the most fundamental, and in the crudest, of ways - by trial and error. The cost of this process is born by the builders, homeowners, and engineers much to the delight of many attorneys.

We have, in this paper, pointed out some of the problems that can be caused by our failure to learn to live with trees in an urban environment. Although much of this discussion was based on Houston, Texas, experience, this information certainly applies to much of Texas and to other parts of the country, as well. All of us who are involved in the design and construction of residential and other low-rise buildings need to be cognizant of these problems and to conduct ourselves accordingly. This may require additional pre-construction testing and may necessitate the need for more expensive designs. Some may say that our clients may not be willing to pay the price for such extra work. So long as there are engineers who are willing to do cheap work, the problems we discussed herein will recur and we will be left to ponder why some people are more willing to pay their attorneys more than they are their engineers and or builders.

We have pointed out the need for research. To the best of our knowledge, the last 2 large research studies conducted on residential foundation issues were the BRAB in the 1960's and the University of Texas at Arlington studies in the 1970's. We do know of some smaller studies, which have been conducted at some Universities, but we believe that larger studies are needed not only on the issues presented herein but on sister issues, as well. Some of the study areas are listed below:

- o A relationship needs to be developed that would address the tree type (species), distance from the foundation, and height of the tree.
- o Studies similar to those conducted by Biddle should be done using trees more typically found in the United States (Oaks, Pecans, China Berry, etc),in clay soils and varying weather patterns that are typical of this country.
- o How to better design floating slabs that would resist the effect of trees.
- o Develop a simple mathematical module that would relate sewer leaks, tree moisture removal, and subsoil movements.

Perhaps the information contained herein will help in the search for research dollars.

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GEOTECH ENGINEERING and TESTING

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Geotechnical, Environmental, Construction Materials, and Forensic Engineering

RECOMMENDED HOMEOWNER FOUNDATION MAINTENANCE PROGRAM FOR RESIDENTIAL PROJECTS IN THE HOUSTON AREA

BY

DAVID A. EASTWOOD, P.E.

10-04

Introduction

Performance of residential structures depends not only on the proper design and construction, but also on the proper foundation maintenance program. Many residential foundations have experienced major foundation problems as a result of owner's neglect or alterations to the initial design, drainage, or landscaping. This has resulted in considerable financial loss to the homeowners, builders, and designers in the form of repairs and litigation.

A properly designed and constructed foundation may still experience distress from vegetation and expansive soil which will undergo volume change when correct drainage is not established or incorrectly controlled water source becomes available.

The purpose of this document is to present recommendations for maintenance of properly designed and constructed residential projects in Houston. It is recommended that the builder submit this document to his/her client at the time that the owner receives delivery of the house.

Typical Foundations

Foundations for support of residential structures in the Houston area consist of pier and beam type foundation, spread footing foundation, conventionally reinforced slab, or a post-tensioned slab. A soils exploration must be performed before a proper foundation system can be designed.

General Soil Conditions

Variable subsoil conditions exist in the Houston Metro area. Highly expansive soils exist in the West University, Bellaire, Southwest Houston, Clear Lake, Friendswood, Missouri City, and First Colony areas.

Sandy soils with potential for severe perched water table problems as a result of poor drainage are present in the North and West Houston, including portions of Piney Point, Hedwig Village, The Woodlands, Kingwood, Atascocita, Cypresswood, Fairfield, etc.

A perched water table condition can occur in an area consisting of surficial silty sands or clayey sands underlain by impermeable clays. During the wet (rainy) season, water can pond on the clays (due to poor drainage) and create a perched water table condition. The sands become extremely soft, wet, and lose their load carrying capacity.

Drainage

The initial builder/developer site grading (positive drainage) should be maintained during the useful life of the residence. In general, a civil engineer develops a drainage plan for the whole subdivision. Drainage sewers or other discharge channels are designed to accommodate the water runoff. These paths should be kept clear of debris such as leaves, gravel, and trash.

In the areas where expansive soils are present, positive drainage should be provided away from the foundations. Changes in moisture content of expansive soils are the cause of both swelling and shrinking. Positive drainage should also be maintained in the areas where sandy soils are present.

Positive drainage is extremely important in minimizing soil-related foundation problems.

The homeowners berm the flowerbed areas, creating a dam between the berm and the foundation, preventing the surface water from draining away from the structure. This condition may be visually appealing, but can cause significant foundation damage as a result of negative drainage.

The most commonly used technique for grading is a positive drainage away from the structure to promote rapid runoff and to avoid collecting ponded water near the structure which could migrate down the soil/foundation interface. This slope should be about 3 to 5 percent within 10-feet of the foundation.

Should the owner change the drainage pattern, he should develop positive drainage by backfilling near the grade beams with select fill compacted to 90 percent of the maximum dry density as determined by ASTM D 698-91 (standard proctor).

This level of compaction is required to minimize subgrade settlements near the foundations and the subsequent ponding of the surface water. The select fill soils should consist of silty clays and sandy clays with liquid limits less than 40 and plasticity index (PI) between 10 and 20. Bank sand or top soils are not a select fill. The use of Bank sand or top soils to improve drainage away from a house is discouraged; because, sands are very permeable. In the event that sands are used to improve drainage away from the structure, one should make sure the clay soils below the sands have a positive slope (3 - 5 Percent) away form the structure, since the clay soils control the drainage away from the house.

The author has seen many projects with an apparent positive drainage; however, since the drainage was established with sands on top of the expansive soils the drainage was not effective.

Depressions or water catch basin areas should be filled with compacted soil (sandy clays or silty clays not bank sand) to have a positive slope from the structure, or drains should be provided to promote runoff from the water catch basin areas. Six to twelve inches of compacted, impervious, non-swelling soil placed on the site prior to construction of the foundation can improve the necessary grade and contribute additional uniform surcharge pressure to reduce uneven swelling of underlying expansive soil.

Pets (dogs, etc.) sometimes excavate next to the exterior grade beams and created depressions and low spots in order to stay cool during the hot season. This condition will result in ponding of the surface water in the excavations next to the foundation and subsequent foundation movements. These movements can be in the form of uplift in the area with expansive soils and settlement in the areas with sandy soils. It is recommended as a part of the foundation maintenance program, the owner backfills all excavations created by pets next to the foundation with compacted clay fill.

Grading and drainage should be provided for structures constructed on slopes, particularly for slopes greater than 9 percent, to rapidly drain off water from the cut areas and to avoid ponding of water in cuts or on the uphill side of the structure. This drainage will also minimize seepage through backfills into adjacent basement walls.

Subsurface drains may be used to control a rising water table, groundwater and underground streams, and surface water penetrating through pervious or fissured and highly permeable soil. Drains can help control the water table in the expansive soils.

Furthermore, since drains cannot stop the migration of moisture through expansive soil beneath foundations, they will not prevent long-term swelling. Moisture barriers can be placed near the foundations to minimize moisture migration under the foundations. The moisture barriers should be at least five-feet deep in order to be effective.

Area drains can be used around the house to minimize ponding of the surface water next to the foundations. The area drains should be checked periodically to assure that they are not clogged.

The drains should be provided with outlets or sumps to collect water and pumps to expel water if gravity drainage away from the foundation is not feasible. Sumps should be located well away from the structure. Drainage should be adequate to prevent any water from remaining in the drain (i.e., a slope of at least 1/8 inch per foot of drain or 1 percent should be provided).

Positive drainage should be established underneath structural slabs with crawl space. This area should also be properly vented. Absence of positive drainage may result in surface water ponding and moisture migration through the slab. This may result in wood floor warping and tile unsticking.

It is recommended that at least six-inches of clearing be developed between the grade and the wall siding. This will minimize surface water entry between the foundation and the wall material, in turn minimizing wood decay.

Poor drainage at residential projects in North and West Houston can result in saturation of the surficial sands and development of a perched water table. The sands, once saturated, can lose their load carrying capacity. This can result in foundation settlements and bearing capacity failures. Foundations in these areas should be designed assuming saturated subsoil conditions.

In general, roof drainage systems, such as gutters or rain dispenser devices, are recommended all around the roof line when gutters and downspouts should be unobstructed by leaves and tree limbs. In the area where expansive soils are present, the gutters should be connected to flexible pipe extensions so that the roof water is drained at least 10-feet away from the foundations. Preferably the pipes should direct the water to the storm sewers. In the areas where sandy soils are present, the gutters should drain the roof water at least five-feet away from the foundations.

If a roof drainage system is not installed, rain-water will drip over the eaves and fall next to the foundations resulting in subgrade soil erosion, and creating depression in the soil mass, which may allow the water to seep directly under the foundation and floor slabs.

The home owner must pay special attention to leaky pools and plumbing. In the event that the water bill goes up suddenly without any apparent reason, the owner should check for a plumbing leak.

The introduction of water to expansive soils can cause significant subsoil movements. The introduction of water to sandy soils can result in reduction in soil bearing capacity and subsequent settlement. The home owner should also be aware of water coming from the air conditioning drain lines. The amount of water from the condensating air conditioning drain lines can be significant and can result in localized swelling in the soils, resulting in foundation distress.

<u>Landscaping</u>

<u>General</u>. A house with the proper foundation and drainage can still experience distress if the homeowner does not properly landscape and maintain his property. One of the most critical aspects of landscaping is the continual maintenance of properly designed slopes.

Installing flower beds or shrubs next to the foundation and keeping the area flooded will result in a net increase in soil expansion in the expansive soil areas. The expansion will occur at the foundation perimeter. It is recommended that initial landscaping be done on all sides, and that drainage away from the foundation should be provided and maintained. Partial landscaping on one side of the house may result in swelling on the landscaping side of the house and resulting differential swell of foundation and structural distress in a form of brick cracking, windows/door sticking, and slab cracking.

Landscaping in areas where sandy, non-expansive soils are present, with flowers and shrubs should not pose a major problem next to the foundations. This condition assumes that the foundations are designed for saturated soil conditions. Major foundation problems can occur if the planter areas are saturated as the foundations are not designed for saturated (perched water table) conditions. The problems can occur in a form of foundation settlement, brick cracking, etc.

<u>Sprinkler Systems</u>. Sprinkler systems can be used in the areas where expansive soils are present, provided the sprinkler system is placed all around the house to provide a uniform moisture condition throughout the year.

The use of a sprinkler system in parts of Houston where sandy soils are present should not pose any problems, provided the foundations are designed for saturated subsoil conditions with positive drainage away from the structure.

The excavations for the sprinkler system lines, in the areas where expansive soils are present, should be backfilled with impermeable clays. Bank sands or top soil should not be used as backfill. These soils should be properly compacted to minimize water flow into the excavation trench and seeping under the foundations, resulting in foundation and structural distress.

The sprinkler system must be checked for leakage at least once a month. Significant foundation movements can occur if the expansive soils under the foundations are exposed to a source of free water.

The homeowner should also be aware of damage that leaking plumbing or underground utilities can cause, if they are allowed to continue leaking and providing the expansive soils with the source of water.

Effect of Trees. The presence of trees near a residence is considered to be a potential contributing factor to the foundation distress. Our experience shows that the presence or removal of large trees in close proximity to residential structures can cause foundation distress. This problem is aggravated by cyclic wet and dry seasons in the area. Foundation damage of residential structures caused by the adjacent trees indicates that foundation movements of as much as 3- to 5-inches can be experienced in close proximity to residential foundations.

This condition will be more severe in the periods of extreme drought. Sometimes the root system of trees such as willow, elm, or oak can physically move foundations and walls and cause considerable structural damage. Root barriers can be installed near the exterior grade beams to a minimum depth of 60-inches, if trees are left in place in close proximity to foundations. It is recommended that trees not be planted closer than half the canopy diameter of the mature tree, typically 20-feet from foundations. Any trees in closer proximity should be thoroughly soaked at least twice a week during hot summer months, and once a week in periods of low rainfall. More frequent tree watering may be required.

Tree roots tend to desiccate the soils. In the event that the tree has been removed prior to house construction, during the useful life of the house, or if tree dies, subsoil swelling can occur for several years. Studies have shown that this process can last as much as 20 years in the area where highly expansive clays are present. In the areas where sandy soils are present this process does not occur.

In this case the foundation for the house should be designed for the anticipated maximum heave. Alternatively, the site should be left alone for several years so that the moisture regime in the desiccated area of the soils (where roots used to be) become equal/stabilized to the surrounding subsoil conditions.

Tree removal can be safe provided the tree is no older than any part of the house, since the subsequent heave can only return the foundation to its original level. In most cases there is no advantage to a staged reduction in the size of the tree and the tree should be completely removed at the earliest opportunity. The areas where expansive soils exist and where the tree is older than the house, or there are more recent extensions to the house, it is not advisable to remove the tree because the danger of inducing damaging heave; unless the foundation is designed for the total computed expected heave.

In the areas where non-expansive soils are present, no significant foundation distress will occur as a result of the tree removal.

In the areas where too much heave can occur with tree removal, some kind of pruning, such as crown thinning, crown reduction or pollarding should be considered. Pollarding, in which most of the branches are removed and the height of the main trunk is reduced, is often mistakenly specified, because most published advice links the height of the tree to the likelihood of damage. In fact the leaf area is the important factor. Crown thinning or crown reduction, in which some branches are removed or shortened, is therefore generally preferable to pollarding. The pruning should be done in such a way as to minimize the future growth of the tree, without leaving it vulnerable to disease (as pollarding often does) while maintaining its shape. This should be done only by a reputable tree surgeon or qualified contractor working under the instructions of an arbor culturist.

You may find there is opposition to the removal or reduction of an offending tree; for example, it may belong to a neighbor or the local authority, or have a Tree Preservation Order on it. In such cases there are other techniques that can be used from within your own property.

One option is root pruning, which is usually performed by excavating a trench between the tree and the damaged property deep enough to cut most of the roots. The trench should not be so close to the tree that it jeopardizes its stability. In time, the tree will grow new roots to replace those that are cut; however, in the short term there will be some recovery as the degree of desiccation in the soil under the foundations reduces.

Where the damage has only appeared in a period of dry weather, a return to normal weather pattern may prevent further damage occurring. Permission from the local authority is required before pruning the roots of a tree with preservation order on it.

Root barriers are a variant of root pruning. However, instead of simply filling the trench with soil after cutting the roots, the trench is either filled with concrete or lined with an impermeable layer to form a "permanent" barrier to the roots. Whether the barrier will be truly permanent is questionable, because the roots may be able to grow round or under the trench. However, the barrier should at least increase the time it takes for the roots to grow back.

Foundations/Flat Works

Every homeowner should conduct a yearly observation of foundations and flat works and perform any maintenance necessary to improve drainage and minimize infiltrations of water from rain and lawn watering. This is important especially during the first six years of a newly built home because this is usually the time of the most severe adjustment between the new construction and its environment. We recommend that all of the separations in the flat work and paving joints be immediately backfilled with joint sealer to minimize surface water intrusion and subsequent shrink/swell.

Some cracking may occur in the foundations. For example, most concrete slabs can develop hairline cracks. This does not mean that the foundation has failed. All cracks should be cleaned up of debris as soon as possible. The cracks should be backfilled with high-strength epoxy glue or similar materials. If a foundation experiences significant separations, movements, cracking, the owner must contact the builder and the engineer to find out the reason(s) for the foundation distress and develop remedial measures to minimize foundation problems.

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Control Group Member, ASCE Standards Committee on the Design of Residential Foundations on Expansive Clays, 1992-present
Control Group Member, ASCE Standards Committee on Independent Peer Review, 1992-present

PROFESSIONAL ACTIVITIES (Cont'd)

Member, General Design Subcommittee, Southern Building Code Congress International, 1986-1988
Chairman, Transportation Research Board Committee A2L06 Environmental Factors Except Frost, 1987-1993
Organizing Committee, Seventh International Conference on Expansive Soils, Dallas, Texas, August, 1992
Organizing Committee, First International Conference on Unsaturated Soils, Paris, France, September, 1995
U.S. Representative on Committee TC-6, International Society of Soil Mechanics and Foundation Engineering, 1987-present
Secretary, Fourth International Conference on Expansive Soils, Denver, Colorado, June, 1980
Secretary, American Society of Civil Engineers Research Council on Expansive Soils
Transportation Research Board Committees: A2L06, Environmental Factors Except Frost; A2B01, Pavement Management Systems; A2B04, Pavement Rehabilitation; Task Force A2T59, Relating Distress to Performance; Task Force A2T56, Non-Destructive Testing of Airfield Pavements
American Concrete Institute Committee 360
Post Tensioning Institute Technical Advisory Board
Publication Advisory Board, International Journal for Numerical and Analytical Methods in Geomechanics, John Wiley and Sons

BRIEF BIOGRAPHICAL SKETCH

Robert L. Lytton was born in Port Arthur, Texas on October 23, 1937, a descendant of a family which came to Texas as part of Stephen F. Austin's Little Colony (1828) and contributed several soldiers to the Texan army which won Texas' independence in the battle of San Jacinto (April 21, 1836) over the Mexican Army of Operations under President Santa Anna. He attended high school in San Antonio, Texas, and graduated from the University of Texas at Austin in June, 1960 with a Bachelor of Science degree in Civil Engineering. He received the College of Engineering Hamilton Watch Award, given to the graduating senior with the highest grade average. He completed a Master of Science degree in August, 1961 as a Graduate Fellow of the National Science Foundation and was inducted into the Friar Society of the University of Texas which elects twelve students each year. He spent two years on active duty with the U.S. Army 35th Engineer Construction Group from 1961 to 1963 during the Cuban missile crisis and the beginning of the war in Vietnam. After another two years working with a consulting civil engineer in Houston, Texas, he returned to the University of Texas once more as a Fellow of the National Science Foundation. He completed his Ph.D. degree in August, 1967 and served as an Assistant Professor at the University of Texas in 1967-68.

A Post-doctoral Fellowship from the National Science Foundation permitted him to spend the next two years engaged in research on foundations on expansive soils with the Australian commonwealth Scientific and Industrial Research Organization Division of Applied Geomechanics. Returning to the United States in 1971 he entered the faculty at Texas A&M University, rising to the rank of Professor in 1976 and being awarded the A.P. and Florence Wiley Chair in Civil Engineering in 1990, and the F. J. Benson Chair in Civil Engineering in 1995. His professional interests are in Expansive Clay Theory and Design; Soil Mechanics; Soil-Structure Interaction; Soil Dynamics; Continuum Mechanics; Fracture Mechanics; Non-destructive Testing of Pavements; Pavement Analysis, Design, and Management; and Sampling, Statistical Methods, and Reliability.

BRIEF PROFESSIONAL BIOGRAPHICAL SKETCH

Dr. Lytton recently completed a project for the Federal Highway Administration to develop an integrated model to predict environmental effects beneath pavements. The analytical method developed uses coupled heat and moisture flow and predicts suction and temperature, freezing and thawing, and frost heave beneath pavements. The calculated results were compared favorably with field measurements made in College station, Texas; Amarillo, Texas; and Deland, Illinois. The model was used extensively in several of the SHRP Asphalt and Long-Term Pavement Performance programs.

He is the author of Chapter 13 of the textbook, "Numerical Methods in Geotechnical Engineering," (McGraw-Hill). The chapter is titled, Foundations in Expansive Soils. He teaches a graduate course in Civil Engineering at Texas A&M University on the same subject.

His doctoral dissertation was on water movement in expansive soils. His two-year period of study in 1969-70 as a Post-Doctoral Fellow of the National Science Foundation was with Dr. Gordon Aitchison of the Australian Commonwealth Scientific and Industrial Research Organization (CSIRO) Division of Applied Geomechanics on the subject of expansive soils.

In 1976-78, he conceived and supervised the research project at Texas A&M for the Post-Tensioning Institute which resulted in the publication of the manual on the Design and Construction of Post-Tensioned Slab-on-Ground which he coauthored. The design procedure contained in that manual has been incorporated verbatim into the Southern Building Code, the Uniform Building Code, and American Concrete Institute Report ACI 360R-92 on Design of Slabs on Grade.

In 1984, his pioneering work in expansive soils and foundation design was recognized by the South African Council for Scientific and Industrial Research which honored him with the Everite Bursary Award which is given to one person each year by that country and is an award of the highest distinction.

His lectures in Central and South America on the same subject are credited with having begun the highly creative and energetic research and engineering design presently being accomplished in Columbia and Mexico.

He has been a member of ACI Committee 360 on Slabs-on-Ground, and is currently a member of the Post-Tensioning Institute Technical Advisory Board, and the Southern Building Code Congress General Design Subcommittee.

Together with Dr. Chris Mathewson, of the Texas A&M University, Department of Engineering Geology, he conducted a three-year long project for the National Science Foundation to survey the damage done by expansive soils to houses in five cities in Texas: Beaumont, College Station, Amarillo, San Antonio and Waco. He developed regression analysis models of the causes of damage in each city. Each survey had at least 100 residences and a total of 700 residences were surveyed. He developed a method of modifying the Post-Tensioning Institute design of stiffened slabs to account for the variability of site conditions using a risk analysis approach.

His experience in field, laboratory, and analytical studies and his proven record of organizing and successfully completing projects which are both complex and highly significant in their impact all contribute to his well earned international reputation for creative advances in the analysis and design of foundations and pavements on expansive soils.

He was the keynote speaker at the 7th International Conference on Expansive Soils which was held in Dallas, Texas in August, 1992. He has been the United States representative on the International Society of Soil Mechanics and Foundation Engineering Technical Committee TC-6 since 1989. He presented the keynote address in the area of foundations and pavements to the 1st International Conference on Unsaturated Soils which was held in Paris, France in September, 1995. Recently, he presented the keynote address on the same subjects to the 3rd International Symposium on Unsaturated Soils in Rio de Janeiro, Brazil in April, 1997.

APPENDIX B: SOIL SUCTION CONVERSION FACTORS

1 Bar	=	0.987 Atmospheres (Atm)
	=	14.503 Pounds/square inch (psi)
	=	1,019.784 Centimeters of water (cm H_20)
	=	100.000 Kilopascals (kPa)
	=	1.0×10^6 Dynes/square centimeter (dynes/cm^2)
1 Atm	=	1.013 Bars
	=	14.695 psi
	=	1,033.296 cm H_20
	=	101.325 kPa
	=	1.013×10^6 dyne/cm^2
1 cm H_20	=	9.806×10^{-4} Bars
	=	9.678×10^{-4} Atm
	=	1.422×10^{-2} psi
	=	9.806×10^{-2} kPa
	=	9.806×10^{2} dyne/cm^2
1 psi	=	6.895×10^{-2} Bar
	=	6.805×10^{-2} Atm
	=	70.314 cm H_20
	=	6.895 kPa
	=	6.895×10^{4} dyne/cm^2
1 kPa	=	1.000×10^{-2} Bars
	=	9.869×10^{-3} Atm
	=	0.145 psi
	=	10.198 cm H_20
	=	1.000×10^{-4} dyne/cm^2

Indirect Measurement of Soil Suction

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Abstract. This paper reports on indirect soil suction measurement methods. Indirect suction measurement techniques measure the moisture equilibrium condition of the soil instead of suction. The moisture equilibrium condition of the soil can be determined by primary means as in vapor pressure, secondary means as through another porous medium or tertiary means as in measuring other physical properties of the porous medium that indicates its moisture equilibrium condition. Indirect suction measurement technique employing primary means include thermocouple psychrometers, transistor psychrometer and chilled-mirror psychrometer. Indirect suction measurement technique employing secondary means includes the filter paper method and indirect suction measurement technique employing tertiary means includes the thermal conductivity sensors and electrical conductivity sensors. These techniques have been widely used in engineering practice and in research laboratories. However, each of these techniques has its own limitations and capabilities, and active research into improving these techniques is still ongoing in the universities, research laboratories, and private sector. This report outlines working principle, calibration, measurement, and application areas of these methods. The report is based on the most recent literature and practice.

Key words. soil suction, thermocouple, transistor, chilled-mirror, psychrometer, filter paper, thermal conductivity, electrical conductivity.

1. Introduction

The understanding and wide acceptance of unsaturated soil mechanics principles has seen a gradual change in geotechnical engineering practice. There is more than ever a greater need for reliable soil suction measurement techniques as soil suction becomes an integral part of engineering practice in many situations involving unsaturated soils. Soil suction is a result of capillary action and ionic concentration of the pore water. Total suction results when both mechanisms are active. Matric suction results when only capillary action is active. Significant contributions have been made by geotechnical engineers in the measurement of soil suction. However, there is still need for research into the measurement of both matric and total suction in the laboratory and in the field. Almost all suction measurement methods have shortcomings with regard to one or more aspects, such as the range of application, cost, reliability, and practicality. For instance, temperature control is essential for suction measurement methods that rely on vapor pressure measurements. At low suction levels or high relative humidity, very small changes in relative humidity result in very large changes in suction. It is this narrow range of relative humidity that most total suction inferring devices are affected by minor temperature fluctuations. Total suction measurements from psychrometers are in great error once the suction drops below 1000 kPa.

This paper reviews indirect suction measurement techniques based on the means of measuring moisture equilibrium condition of the soil. Measurement techniques such as time-domain reflectometry (TDR) method which measure moisture content of the soil from which suction can be inferred if the soil water characteristics curve of the soil is available are not examined in this paper. Indirect measurement techniques employing primary means measure the vapor pressure. Included in the primary methods are thermocouple psychrometer, transistor psychrometer, and chilled-mirror psychrometer. Indirect measurement techniques employing secondary means measure the moisture equilibrium condition of another porous medium. An example of the secondary method is the filter paper method. Indirect measurement techniques employing tertiary means measure other physical properties of the other porous medium's moisture equilibrium condition. Examples of tertiary methods of indirect suction measurement are thermal conductivity sensors and electrical conductivity sensors. Each of these techniques has its own capabilities and limitations, and they may be used for complementing each other for different ranges of suction. The paper summarizes basic working principle, calibration, measurement, and application areas of indirect soil suction measurement methods based on the most recent literature. A critical evaluation of the capabilities, limitations, and pitfalls of these methods is also presented.

2. Primary Methods

Total suction of a soil sample may be inferred from measurements within the vapor phase that is in equilibrium with the sample. Devices that measures relative humidity can be employed to measure total suction. Thermocouple psychrometer, transistor psychrometer, and chilled-mirror psychrometer are examples of such devices.

2.1. THERMOCOUPLE PSYCHROMETERS

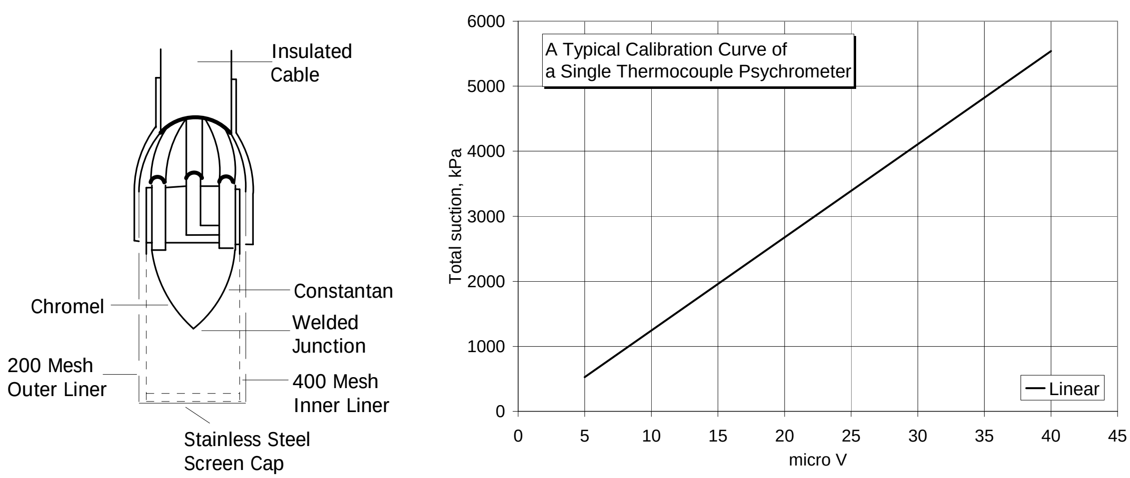
There are two types of thermocouple psychrometer for determining total suction measurements of soils: the wet-loop type sensor described by Richards and Ogata (1958) and the Peltier type sensor described by Spanner (1951). The wet-loop sensor is only used with the psychrometric measurement technique, whereas the Peltier sensor can be used with both the psychrometric and hygrometric measurement methods. The primary difference between these two sensors is the manner by which water is applied to the sensing junction. The wet-loop sensor is wetted by manually placing a drop of water on a small ring that is at the sensing junction. The wet-loop type sensor technique has been improved in a new psychrometer device by replacing the wet and dry thermometer bulbs with the wet and dry transistors. This new device is called transistor psychrometer and it is discussed in the next section.

Two important principles underlie the usefulness of Peltier type thermocouple psyhcrometers: the Seebeck effect and the Peltier effect. The Seebeck effect is the phenomenon that permits a thermocouple to be used for temperature measurement. A thermocouple is formed when two different metals are joined together (Figure 1(a)). If both ends of the wire are joined to form a closed loop, electrical current will flow through the wires whenever the junctions are at different temperatures. The magnitude of the voltage produced is dependent upon the temperature difference between the junctions. The Peltier effect is the phenomenon which allows a thermocouple junction to be cooled by passing an electrical current through the junction. When current flows across the junction of two dissimilar metals, heat will be either absorbed or released at the junction. If the current flows in the same direction as the current produced by the Seebeck effect at the hot junction, heat is absorbed. If the current flows in the opposite direction, heat is released.

Wescor, Inc. and Campbell Scientific, Inc. developed two thermocouple psychrometer methods for the measurement of equilibrium relative humidity from which total suction can be determined. These are the psychrometric (wet bulb) and the hygrometric (dew point) methods. Thermocouple psychrometers that are commercially available from Wescor are PST-55 stainless steel and PCT-55 ceramic cup. The PST-55 sensor has a non-removable stainless steel shield, which has a larger pore size. The PCT-55 sensor has a removable ceramic shield. The same sensors are used for either method but the electronic control and measuring apparatus operate differently. The Wescor HR-33T is a single-channel datalogger and can be used to determine the total suction of a sample using either dew point (hygrometric) or wet bulb (psychrometric) methods. The Wescor/Campbell CR7 datalogger and the new Wescor datalogger PSYPRO use only the psychrometric method. The PSYPRO data logger has 8 channels. The CR7 series data logger has several channel configurations (14, 28, 40, 70 and 140 channels). Using either method with any of the instruments, a cooling current is used to cool the thermocouple junction below the dew point of the air surrounding the sample causing water to condense on the junction. Using the dew point method, the mode of the HR-33T is switched to dew point and the thermocouple junction is kept at the dew point temperature using the duty cycle of the HR-33T. Water evaporation and condensation is equilibrated and a voltage is created. This voltage is converted to total suction using standard salt solutions in the case of calibration or using the calibration curve (Figure 1(b)) in the case of total suction measurements. The microvolt output from a thermocouple psychrometer is very sensitive to ambient temperature fluctuations. The cooling coefficient of the sensor must be matched to the duty cycle of the HR-33T for accurate measurements. Using the psychrometric method, after the cooling current has ceased, the water begins to evaporate from the thermocouple junction which creates a voltage. Initially, the rate at which the water evaporates from the thermocouple is approximately constant and is called the psychrometric plateau. A measurement of voltage is taken over this constant water evaporation rate period and converted to total suction using standard salt solutions in the case of calibration or using the calibration curve (Figure 1(b)) in the case of total suction measurements. This method does not require setting the cooling coefficient.

Careful cleaning and thorough drying of the psychrometers before and after calibration and soil total suction measurements are essential to reliable instrument performance. Contaminants, such as salts, can affect cooling, evaporation, and microvolt output. The pore size of the protective housing on the thermocouple psychrometer prevents most of contaminants, such as soil particles, from entering the sensor cavity. The most serious contamination occurs if dissolved contaminants migrate through or accumulate on the protective housing. Psychrometers can be cleaned with distilled or deionized water. Most of the excess water can be removed by shaking and blowing dry air to ensure that water is removed and sensors are dry. A range of sodium chloride (NaCl) and potassium chloride (KCl) solutions of known osmotic suctions is typically used to establish the relation between total suction and microvolt output. Typical NaCl solution concentrations versus their osmotic suction values are given in Table 1. Calibration solutions are chosen to cover the anticipated range of total suction to be measured. Correct calibration of thermocouple psychrometers is extremely important because the accuracy of all subsequent measurements and interpretations will be based on these data. For routine measurements across the entire range, a minimum of four calibration solutions are typically selected to characterize each psychrometer's response to changes in total suction at a given temperature. Thermocouple psychrometers are typically calibrated by direct immersion into a small container of calibration solution or by suspension of the sensor over the solution in the container. The immersion method has often been selected because this configuration helps to control the temperature fluctuations better. The disadvantage with the immersion method is the possibility of salts getting on the sensors. Sensors should be immersed at a fairly shallow depth otherwise there will be an added pressure component that may force the solution getting on the sensors. The pore size of typical screen-cage and ceramic-cup housing is sufficiently small to prevent liquid from entering the air-filled sensor cavity (Pinnock, 2005). A water bath is usually employed to maintain temperature stability. Under isothermal conditions, the equilibration between thermocouple psychrometer sensor and vapor pressure from the salt solution is usually established within an hour. The resulting microvolt readings from a psychrometer connected to a datalogger are plotted against corresponding osmotic suctions of the salt solutions to obtain a calibration curve (Figure 1(b)). The practical range over which total suction measurements can be made with thermocouple psychrometers is between about 300 kPa and 7000 kPa. Total suction values between about 300 kPa and 500 kPa should be evaluated very carefully since this is the range most affected by very small temperature fluctuations. Suction values below 300 kPa should be carefully evaluated for validity.

Application of thermocouple psychrometers to infer total suction of unsaturated soils in geotechnical engineering research and practice has greatly broadened in the recent years. In one recent application, Bulut et al. (2005) monitored the total suction change of cylindrical Shelby tube soil samples over time with thermocouple psychrometers to determine unsaturated soil moisture diffusion coefficients.



(a) (b)

Figure 1. (a) Schematic drawing of a thermocouple psychrometer and (b) a typical calibration curve.

Table 1. Osmotic suction of NaCl solutions at 25^{o}C (from Hamer and Wu, 1972; Bulut et al., 2001).

Molality (m)	Osmotic Coefficient (ϕ)	Osmotic Suction (kPa)	Molality (m)	Osmotic Coefficient (ϕ)	Osmotic Suction (kPa)
0.000	1.00000	0.00	0.300	0.92123	1370.19
0.002	0.98402	9.76	0.500	0.92224	2286.15
0.005	0.97604	24.20	0.700	0.92691	3216.82
0.010	0.96804	47.99	0.900	0.93350	4165.31
0.020	0.95832	95.02	1.200	0.94567	5626.15
0.050	0.94357	233.90	1.600	0.96487	7653.84
0.100	0.93250	462.32	2.200	0.99818	10887.35
0.200	0.92387	916.08	2.600	1.02263	13182.03

2.2. TRANSISTOR PSYCHROMETER

Transistor psychrometer consists of a thermally insulated container that holds the psychrometer probes and a datalogger for the measurement and recording of the output. The instrument is very similar in operation to the thermistor psychrometer (Woodburn et al., 1993). Thermistor psychrometers are different from thermocouple psychrometers in that they require a water drop to be manually placed on the wet bulb temperature sensor. The transistor psychrometer is an electronic wet and dry bulb thermometer in which a wet and dry transistor probe is used instead of wet and dry thermometer bulbs as in thermistor psychrometers to measure the relative humidity of the air space in equilibrium with a soil sample. The temperature depression of the wet transistor, which holds a standard-size water drop, is measured with the sensors in the probe (Figure 2(a)). The wet and dry transistors are employed as heat sensors and the voltage output from the probe is used to infer the total suction. The dry bulb transistor has the characteristics of the wet bulb transistor, and it is used as a reference sensor for temperature and vapor pressure measurements.

Improvements in performance have been made and the device can measure a much wider range of total suction, from about 100 kPa to about 10000 kPa. Much of the improvement is due to calibration procedure and advances in micro-chip technology (Woodburn et al., 1993). Transistor psychrometer improves on the thermistor or thermocouple psychrometer in that it has a larger suction measurement range. The range and accuracy in the measurements are also attributed to the sensitivity of the transistors to very small changes in temperature. Soil Mechanics Instrumentation (Woodburn et al., 1993; www.smi-unsat.com) produces two types of thermally insulated containers for the transistor probes: 12-probe unit and 8-probe unit. The 8-probe psychrometer is equipped with an insulated lid for better temperature control. Each probe can measure total suction in about one hour. Twelve and eight soil total suction measurements can be made within an hour with the 12- and 8-probe units, respectively.

Prior to any measurements of total suction with the psychrometer, the wet and dry sensors at the tip of the probe should be cleaned carefully with distilled water and thoroughly dried. Before applying the standard size water drop, the sleeve on the wet transistor should be checked for its specified height to hold the water drop.

The calibration of the psychrometer probes, determined from the relationship between microvolt output from the transistor and a known osmotic suction value of a salt solution (Table 1), should be carried out carefully. NaCl solutions covering the measurement range of the transistor psychrometer sensors (e.g., from about 100 kPa to about 10000 kPa) are prepared to obtain calibration curves for each probe. A typical calibration curve of a transistor psychrometer probe is depicted in Figure 2(b). The calibration curve can be affected by several factors: temperature fluctuations, hysteresis, and drop size. The transistor probes are first equilibrated for at least 4 hours at zero total suction over distilled water and the output is adjusted to the initial zero reading before any calibration process or soil suction measurements. Afterwards, the different voltage outputs are recorded from the datalogger following one hour equilibration period for each suction level, in order to avoid any hysteresis effects. The relationship between relative humidity and total suction (e.g., Kelvin equation) is used to determine the soil total suction. The thermally insulated container provided for the probes maintains a fairly constant temperature during the period of the test. Greater accuracy and reproducibility of results is obtained in a room where temperature is controlled to about $\pm$ 0.5^{o}C (Woodburn et al., 1993).

Transistor psychrometers have been used in many universities and geotechnical engineering laboratories around the world. In Australia and New Zealand this instrument has been used for unsaturated expansive soils applications (Woodburn, 2005). It has practically replaced thermocouple psychrometers in many laboratory soil suction measurements. Recent studies by Bulut et al. (2000, 2002, 2005) show that transistor psychrometer has a better capability of measuring total suction at lower levels when compared with other psychrometric methods. Another promising psychrometer that has been used for measurement of total suction is the polymer capacitance sensor which consists of two electrodes separated by a film of thermoset polymer that absorbs or releases water as the relative density of the surrounding air changes (Albrecht et al., 2003).

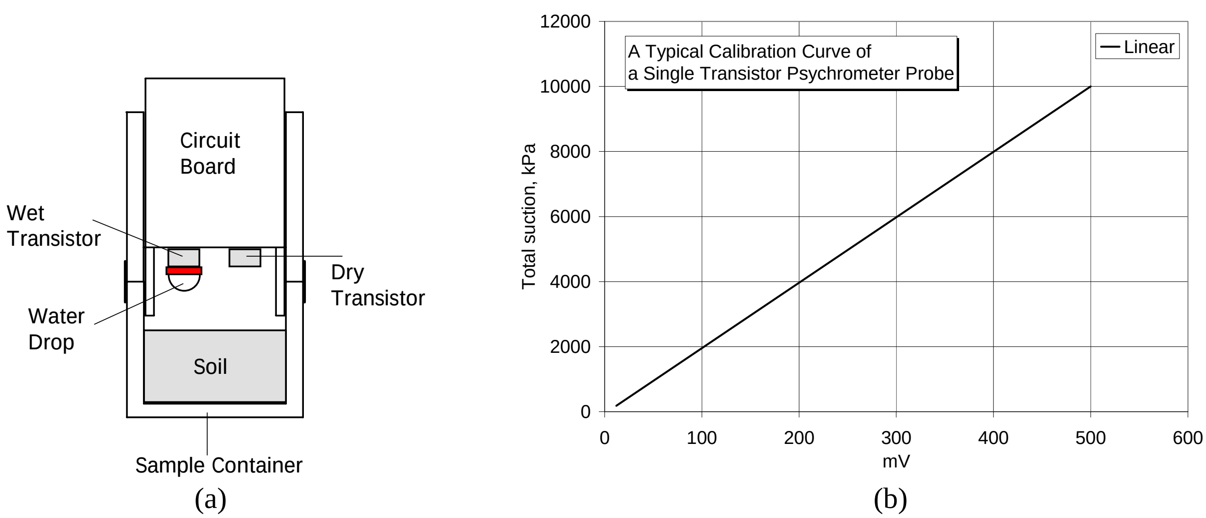


Figure 2. (a) Schematic drawing of a transistor psychrometer probe and (b) a typical calibration curve.

2.3. CHILLED-MIRROR PSYCHROMETER

Chilled-mirror psychrometer uses the chilled mirror dew point technique to measure total suction under isothermal conditions in a sealed container (Figure 3). The chilled-mirror psychrometer discussed in this paper is a product of Decagon Devices, Inc. and is known as WP4 Dew Point Potentiameter (source: Decagon Devices, Inc. website www.decagon.com). Measurement of total suction with the WP4 is based on equilibrating the liquid phase of the water in a soil sample with the vapor phase of the water in the air space above the sample in a sealed chamber. A

Peltier cooling device is used to cool the mirror until dew forms and then to heat the mirror to eliminate the dew. Temperature of the sample is measured with an infrared thermometer. An optical sensor is also employed to detect the dew formed on the mirror. A thermocouple attached to the chilled mirror measures the dew point temperature. A small fan is also employed to circulate the air in the sensing chamber and speed up vapor equilibrium. Both the dew point and soil sample temperature are then used to determine the relative humidity above the soil sample within the closed chamber.

The device determines the dew point temperature repeatedly until water vapor equilibrium is reached between the soil and the air in the chamber. The chilled mirror technique offers a fundamental characterization of humidity in terms of the temperature at which vapor condenses. Therefore, the calibration of the instrument with different concentrations of salt solutions is not necessary. However, the performance of the instrument should be checked prior to total suction measurement by measuring the total suction of a salt solution with a known osmotic potential (WP4 User Manual, Decagon Devices, Inc.). When the temperature readings have stabilized, the instrument will determine the relative humidity of the enclosed space above the soil sample and will display the total suction of the sample on a digital screen. Temperature control is very important. The measured difference between dew point and sample temperatures must be kept small. The WP4 chilled-mirror pyschrometer is a very robust instrument that is suitable for rapid total suction measurements, usually less than 10 minutes. Detailed measurement procedures are provided in the user's manual. If the instrument readings are offset from standard solution readings, the linear offset of the meter should be corrected. It is important to avoid contamination of the instrument. The sample cup should be filled to less than full capacity to minimize the potential of contaminating the chamber. If necessary, the mirror and fan can be cleaned according to procedures outlined in the user's manual. Because of the high precision of the WP4 chilled-mirror pyschrometer, annual maintenance is required.

Bulut et al. (2002) developed a complete characteristic curve for the WP4 instrument using the relationship between osmotic suction and salt solution concentration. Figure 4(a) shows the characteristic curve for this instrument. In order to interpret the sensitivity portion of the characteristic curve more clearly, Figure 4(b) was developed from Figure 4(a) by magnifying the lower portion of Figure 4(a) between salt solution molality of 0.0 and 0.5. Figure 4(b) shows more clearly that once osmotic suction falls below about 1000 kPa the scatter in suction data increases. The total suction measurements with the WP4 psychrometer should be considered as error once suction falls below 100 kPa (Wacker, 2002). Bulut et al. (2002) evaluated the accuracy of the chilled-mirror psychrometer by comparing the results of the total suction measurements of undisturbed soil samples with the filter paper method. Bulut et al. (2002) found that the degree of error associated with the WP4 psychrometer is higher than with the filter paper method at low suction levels, but very good correlation between the two methods at high total suction levels.

Leong et al. (2003) evaluated the accuracy of a chilled mirror dew point device using compacted soil samples. A thorough calibration of the instrument using several standard salt solutions was performed. The equilibration time during calibration and total suction measurement was short, less than 15 minutes. The total suction measurements on the compacted samples were compared to the sum of matric and osmotic suctions of the same soils that were measured independently. The matric suction of the soils was measured with the null-type axis-translation apparatus and the osmotic suction of the samples was measured from the soil water solution obtained from a pore fluid squeezer device. The electrical conductivity measurements were performed on the extracted solutions to infer the osmotic suction of the soil samples. The test results showed that total suctions obtained using the chilled mirror dew point device were always greater than the sum of the matric and osmotic suctions measured independently. To reconcile the discrepancies between the sum of the matric and osmotic suctions and the total suction from the chilled mirror dew point device, a correction equation for the total suction was suggested. In ASTM D6836-02, the chilled-mirror hygrometer is used in Method D for determining the desorption soil water characteristic curve for suction range above 1000 kPa as the limitation of the chilled-mirror hygrometer for low suction level is recognized.

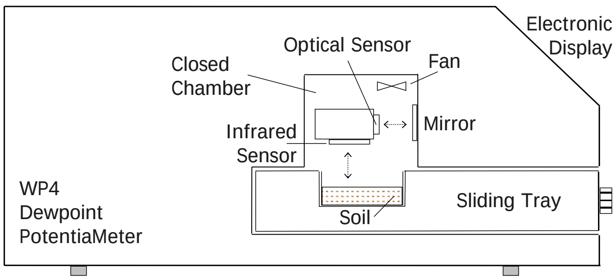


Figure 3. Schematic drawing of the WP4 chilled-mirror psychrometer.

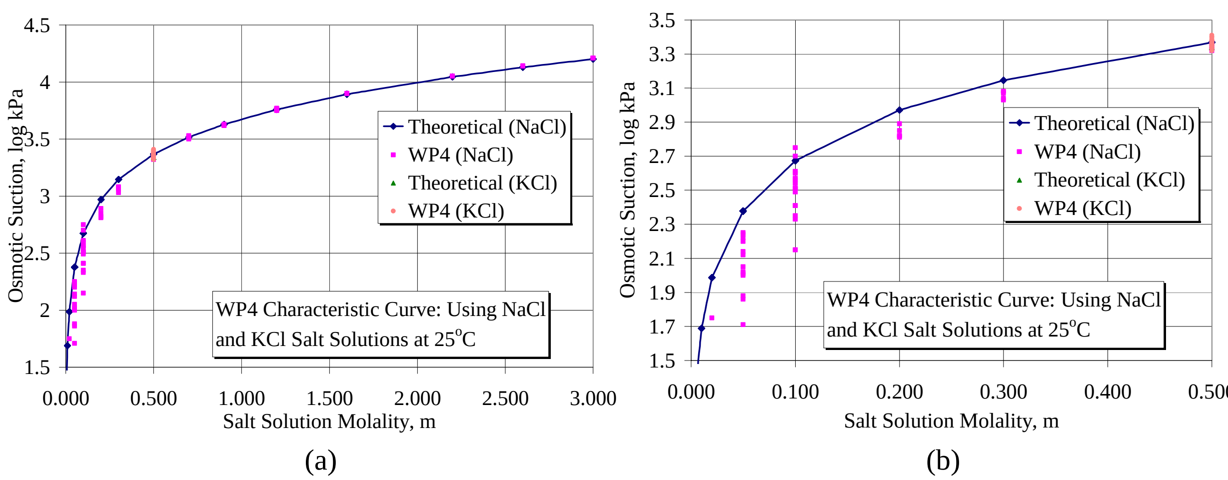


Figure 4. The characteristic curve of the WP4 chilled-mirror instrument; (a) the whole practical suction range and (b) the magnified lower range of suction in (a) (from Bulut et al., 2002).

3. Secondary Methods

Secondary methods employed another medium to achieve moisture equilibrium with the soil where suction measurement is required. Sibley and Williams (1990) evaluated five kinds of absorbent materials for suction measurement in the range 0 to 200 MPa. The sensitivity of the five materials varies with the suction range as shown in Table 2. Sibley and Williams (1990) recommended that the Whatman No. 42 filter paper is the most appropriate over the entire range of suction investigated. Filter paper has become the de-facto material for suction measurement. Soil suction measurement procedure using filter paper is outlined in detail in ASTM D 5298-94, Lee (1991), Houston et al. (1994), and Bulut et al. (2001).

Table 2. Sensitivity of absorbent materials (data from Sibley and Williams 1990).

Material	Range of higher sensitivity
Whatman No. 42	0 to 30 MPa
Unwashed dialysis tubing	10 MPa to 100 MPa
Washed dialysis tubing	1000 kPa to 10 MPa
Millipore MF 0.025 μm	30 kPa to 300 kPa
Millipore MF 0.05 μm	30 kPa to 1000 kPa

3.1. THE FILTER PAPER METHOD

Among all the known suction measurement methods, the filter paper technique is the only method from which both total and matric suctions can be inferred. In the filter paper method, the soil specimen and filter paper are brought to moisture equilibrium either in a contact (matric suction measurement) or in a noncontact (total suction measurement) method in a constant temperature environment (Figure 5). Direct contact between the filter paper and the soil allows water in the liquid phase and solutes to exchange freely, whereas separation between the filter paper and the soil by a vapor barrier limits the water exchange to the vapor phase only and prevents solute movement. After equilibrium is established between the filter paper and soil the water content of the filter paper is measured. Then, by using the appropriate filter paper calibration curve, the suction of the soil is estimated. The calibration curves are usually obtained from the processes of wetting and drying the filter papers through vapor transfer (from salt solutions) and drying and wetting the filter papers through fluid transfer (pressure plate type instruments). Salt solutions are employed in the noncontact method for measuring total suction and porous plates are usually used in the contact method for measuring matric suction. The filter paper method is the only known method that covers the widest range of suction. However, it is also considered as one of the most controversial techniques among the practitioners and researchers. There are still many concerns about the reliability of the filter paper method. The filter paper method is a simple technique and can be reliable if the basic principles of the method are understood and a strictly practiced laboratory protocol is carefully followed.

As accuracy of the filter paper technique is dependent on the accuracy of the filter paper water content versus suction calibration curve, the calibration technique of the filter paper method has been investigated by numerous researchers. Different aspects of the method by using different types of filter papers, measuring devices, and experimental techniques to calibrate the filter paper and to infer suction of the soil sample (e.g. Leong et al. 2002). Calibration equations should be developed specifically for the filter papers being used. The measurement procedure for calibration is outlined in detail in ASTM D 5298-94, Lee (1991), Houston et al. (1994), and Bulut et al. (2001). The most commonly used filter papers are Whatman No. 42 and Schleicher & Schuell No. 589-WH. The Schleicher & Schuell No. 589-WH filter paper is now called grade 989-WH in the US. The reason for this name change in the US is that in Europe the grade name 589-WH is used for a filter paper that has different specifications to the US version (Reeves, 2003). This means that prior to year 2003 the calibration curves that were produced from Schleicher & Schuell No. 589-WH filter papers originated from the US and Europe have different specifications and thus different calibration curves.

Bulut et al. (2001) developed two calibration curves for Schleicher & Schuell No. 589-WH filter papers: one by the process of wetting from initially dry filter papers through vapor flow using NaCl solutions and one by the process of drying from initially saturated filter papers through fluid flow using pressure plates and membranes. Leong et al. (2002) developed different calibration curves for total and matric suctions for Whatman No. 42 and Schleicher & Schuell No. 589-WH filter papers. The calibration curves constructed by Leong et al. (2002) are given in Figure 6. In a more recent study, Bulut and Wray (2005) re-evaluated the filter paper method based on a new calibration curve and the most recent published literature. Until Houston et al. (1994) all suction measurements were based on a single calibration curve. Houston et al. (1994) developed two calibration curves for Fisher quantitative coarse (9.54 A) filter paper: one for total suction and one for matric suction and reported that the curves were not compatible.

The differences in the filter paper calibration curves in the literature are attributed to several factors such as the suction source for the calibration, thermodynamic definitions of suction components, and equilibration time (Fredlund and Rahardjo, 1993; Houston et al., 1994; Bulut et al., 2001; Leong et al., 2002; Bulut and Wray, 2005; Walker et al., 2005). Walker et al. (2005) evaluated total suction measurements of a soil sample using transistor psychrometer and filter paper method. Walker et al. (2005) adopted the filter paper calibration curve in Hamblin (1981) and found that total suction measurements from the filter papers were significantly smaller than the total suction measurements from the transistor psychrometer. Walker et al. (2005) suggest that the total suction calibration curves represent a transient condition during calibration period and that a unique, single calibration curve should be used for both total and matric suction measurements. In other words, Walker et al. (2005) suggest that if sufficient time is allowed for equilibration, the total suction calibration curve will tend towards the matric suction calibration curve. Bulut et al. (2001) and Bulut and Wray (2005) state that a single calibration curve based on water vapor measurements is adequate for both total and matric suction measurements. Leong et al. (2002) and Bulut and Wray (2005) discuss in detail the different calibration curves of filter papers and the time required for equilibration. The laboratory testing protocol adopted by Bulut and Wray (2005) shows that suction measurements as low as 50 kPa can be made reliably using the wetting calibration curve (Figure 7(a)).

It is extremely important to minimize temperature gradients during the calibration with salt solutions as well as during total suction measurement. During calibration, it is suggested that temperature fluctuations should be maintained within an accuracy of $\pm 0.1^{o}C$ or better. It would be ideal to maintain a similar accuracy during total suction measurements, but it may be difficult to obtain such accuracy in a geotechnical engineering laboratory. Therefore, this accuracy may be relaxed to some degree. However, temperature fluctuations should not be more than $\pm 1^{o}C$ in the laboratory. Temperature fluctuations are extremely critical at high relative humidity levels. Bulut and Wray (2005) describe the sensitivity of suction at high relative humidity levels and illustrated that minor changes in relative humidity result in very large changes in suction. For instance, relative humidity values of 0.999656 and 0.999063 result in osmotic suction values of 47.199 kPa and 128.6 kPa, respectively. Filter papers should also be allowed to equilibrate for a sufficient time period. Recent literature suggests that an upper limit of equilibration time of 14 days is sufficient for filter paper calibration over salt solutions and distilled water, and one week of equilibrium period is usually considered satisfactory for most soil suction measurements.

The above discussions for filter paper calibration with salt solutions are also applicable to total suction measurements. Matric suction measurements using filter paper method are also similar to the total suction measurements except that an intimate contact should be provided between the filter paper and the soil (Figure 5).

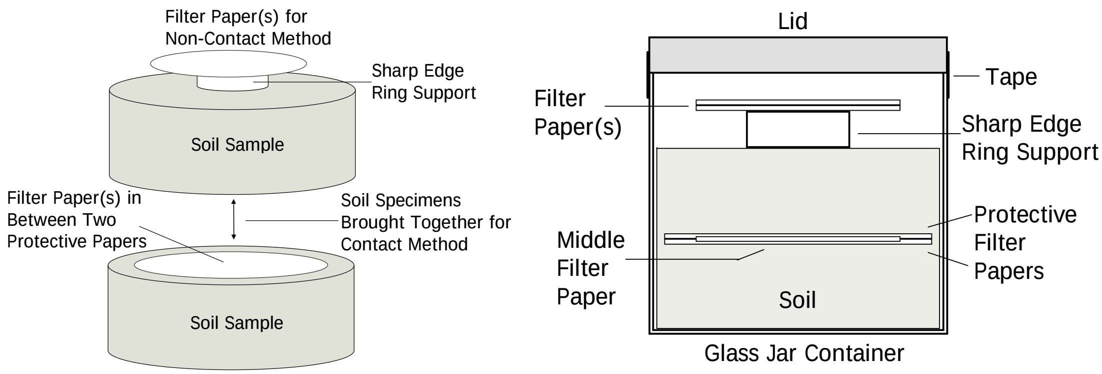


Figure 5. Schematic drawings of soil total and matric suction measurements.

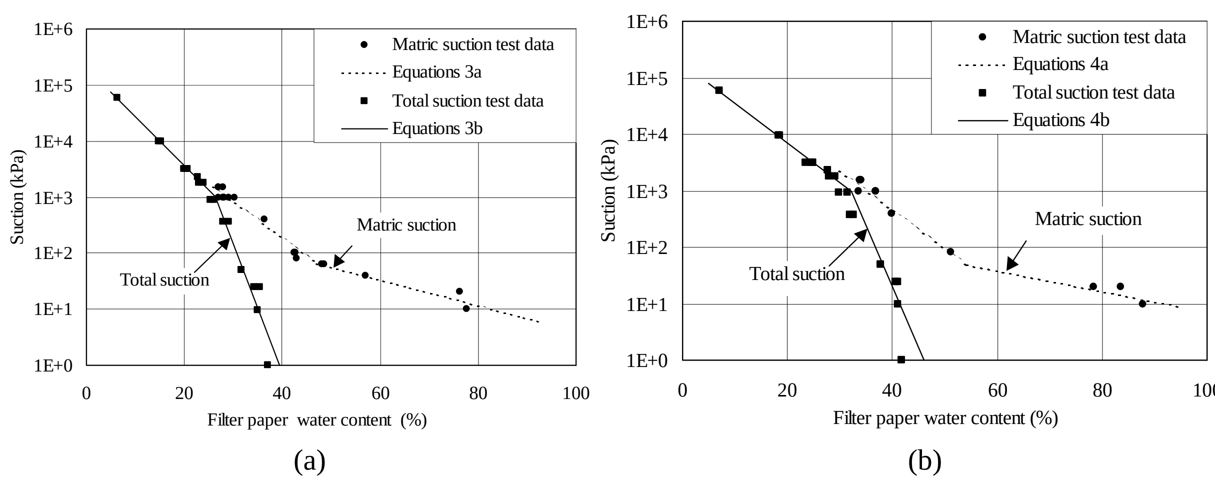


Figure 6. Total and matric suction calibration curves of (a) Whatman No. 42 and (b) Schleicher & Schuell No. 589-WH filter papers (from Leong et al., 2002).

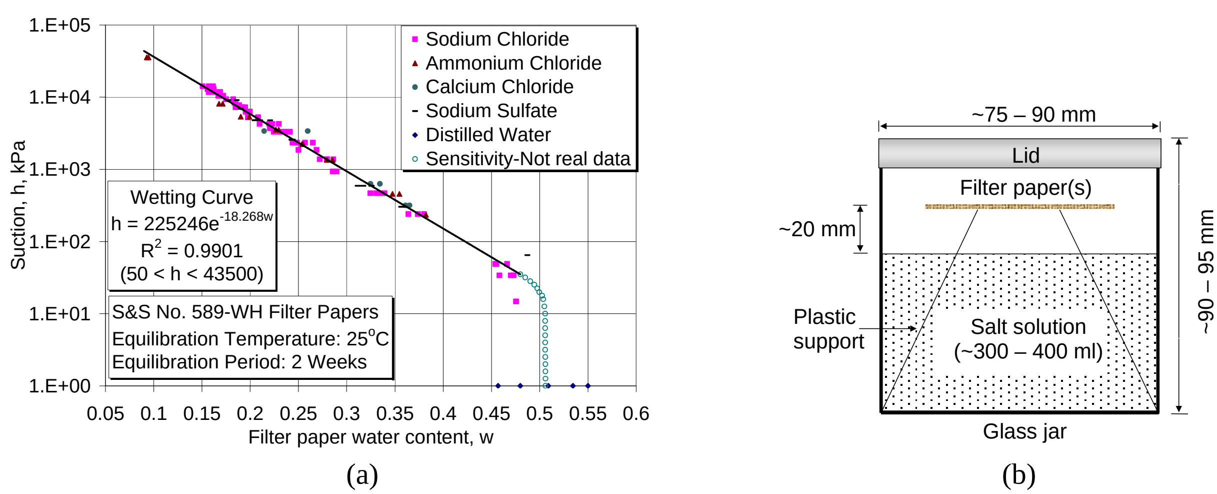


Figure 7. (a) Calibration curve of Schleicher & Schuell No. 589-WH filter papers and (b) schematic drawing of the calibration test configuration (from Bulut and Wray, 2005).

4. Tertiary Methods

The disadvantage of the secondary methods of indirect suction measurement is the need to determine the moisture content of the porous medium in equilibrium with the soil. Tertiary methods of indirect suction measurement overcome this problem by measuring properties of the porous medium that indicates its moisture content. Examples of devices that employ the tertiary method of indirect suction measurements are thermal conductivity sensor and electrical conductivity sensor. Both the thermal conductivity sensor and electrical conductivity sensor has a porous block within which the electrical circuitry is embedded. A common limitation of these sensors is the strength and durability of the porous block during installation and service life of the sensor in the field.

4.1. THERMAL CONDUCTIVITY SENSOR

A thermal conductivity sensor employs a porous block typically ceramic as a medium to measure matric suction indirectly. If a matric suction gradient exists between the soil and block, water flux will occur until their suctions are equal. The thermal conductivity of the block consists of the thermal conductivity of the solid and the fluid (air or/and water) that fills the void of the porous block. Thermal conductivities of air and water at 20^{o}C are 0.026 W/mK and 0.60 W/mK, respectively (van Wijik, 1963). The thermal conductivity of water is about 25 times that of air. Therefore, as the moisture content of the porous block increases, the thermal conductivity of the block increases. The moisture content of the block is measured by heating the porous block with a heater embedded in the centre of the porous block and measuring the temperature rise during heating. The temperature rise is related to the thermal conductivity of the porous medium and the moisture content. The temperature rise can then be used as an index of matric suction in the soil. The time to equilibrate depends on the temperature gradient and the hydraulic conductivity of the porous medium and the surrounding soil. Differences between types of thermal conductivity sensors are mainly due to the temperature-sensing element. Soil salinity has insignificant effect on the thermal conductivity sensor readings. The basic design of thermal conductivity sensor essentially follows the design of Phene et al. (1971) as shown in Figure 8. Over the years, the performance of the thermal conductivity sensor has been improved. Thermal conductivity sensors have been used in the laboratory as well as in the field (van der Raadt et al., 1987; Fredlund and Wong, 1989; Oloo and Fredlund, 1992; O'Kane et al., 1998; Marjerison et al., 2001; Zhang et al., 2001; Nichol et al., 2003). Currently, thermal conductivity sensors are available commercially (e.g. Campell Scientific, Inc. and GCTS). The Campbell Scientific thermal conductivity sensor CSI 229 has a matric suction measurement range from 10 to 1500 kPa whilst the GCTS thermal conductivity sensor FTC-100 has a matric suction measurement range from 1 to 1500 kPa. For the CSI 229, a 50 mA current is used with a 20-30s heating time. Typically the ambient temperature and the temperature after the heating period is recorded from which the matric suction is inferred from the calibration curve. For the FTC-100, a 200 mA current is used with a 60s heating period. The heating curve is recorded for three minutes during a measuring cycle. The diameter and length of the CSI 229 thermal conductivity sensor porous block are 15 mm and 25 mm, respectively, whilst those of the FTC-100 thermal conductivity sensor are 28 mm and 38 mm, respectively. The CSI 229 is more sensitive at matric suctions less than 300 kPa (He, 1999). The resolutions of the FTC-100 suction measurements in the ranges of 1-10, 10-100, 100-1000 kPa are 0.1, 1, and 5-10 kPa, respectively (UST, 2004).

The main problem with the thermal conductivity sensor is the variable uniformity of the porous block from sensor to sensor. This has entailed the need of a separate calibration curve for each thermal conductivity sensor. In addition, the thermal conductivity sensor shows hysteretic behavior on drying and wetting. Reece (1996) suggested that the thermal conductivity of the CSI 299 be normalized with the thermal conductivity measured after oven drying the sensor. With the normalization, Reece (1996) obtained a linear calibration curve between the inverse of the normalized thermal conductivity and the natural logarithm of matric suction up to 1200 kPa. Above a matric suction of 1200 kPa, a non-linear calibration curve is obtained. The hysteretic effect was not considered in the interpretation of matric suction measurement. Zhang et al. (2001) evaluated 30 CSI 229 sensors and showed that the effect of hysteresis in the CSI 229 thermal conductivity sensor should be taken into consideration when measuring matric suction. The hysteresis of a typical CSI 229 thermal conductivity sensor is shown in Figure 9(a). Different calibration curves are obtained for the drying and wetting of the CSI 229 as shown in Figure 9(b). Similar hysteretic effects were also observed in the precursor of the FTC-100 sensor (Feng and Fredlund, 2003). The equilibration time of the thermal conductivity sensor is dependent on the contact condition between the central element (heater and temperature sensor) and the porous block. Even the contact condition between the sensor and the soil affects the response of the CSI 229 (Zhang et al., 2001). Zhang et al. (2001) found that equilibration time of the CSI 229 thermal conductivity sensor can vary from several hours to several tens of hours irrespective of the suction level due to contact condition between the sensor and the soil. Furthermore, the porous block of the CSI 229 thermal conductivity sensor could be easily damaged during installation. Nichol et al. (2003) installed eighteen FTC-100 type of thermal conductivity sensors in the field at depths of 0.2m and 4.5m. They found long-term drift of the thermal conductivity sensors. However O'Kane et al. (1998) and Marjerison et al. (2001) did not experience such problems in their long term monitoring of matric suction with thermal conductivity sensors.

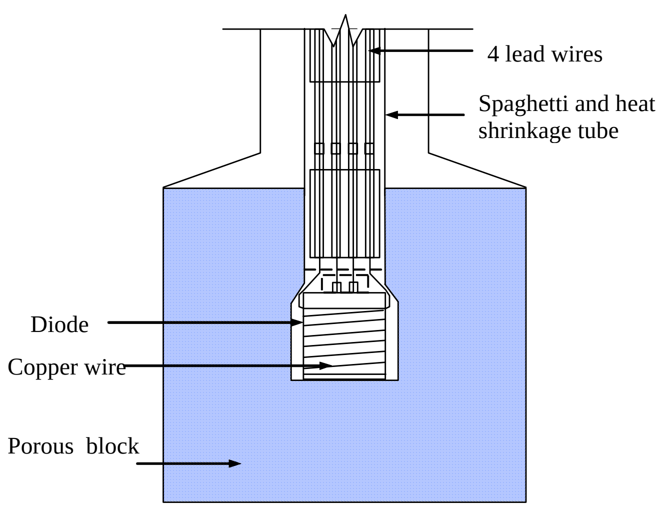


Figure 8. Cross-section of a thermal conductivity sensor (from Phene et al., 1971).

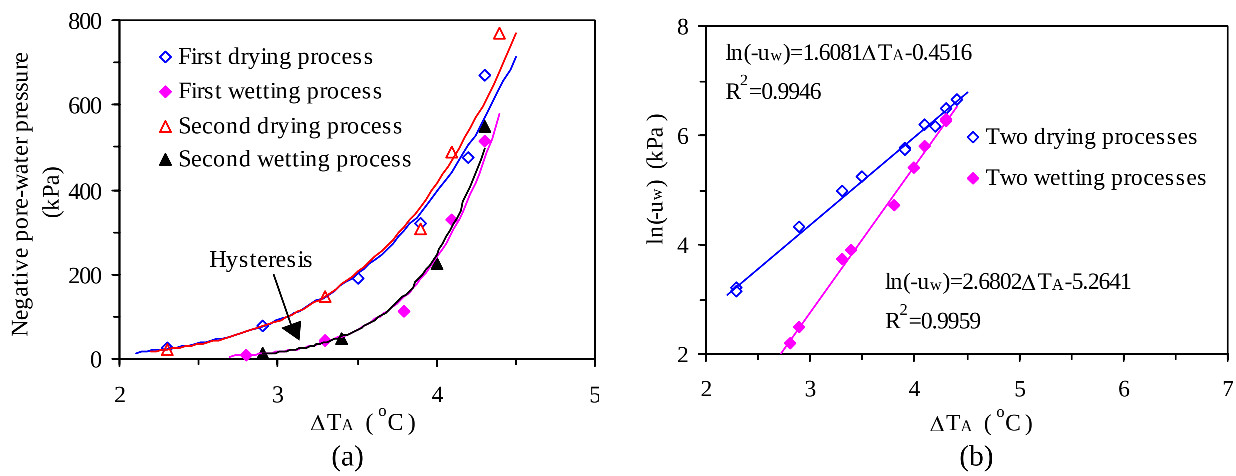


Figure 9. (a) Hysteresis effect of CSI 229 thermal conductivity sensor and (b) calibration curves of a typical CSI 229 thermal conductivity sensor for 24s heating time with a 50 mA current (from Zhang et al. 2001).

4.2. ELECTRICAL CONDUCTIVITY SENSOR

The electrical conductivity sensor has been used to measure matric suction indirectly since the 1940s (Buoyoucos and Mick, 1940), making it one of the oldest methods of soil matric suction measurement. Electrical conductivity sensors are commercially available (e.g. Soilmoisture Inc., Measurement Engineering Australia, Delmhorst Instrument Company, Irrometer Company Inc. and Environmental Sensors Inc.). The electrical conductivity sensor consists of a porous block and two concentric electrodes embedded inside the block (Figure 10(a)). The porous block serves the same purpose as the porous block in the thermal conductivity sensor. However instead of thermal conductivity, the electrical conductivity sensor measures the electrical conductivity of the porous block. As the moisture content of the porous block increases, the electrical resistance of the block decreases. The electrical resistance of the porous block can be related to the matric suction of the block. Unfortunately, the electrical resistance of the porous block is also dependent on the salt concentration of the soil solution and may not be a direct indication of the moisture content of the porous block. The electrical conductivity sensor must be excited by a small AC voltage to prevent polarization. Polarization effects will cause the results to be distorted and deterioration of the electrical conductivity sensor. The AC signal must then be converted back to DC voltage for reading. The need for conversion of AC signal to DC signal means additional hardware is needed to interpret the reading. Usually the electrical conductivity sensor reading is read manually from a meter, limiting the number of readings when used in the field (Skinner et al. 1997).

Gypsum was found to be the most suitable porous block material as gypsum took the shortest time to saturate and responded fastest in matric suction measurements (Buoyoucos and Mick, 1940). Gypsum also tends to buffer the soil salinity thereby decreasing the effect of soil salinity. This however has the unintended effect of degrading the electrical conductivity sensor as the gypsum eventually dissolves completely into the soil solution. Similar to the thermal conductivity sensor, the gysum block of the electrical conductivity sensor also suffers from hysteresis. The electrical conductivity sensor has a long equilibration time (2 to 3 weeks) for measuring matric suction in a rapidly changing moisture environment (Aitchison and Richards 1965).These shortcomings had led to a diminished use of electrical conductivity sensor for matric suction measurement even in the agricultural industry (Skinner et al. 1997). He (1999) evaluated the performance of the Soilmoisture model 5201 gypsum electrical conductivity sensors. The gypsum block has a diameter of 32 mm and a length of 35 mm. The electrical conductivity sensor is used together with a Soilmoisture model 5910-A meter. The meter provides a 60 Hz square wave, 1-Vpp excitation voltage and gives a digital readout. The equilibration times of the gypsum electrical conductivity sensors were found to vary with matric suction ranging from 6 hours for a matric suction of 50 kPa to 50 hours for a matric suction of 1500 kPa. The calibration curves of two Soilmoisture model 5201 gypsum electrical conductivity sensors (ECS1 and ECS2) are shown in Figure 10(b). The sensitivity of the electrical conductivity sensor becomes very low when the matric suction exceeds 300 kPa. Currently research on the electrical conductivity sensor is still ongoing to overcome its shortcomings.

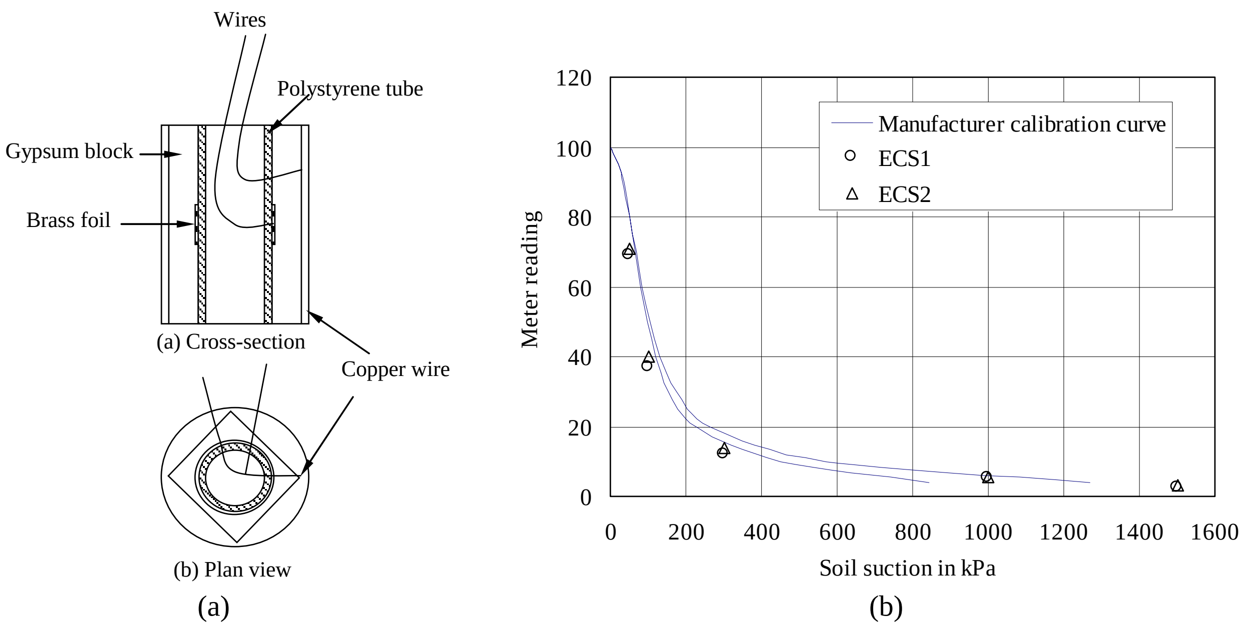


Figure 10. (a) A typical electrical conductivity sensor (from Ridley, 1993) and (b) calibration curves for the Soilmoisture model 5201 electrical conductivity sensor using Model 5910-A Soilmoisture meter (from He, 1999).

5. Summary

This paper has summarized basic working principle, calibration, measurement, and application of indirect soil suction measurement methods based on the most recent literature. The indirect suction measurement methods have been grouped into primary methods which measure the vapor pressure, secondary methods which measure the moisture content of a porous medium, and tertiary methods which measure other properties of the porous medium which indicates its moisture content. Table 3 summarizes the key characteristics of the indirect suction measurement methods. The source/manufacturer mentioned in the paper is meant for reference and does not represent product endorsement by the authors. The list is also not meant to be exhaustive.

Table 3. Summary of indirect soil suction measurement methods.

Category	Method	Suction Component	Suction Range (kPa)	Equilibration Time	Source/ Manufacturer
Primary	Thermocouple Psychrometer	Total Suction	300 – 7000	1 hour	Wescor: www.wescor.com Campbell Scientific: www.campbellsci.com
	Transistor Psychrometer	Total Suction	100 – 10000	1 hour	Soil Mechanics Instrumentation: www.smi-unsat.com
	Chilled-Mirror Psychrometer	Total Suction	500 – 30000 (or higher)	10 minutes	Decagon Devices: www.decagon.com
Secondary	The Filter Paper Method	Total/Matric Suction	50 – 30000 (or higher)	5 to 14 days	Whatman: www.whatman.com Schleicher & Schuell: www.schleicher-schuell.com
Tertiary	Thermal Conductivity Sensor	Matric Suction	1 – 1500	hours to days	Campbell Scientific: www.campbellsci.com GCTS: www.gcts.com
	Electrical Conductivity Sensor	Matric Suction	50 – 1500	6 to 50 hours	Soil Moisture Equipment: www.soilmoisture.com

6. Conclusion

Accurate total suction measurement is still difficult with current technology, especially for total suction levels below about 100 kPa. Difficulties with the primary methods measurement techniques basically arise from two main sources. The first stems from the fact that relative humidity in the soil air phase changes only a small amount within the typical range of suction interest. Most measurements of interest to studies of unsaturated soils lie in the narrow relative humidity range from about 0.94 and 1.00. The second main source of difficulty arises from the fact that minor temperature fluctuations may lead to large errors in the determination of total suction. Refinements to minimize the temperature sensitivity of the psychrometric techniques may be possible through a careful analysis of heat and vapor flow through and around the measuring sensors. More research is needed to improve primary methods of suction measurement. The secondary methods of indirect suction measurement are prone to operator's error. Unless a strictly practiced laboratory protocol is followed, the filter paper method may give questionable results. However the filter paper method is simple and is the most affordable indirect suction measurement method. The tertiary methods of indirect suction measurement (thermal conductivity sensor and electrical conductivity sensor) measure the properties of the porous medium associated with its moisture condition from which matric suction is inferred. However, the most severe limitation of the thermal conductivity sensor and the electrical conductivity sensor suffer is hysteresis. Correct matric suction is obtained only if the appropriate calibration curve is used.

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Engineering Structures in Expansive Soils

Estruturas de Engenharia em Solos Expansivos

Robert L. Lytton

ABSTRACT: The design of engineering structures on expansive soils must be based upon a rational analysis of the movements and stresses they must withstand during their expected service life. Measured suction profiles can be used to determine the depth of the moisture active zone. The lateral moisture active zone may be determined in two different ways depending upon whether the climate is semi-arid to wet or drier than semi-arid. Steady state and transient solutions for suction change and the controlling levels of the suction at the top and bottom of the moisture active zone are presented. Vertical movement and lateral pressure can be determined from these predicted changes of suction. Downhill creep can be measured with viscoelastic properties of the soil.

RESUMO: O cálculo de estruturas de engenharia em solos expansivos deve ser realizado com base em análise racional dos movimentos e tensões a que estarão sujeitas durante a vida útil. Perfis de sucção podem ser utilizados para determinar a profundidade da zona ativa de humidade dependendo se o clima é de semi-árido a húmido ou mais seco. Soluções *steady state* e transientes da mudanca de sucção e os níveis de controle da sucção no topo e fundo da zona de humidade ativa são apresentados. Movimento vertical e pressão lateral podem ser determinados a partir das mudanças em sucção previstas. *Downhill creep* pode ser medido a partir das propriedades viscoelásticas do solo.

1. INTRODUCTION

The properties of expansive soils achieve economic importance when they affect the performance of engineering structures that are founded on them. The engineering structures which are considered in this paper include the following: foundations (slabs, mat foundations, and pier and beam), pavements (highway and airport), retaining walls, pipelines, canals, slopes, moisture barriers, landfill covers and liners, rehabilitation structures (piers, root barriers, moisture barriers). Each of these have their own performance criterion which in every case should be the objective of the analysis to predict and design to accommodate.

2. PERFORMANCE CRITERIA

The performance criteria for each of the engineering structures listed above are as follows:

Engineering Structurea			Performance Criteria
Foundations -	slab	:	Differential movement: vertical and lateral and allowable stresses
	mat	:	Differential movement and allowable stresses
	pier	:	Total vertical and lateral movement; lateral pressure; allowable stresses
Pavements -	Highway	:	Roughness spectrum, International Roughness Index
	Airport	:	Roughness spectrum, Pilot and Passenger acceleration
Retaining Walls			Lateral pressure and movement, allowable stresses
Pipelines			Roughness spectrum, allowable stress, fatigue criteria, corrosion
Slopes			Downhill movement, shallow slope failure, slope stability
Canals			Combination of the performance criteria of retaining walls, pipelines, and slopes; thermal and shrinkage cracking; permeability of the cracks and joints
Moisture Barriers			Reduction of the movement of water in the soil and of total vertical movement
Land Fill Covers and Liners			Moisture and leachate transmission (including the effects of cracks)
Rehabilitation Structures -	Piers	:	same as piers (above)
	Moisture Barriers	:	same as moisture barriers (above)
	Root Barriers	:	same as moisture barriers but also to exclude roots

Design of these structures should always involve the prediction of the movement of the moisture and of the expansive soil that have a direct relation to the performance criteria. These criteria, in turn, should be met over the expected life of the structure which, in most cases, exceeds twenty years. This paper addresses the soil, climatic, and site conditions that have a major impact upon soil and moisture movement and the design-and-performance criteria. These include problem site conditions and how to recognize them; some methods of predicting the movement of expansive soils under many of these conditions for use in design; and finally some design criteria including seasonal and long-term effects of the local climate and the effects of the activities of the occupants of the engineered structures.

3. PROBLEM SITE CONDITIONS

The design of engineered structures on expansive soils is a challenge in any condition but in the absence of the problem site conditions of vegetation, drainage, and slopes, the prediction of movement and design to accommodate it seems almost simple.

3.1 Vegetation

The effect of vegetation on expansive soil movement is dictated primarily by two features of the vegetation below ground level: the depth and extent of the root zone and the cracks in the soil that are generated by the growing roots. No vegetation can survive beyond the wilting point and so one should not expect to see the cracks in the soil that are generated by roots to penetrate into soil with total suction levels <u>above</u> the wilting point. The roots will break the soil up into small blocks (clods or peds) and water travels much more easily in the cracks between these small blocks. In fact, the soil zone in which these small blocks of soil are found and in which water travels relatively more easily, both in liquid and vapor form, is the <u>moisture active zone</u>. The soil zone in which movement occurs is always more shallow than this and is called the <u>movement active zone</u>. The depth of the moisture active zone is dictated principally by the presence of soil broken into clods and peds, which in turn is principally done by vegetation. In the upper 0.6 - 1.0 m, this disintegration of the soil into small blocks is assisted by evaporation and shrinkage and burrowing animals. The wilting point of most plants is around 3100 kPa or in terms of the Gibbs Free energy of the soil moisture it is 3.16×10^6 mm (5.5 on a log scale to the base 10). One should not expect, and normally does not see in the field, a moisture active zone that extends into soil with a suction level higher than those noted above.

The roots of a tree within the moisture active zone can subject the soil to extreme variations of suction ranging from very wet (31 KPa or 3.5 on the mm - log scale) to the wilting point (3100 kPa or 5.5 on the mm - log scale). This, together with the crack fabric in the soil, which provides lessened lateral restraint, allows the soil to expand and contract large amounts both vertically and horizontally. Nearby engineering structures, or those beneath which roots intrude, will be affected by this movement. The movement of the soil for a distance of 0.3 m to 3.0 m from the root zone will be affected by the seasonal fluctuation of suction in the root zone. When a tree is pulled out of the ground or cut down to make way for new construction, it is usually done in the warm and dry construction season when the tree has increased the suction in its root zone to a level near the wilting point. When a structure is placed over the location where the tree was, and the suction in the root zone returns closer to its equilibrium value, the soil in the root zone heaves, causing large differential movements in the overlying structure.

Effective countermeasures to this include injecting water into the root zone to lower its suction level, and <u>monitoring</u> the suction level achieved to assure that the expected heave has been neutralized.

3.2 Drainage

The drainage around any engineering structure should always be "positive," that is, all water falling near the structure should drain, or be channeled away from it. If it is allowed to stand, the water will percolate into the system of cracks in the moisture active zone. The suction will decrease as the water percolates downward in accordance with diffusion laws, and will be limited by the boundary suction at the surface. The wettest this suction has been found in the field is around 31 KPa (3160 mm or 3.5 on the log mm scale). It takes consistent ponding of water for a period of several months to permit the suction to change to this lower level down to a depth of 2.5 m. Poor drainage that ponds water for no more than a day after a rainfall and then evaporates, or lawn watering, which has a similar effect, will induce an oscillatory pattern of suction with depth, typically centered around the long-term equilibrium suction level for that site. Lawn watering does not and cannot cause a shift in this long-term equilibrium suction. If there is a shallow moisture active zone, below which there is a layer of intact soil with a suction level at or above the wilting point, water will accumulate on top of that intact soil layer (called a "clay pan") and lower the suction in the soil above the intact layer. A "shallow" moisture active zone is one in which an annual change of suction greater than 0.2 on the log - mm suction scale occurs above the top of the intact, high suction soil layer. Such shallow zones are up to 6 m thick, but are more frequently less than 3 m thick. The water that accumulates on top of the intact layer will penetrate that layer only very slowly, in accordance with Gardner's law of hydraulic conductivity (see Lytton, 1994). Water accumulating above this intact, high suction layer will form an intermittent perched water table with a total suction level around 31 kPa (3.5 on the log - mm scale). This shallow moisture active zone with an intermittent perched water table should not be confused with the case of a deep permanent water table in residual soils such as are found in South Africa. Such a water table will form an equilibrium suction profile with the long-term climate in its location, centered upon a steady-state efflux of moisture. If a building or other extensive ground cover is placed on such a site, the long-term efflux is interrupted and water begins to accumulate above the permanent water table, mounding up beneath the center of the covered area. This lowers the suction in the entire soil column above the water table and can result in an extensive heave pattern. The depth to which the upward movement occurs is governed principally by the amount of suction change that has occurred. Except in the capillary fringe immediately above such a permanent water table, the suction will never drop below 31 kPa (3.5 on the log - mm scale).

3.3 Slopes

Slopes can be either natural or compacted fill, the latter being from less than 1 m to well over 30 m deep. The soils in such slopes obey the same laws that govern the fluctuation of suction in soils on flat sites. The only difference in movement that occurs in slopes is that the normal heaving and shrinkage, both vertical and lateral, is superimposed upon a downhill creep due to gravity. If the fill is poorly compacted, there will be an additional compression of the fill as the soil adjusts and densifies.

Vegetation on the slopes will open cracks during dry weather that fill with water when rain or irrigation watering flows down the slope. The water runs into the cracks, soaking into the sides of the cracks, especially at or near the bottom of the cracks, lowering the suction and strength of the Soil. The wetter and weaker zones are shallow, less than 2 m generally, and can result in shallow slope failures if the suction drops low enough in the intact soil along the bottom of the zone and water fills the cracks to a height above a point of incipient failure sufficient to cause the effective stress to reach zero. The pattern of cracks is principally orthogonal, one set parallel with the strike of the slope and the other set pointing downhill in the direction of the dip.

Water ponding at the top of the slope can feed water into the gallery of cracks in the slope and cause these shallow slope failures. Intercepting this water and draining away from the slope is usually a simple matter that can reduce or eliminate the occurrence of shallow slope failure.

Regardless of whether there is a danger of this shallow slope failure, compression of poorly compacted fill and downhill creep will certainly occur. The rate of creep is increased with larger slope angles and less stiffness of the soil. The latter is governed largely by its suction level, and its water content at that suction level. The higher the water content, the faster will be the rate of creep. Thus, the finer-grained soils will be particularly vulnerable.

This discussion of problem site conditions has been narrative. In the next section of this paper, some of the physical principals and equations that can be used to predict these movements of soil and moisture will be presented.

4. PREDICTION OF MOVEMENT IN EXPANSIVE CLAY

In this section of the paper, the following will be presented and discussed:

1. The relation between total stress and moisture stress
2. A constitutive equation for volume change
3. The relation between the edge moisture variation distance and the Thornthwalte moisture index
4. A catalog of active suction profiles from a wide variety of sites
5. Transient suction changes due both to cyclic and steady suction at the boundary
6. Trees
7. Drainage
8. Slopes

The presentation cannot be exhaustive because of the broad scope of these subjects, but several of the more useful concepts will be discussed.

4.2 Relation Between Total Stress and Moisture Stress

Two spheres in contact held together by films of water which wet both of the spheres has formed the basis for a relation between total stress, σ, and moisture stress, u_w, in the presence of an air pressure, u_a. Figure 1 illustrates a free body diagram of these stresses acting upon a sphere, of radius r, with air pressure acting all around it. This is characteristic of moist to dry soils, but not very wet soils. The moisture stresses are characterized by a surface tension, T, the water

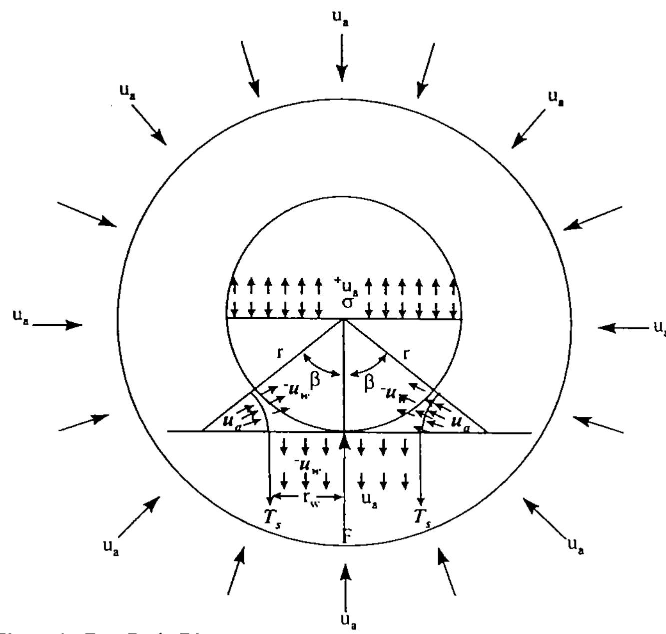


Figure 1. Free Body Diagram

being in tension with a stress of u_a-u_w, a contact force between spheres of N, and a total stress of σ acting at the midplane of the sphere. The surface tension force, T, has a wetting angle, α, which can be, but is typically not zero. The point of contact between the surface tension force and the surface of the sphere is at an angle, β. The equation of vertical equilibrium of the sphere is:

$$\frac{N}{4r^2} = \sigma' = (\sigma - u_a) + \frac{\pi}{4}\left(\frac{rw}{r}\right)^2 (u_a - u_w) + \frac{\pi T_s}{2r}\left(\frac{rw}{r}\right) \tag{1}$$

where σ' = the "effective" stress
σ, u_a, u_w = total stress, air pressure and stress in the water
T_s = surface tension
r, r_w = radii of the sphere and of the water film

and the equation of vertical equilibrium of the water film is:

$$T_s(1 - \sin\beta) = r\,\frac{(1-\cos\beta)(1-\sin\beta)}{\cos\beta}\,(u_a - u_w) \tag{2}$$

with a wetting angle, α, of zero degrees the equation relating the effective stress, σ'

(=$N/4r^2$), to the total stress (σ-u_a) and the moisture stress (u_a-u_w) uses a collection of terms known historically as the χ-factor

$$\chi = \frac{\pi}{2}\left(\frac{1-\cos\beta}{\cos\beta}\right) \tag{3}$$

with a non-zero wetting angle, the equation for the χ-factor is:

$$\chi = \frac{\pi}{4}\left(\frac{\sin^2(\alpha+\beta)+2\cos\beta-2\sin(\alpha+\beta)\sin\alpha-\cos^2\beta-\cos^2\alpha}{\cos^2(\alpha+\beta)}\right) \tag{4}$$

The following table shows the relation between the central angle, β and the χ-factor for wetting angles of zero degrees and 20 degrees.

Central Angle, β	χ-factor α=0°	χ-factor α=20°
0	0	0
30	0.244	0.234
45	0.650	0.671
52.34	1.000	1.127
60	1.571	2.385

The χ-factor does not reach 1.0 until a central angle of 52.34°. Beyond 45°, all of the sphere's surfaces are covered with water films and the free-body conditions illustrated in Figure 1 are no longer valid. These χ-factor results for soils with non-spherical particles obviously must be modified. However, these results closely parallel the use of the volumetric water content instead of the χ-factor for moist soils by Lamborn (1986), who uses the principals of reversible thermodynamics to arrive at that result. As the soils become wetter, there is a transition zone from a value nearly equal to the volumetric water content, θ, to a value of 1.0. The transition occurs between the suction values of +310kPa (4.5 on the log - mm scale) and +10 kPa (3.0 on the log - mm scale). This is discussed in more detail in Lytton (1995).

Thus it is appropriate to state that a change of suction, h, has the same effect upon volume change and shear strength as an equivalent change of mean principal stress, σ, in accordance with the relation:

$$\Delta\sigma = \theta f\,|\Delta h| \tag{5}$$

where θ = the volumetric water content

f = a function of volumetric water content which varies from 1.0 at a suction level of -310 kPa to a value of $1/\theta$ at a suction level of 10 kPa

$\Delta\sigma, |\Delta h|$ = corresponding changes in mean principal stress and suction

The effect of osmotic suction components in the water will alter the surface tension and the wetting angle and thus, necessarily, will alter the relation between total stress and moisture stress. Now that surface chemistry methods are able to measure surface energies and wetting and dewetting angles, (Good and Van Oss, 1992) it is possible to explore the relation between total stress, matric suction, and osmotic suction. Such an exploration will provide interesting and useful results. It will show the separate effect of matric and osmotic suction, wetting and dewetting, on shear strength and volume change characteristics of an expansive soil. A study using the free body diagram of a sphere acted upon by water films will give valuable qualitative insight into these relations.

4.2 Constitutive Equation of Volume Change

The heave and shrinkage of expansive soil in a profile follows a large strain volume change function which has limits, as explained in Lytton (1995). Subsequent correspondence with Juarez-Badillo, whose work was referred to in that paper suggested some revisions to the model proposed, as illustrated below in Figure 2.

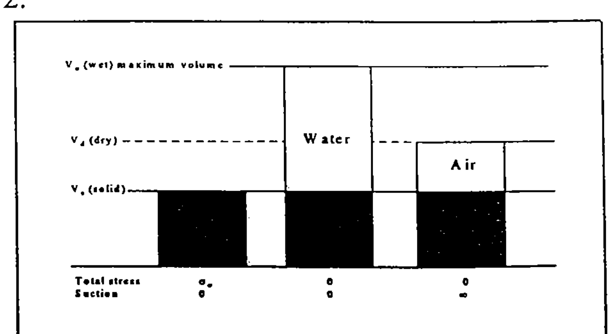


Figure 2. Natural Limiting Volumes in Unsaturated Soils and Corresponding Stress States.

The suggestion was that some mechanical stress, σ_s, is required to reduce the volume of the soil to the volume of the solids. In the previous paper (Lytton, 1995) it was assumed that an infinite stress was required. Using Juarez-Badillo's approach to determining a constitutive volume-total stress-suction surface produces the following relation at a small total stress of σ_I as suction changes

$$V_h = \frac{V_\sigma + aV_a \mid h \mid^{-\gamma_h}}{1 + a \mid h \mid^{-\gamma_h}} \tag{6}$$

The volume change between V_h at a stress level of σ_I and V_s at a stress level of σ_s is

$$V = \frac{V_h + bV_s \left(\frac{\sigma_s - \sigma}{\sigma - \sigma_i} \right)^{\gamma_\sigma}}{1 + b \left(\frac{\sigma_s - \sigma}{\sigma - \sigma_i} \right)^{\gamma_\sigma}} \tag{7}$$

where σ = the level of mean principal stress corresponding to the volume, V.

σ_s = the level of mean principal stress required to compress the soil to a volume equal to the volume of solids, V_s

σ_i = the level of mean principal stress above which the soil volume begins to decrease; measured values of σ_i are around 7 - 10 kPa

V_σ = the column of a soil at zero suction and under a confining mean principal stress of σ, which is greater than σ_i.

$|h|$ = the positive value of suction

a, b = coefficients to be determined from the measured volume—suction –mean principal stress surface

γ_h,γ_σ = coefficient for the volume change due to a change of mean principal stress, respectively.

There is an interaction between the suction and the mean principal stress at a point in a soil mass below the surface. As suction level decreases, the Helmholtz free energy stored in the water is released and is able to do work. The work that it does is to increase the potential energy stored in the surrounding soil, and correspondingly to increase the volume and the confining pressure. When the suction increases, the surrounding confining pressure decreases, releasing the potential energy stored in the soil and transferring it to the water. The work the water does is to decrease the volume of the soil. This exchange of potential energy between the soil volume and stress state and the water volume and the suction state is an energy balance which explains the relations between heave and shrinkage, lateral confining pressure increases and decreases and the corresponding decreases and increases in suction. In swelling, as in shrinking, the net change of energy is zero as summarized in the following relation:

$$\int_h V_h \theta f \, d|h| - \int_\sigma V \, d\sigma = 0 \tag{8}$$

If the level of mean principal stress is high enough, no volume change takes place. Instead, a decrease of suction results in an increase of lateral confining pressure and of the mean principal stress, σ. It is for this reason that the depth of <u>moisture</u> active zone is always deeper than the depth of the <u>movement</u> active zone. The depth at which volume change becomes possible depends mainly upon how much suction changes, the magnitude of the volumetric water content, θ, and the function, f, and the relative sizes of the coefficients γ_h and γ_σ.

Methods of measuring or estimating the coefficients γ_h and γ_σ are given by McKeen (1981) and Lytton (1994), among others

The mean principal stress, σ, increases as the suction decreases and the soil attempts to swell against its confining pressure. The mean principal stress is given by

$$\sigma(z) = \left(\frac{1 + 2K_o}{3} \right) \left(\gamma_t z + surcharge\,pressure \right) \tag{9}$$

where γ_t = the total unit weight of the soil

z = the depth below the surface

K_o = the "at rest" lateral earth pressure coefficient. It is "at rest" according to common usage as long as the total stress is not changing.

In an expansive soil, the value of K_o is a nearly static value only when the soil is in a steady-state suction condition and neither swelling nor shrinking is taking place. In all other conditions, the value of K_o changes and depends upon whether there are cracks in the soil, and if they are opened or closed, and if the soil is shrinking or swelling. Using small strain theory, the following expressing can approximate the current value of K_o.

$$K_o = \frac{3}{2}\left(\frac{\sigma_i}{\sigma_v}\right)\left(\frac{h_i}{h}\right)^r\left(\frac{h_e}{h_d}\right)^{\frac{2r}{3(1-f)}} - \frac{1}{2} \qquad (10)$$

where r = the ratio of (γ_h/γ_σ), the volume change coefficients for suction and mean principal stress, respectively

f = the fraction of the total volume change, $\Delta v/v$, that is directed vertically

h_i, h = the initial and current levels of total suction (mm)

h_e, h_d = the equilibrium and most recent dry suction. (This term estimates the shrinkage cracking that must be closed when the soil is wetting. The term involving h_e and h_d should not be used if the soil is drying. (Measured in mm)

σ_v = the vertical total stress including overburden and surcharge

σ_i = as noted before, the mean principal stress level above which volume change takes place.

Thus, the values of K_o and f are not independent of one another. Common values of f and K_o that are used in practice and the conditions to which they apply are as follows

f = 0.5 soil is drying

f = 0.8 soil is wetting

These values have been back-calculated from field observations by McKeen (1981).

$K_o = 0$ 0 soil is dry and cracked

$K_o = 1/3$ soil is dry and cracks are opening

$K_o = 1/2$ cracks are closed and suction is in a steady state condition

$K_o = 2/3$ cracks are closed and soil is wetting

$K_o = 1$ soil suction is at or below its climatic equilibrium value and the soil is wetting. Soil is in a hydrostatic stress condition

$K_o = 2\text{-}3$ Passive earth pressure, or maximum lateral pressure

Thus, the exchange of potential energy between the water phase and the soil mass is one that involves an interaction between the two, whether the soil mass is expanding or contracting. The K_o-value should not be regarded as a constant even under steady state moisture and stress conditions because of the ability of these soils, which are highly viscoelastic, to relax under constant stress conditions, A more extended discussion of the lateral earth pressure coefficient is found in Lytton (1995).

4.3 Relation Between Edge Moisture Variation Distance and Thornthwaite Moisture Index

In 1994, a series of graphs of edge moisture variation distance plotted versus the Thornthwaite moisture Index was presented for both the edge drying and edge wetting conditions (Lytton 1994). These graphs were intended to be used in the design of pavements and foundations on expansive soils. There were seven curves shown on each graph, one for each of several different soil types, differentiated by their unsaturated diffusivity ranging between 3.9 x 10^{-2} mm^2/sec and 7.8 x 10^{-1} mm^2/sec. The points on the curves were computed using a pair of finite element programs coupled to compute transient suction change and non-linear elastic response (Gay. 1993). Severe climatic boundary conditions were imposed. For edge drying, the soil profile was initially very wet for the climate and severe drying condition was imposed. For edge wetting, the soil profile was initially very dry for the climate and a severe wetting condition was imposed on the soil beside the covered area. Nine different climatic zones ranging from a Thornthwaite Moisture Index (TMI) of -46.5 to +26.8 were used . Weather data used to calculate the TMI in each location spanned 50 years. The two graphs are repeated below.

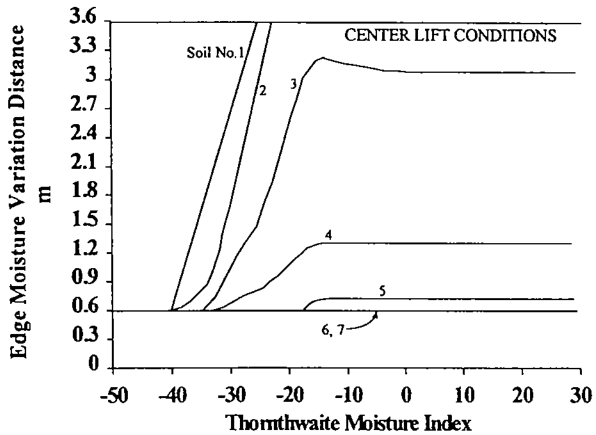


Figure 3. Edge Drying

Soil No.	Diffusion Coefficient mm^2/sec
1	7.8×10^{-1}
2	5.8×10^{-1}
3	3.9×10^{-1}
4	1.9×10^{-1}
5	8.0×10^{-2}
6	5.8×10^{-2}
7	3.9×10^{-2}

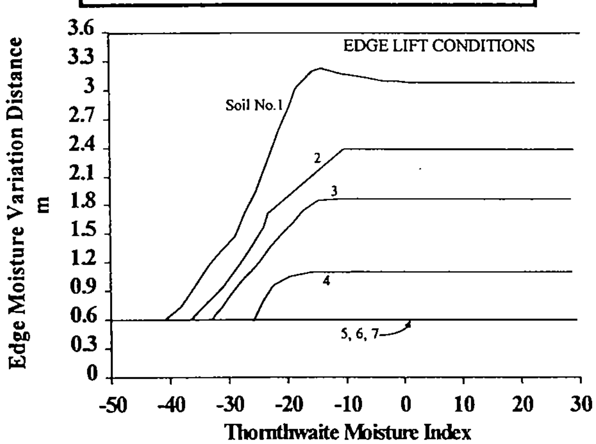


Figure 4. Edge Wetting.

The unsaturated diffusivity coefficients make use of the approach adopted by P. W. Mitchell (1980) to describe unsaturated flow of water in the cracked, moisture active zone. When the soil is at or near its equilibrium suction value, the Mitchell hydraulic conductivity is the same as that predicted by the Gardner relation (Gardner, 1958) for intact, uncracked soils. However as the soil dries to suction levels that are over a decade from equilibrium, Mitchell's relation shows a higher hydraulic conductivity than does the Gardner relation, thus in some measure accounting for the higher conductivity of the cracked soil.

4.3.1 Edge Moisture Variation Distance in Drier Climates

It may seem puzzling at first why the edge moisture variation distances begin to drop downward at Thornthwaite Moisture Index values more negative than -10. This is explained by the lower hydraulic conductivities in soils in the drier climates. However, it is known that pavements and foundations experience severe distress due to expansive clay subgrade movements in arid and semi-arid areas characterized by Thornthwaite Moisture Indexes more negative than -10. Aside from the obvious conditions in which poor drainage forms continuous ponds and high water tables (shallower than 10 m), there is damage of a cumulative nature done by the wet-and-dry cycling that occurs in these climates. The suction amplitudes are recorded in the $\log_{10}$|mm| scale. This means that the amplitude is half of the difference between the maximum and minimum total suctions <u>on the log mm scale.</u>

Thornthwaite Moisture Index	Soil No. 1	Soil No. 2	Soil No. 3	Soil No. 4	Soil No. 5	Soil No. 6	Soil No. 7
-46.5	0.25	0.22	0.19	0.14	0.10	0.09	0.09
-40.0	0.36	0.32	0.27	0.20	0.15	0.14	0.12
-35.0	0.52	0.45	0.38	0.28	0.21	0.19	0.18
-30.0	0.74	0.64	0.54	0.40	0.30	0.28	0.25
-25.00	1.13	1.00	0.85	0.67	0.54	0.51	0.49
-21.3	1.40	1.24	1.06	0.84	0.68	0.65	0.62
-11.3	1.84	1.63	1.40	1.10	0.91	0.87	0.83
14.8	1.62	1.56	1.33	1.05	0.86	0.82	0.79
26.8	1.62	1.56	1.33	1.05	0.86	0.82	0.79

Figure . Suction Amplitude [log |mm| Total Suction]

The edge moisture variation distance used in the design of foundations and pavements in the climatic zones more negative than -10 is computed with the oscillating suction transient equation proposed by Mitchell (1980).

$$u\,(x,t) = u_e + u_o \exp\left[-x\sqrt{\frac{n\pi}{\alpha}}\right] \cos\left[2\pi n t - x\sqrt{\frac{n\pi}{\alpha}}\right] \tag{11}$$

where u_e = the equilibrium value of suction expressed on the log mm suction scale

u_o = the $\log_{10}|mm|$ suction amplitude
x = the horizontal distance from the edge of the covered area
n = the number of suction cycles per second (1 year = 31.5 x 10^6 seconds)
t = time in seconds
α = the unsaturated soil diffusion coefficient (ranges between 10^{-3} and 10^{-1} mm^2/sec)

The edge moisture variation distance within which the total cyclic change of $\log_{10}|mm|$ total suction is no more than 0.2. The equation is given above can be used to solve for the edge moisture variation distance, e_m, and the result is as follows

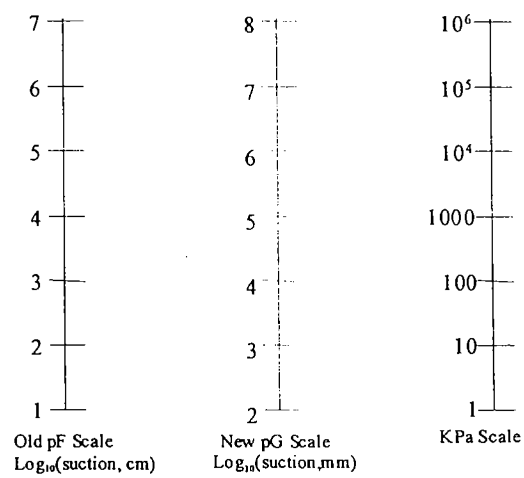
$$e_m = 10^{-3}\left(\sqrt{\frac{\alpha}{n\pi}}\right)\ln\left(\frac{2u_o}{0.2}\right) \qquad (12)$$

where e_m= the edge moisture variation distance in m.

Methods of estimating the diffusion coefficient from the Atterberg limits, and percent of the soil passing the 64 µm and 2 µm sizes are found in Lytton (1994).

4.4 Active Soil Profiles

It is beginning to become apparent that design practice can be made, if not simpler, then more rational and reliable by classifying profiles and suction patterns. Water flow in the field occurs in the cracks in the soil and in the intact clods and peds between the cracks. It occurs in liquid and vapor form as well.. In all such conditions, moisture will and must always move along a negative energy gradient and thus the energy expression of suction is most useful in both classifying profiles and predicting water movement. The symbols h, h_m, and h_s for the energy expression of total, matric, and osmotic suction in the g -mm/g form has the mnemonic value of standing for "head" or "energy head." This energy potential, a Gibbs free energy, is inherently negative. Using suction expressed as a negative head, the usual flow equations do not need to be rewritten since flow will always occur from a less negative to a more negative head. If suction were expressed as a positive stress as is convenient when dealing with shear strength and volume change, flow would occur from a lower to a higher suction. It is for this reason that the energy expression of suction, which is inherently a negative number, the mnemonic symbols h, h_m, and h_s are preferred in dealing with moisture flow, and measured in mm. The non-SI pF-scale was very useful, as well, in keeping the numerical values of the suction within a range that can be grasped readily. Thus, it is proposed that this very useful log scale of suction be transferred into the SI-units as the $\log_{10}$ mm scale with the symbol pG, with the p standing for the logarithm and G standing for the Gibbs free energy. The corresponding scales will be as follows



Along the $\log_{10}|mm|$ (pG) scale these are several important marks for classifying soil profiles. They are as follows

Moisture Condition	$\log_{10}|mm|$ (pG)
Field Capacity	3.0
Clay Wet Limit	3.5
Wilting Point	5.5
Air Dry	7.0

The suction measured in the field will never be found outside the range. Several examples of these will be used as illustrations. Some general principles must be noted first.

1. It is total suction, h, that governs the flow of water in the soil.
2. No clay soil will be found in the field wetter than pG 3.5.

3. No soil in the field will be drier than pG 5.5 if the suction is controlled by vegetation.
4. No soil in the field will be drier than pG 7.0 if the suction is controlled by surface evaporation.
5. Any soil in the field with a suction level above the wilting point (pG = 5.5) cannot be penetrated by the roots of vegetation and must be presumed to be intact, that is, not broken into small blocks, clods, or peds as is done by roots. Soils at such levels of suction may have high osmotic suctions or have been cemented by diagenetic bonding.
6. Soils at or near the surface within suction ranges of pG 3.5 to 5.5 (or 7.0 in the upper 1.0m)form the <u>moisture active zone</u>. In this zone, most of the moisture moves in the cracks in the soil and use of the Mitchell form of hydraulic conductivity is appropriate.
7. Soils deeper than 1.0 m with suction levels greater than pG 5.5 are in a <u>moisture inactive zone</u>. The soil may be presumed to be intact and that water flows through the intact soil governed by the Gardner form of hydraulic conductivity. Occasionally in such soils, fissures, or seams will be found that carry moisture. These features transmit water very slowly and can be identified in a suction profile by a horizontal v-shape, the suction increasing away from the seam, both above and below it. Contraction and expansion of the soil in such a zone can occur but only if large enough suction changes occur to overcome the confining pressures. Suction changes occur so slowly in these soils that expansion in such high suction soil will affect the performance of an engineering structure built upon it only very slowly
8. Corresponding graphs of total and osmotic suction (the latter determined by the difference between total and matric suction) will help to confirm the identification of a moisture inactive zone due to high osmotic suctions. Cementation may permit large values of matric suction at or above the wilting point. It is a good idea to confirm the existence of such an inactive zone by computing the hydraulic conductivity using Gardner's relation.

In classifying soil profiles using measured suction values, the objective is to identify

1. The depth of the moisture active zone and the beginning of the moisture inactive zone. The Mitchell hydraulic conductivity relation may be used in the moisture <u>active</u> zone whereas the Gardner relation must be used in the moisture <u>inactive</u> zone.
2. The governing suction levels in the soil profile: at the bottom of the moisture active zone, and at the surface, the maximum and minimum values

Having determined these two, it is then possible to predict the changes of suction that will occur in the future to control the vertical and horizontal movements and pressures in the soil profile. In order to demonstrate the principles of suction profile classification, several suction profiles measured in various locations in Texas and Louisiana will be used as illustrations.

4.4.1 Depth of the Moisture Active Zone

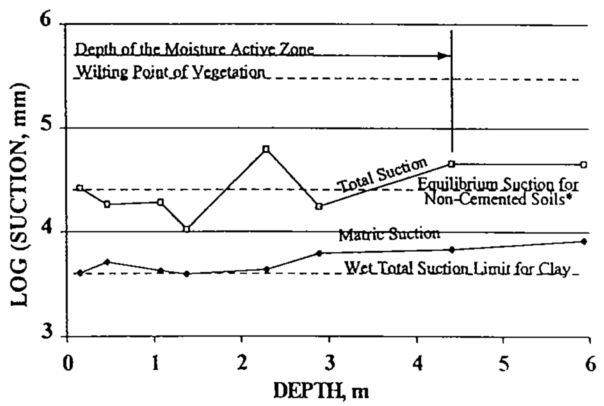
Several clues are available in the suction profile to indicate the depth of the moisture active zone, as follows:

1. The first point at which the total suction does not vary more than 0.08 $\log_{10}|mm|$ suction units per meter with depth. The suction level at which this occurs is the equilibrium suction level.
2. a permanent water table or one that is changing its elevation steadily over a multiple-year period
3. a distance 0.6m below the deepest recorded root fiber
4. The first point at which the $\log_{10}|mm|$ suction begins to be consistently at or above the wilting point of vegetation. This point occurs where the $\log_{10}|mm|$ suction level is 5.5. This indicates the presence of cemented, intact soils or

soils with high osmotic suction which would discourage penetration by roots. Cemented soils may have high matric suction values while an high osmotic suctions will have the osmotic suction nearly as large as the total suction.

5. The point where the matric suction is the same as or within 0.1 $\log_{10}$ |mm| suction units of the total suction an the total suction has become nearly constant with depth, changing no more than 0.08 $\log_{10}$ |mm| suction units with depth.

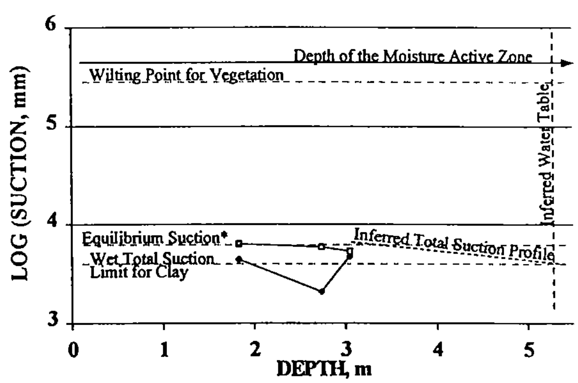
The <u>first</u> criterion is illustrated in Figure 4



• From Epirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)

Figure 4. Suction Profile with Depth Illustrating the Point where Suction Becomes Constant with Depth.

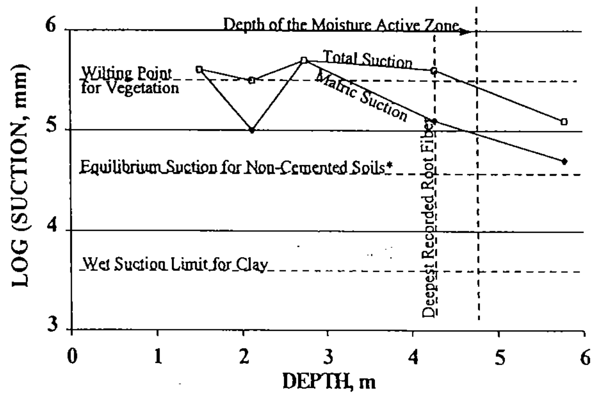
The <u>second</u> criterion is illustrated in Figure 5, which is a set of suction measurements made in and around a swamp in Louisiana. The location of the water table was inferred by projecting the total suction in mm downward on a 1:1 slope until it reached the wet limit of suction in clay of 3162 mm (pG = 3.5 or -31kPa).



• From Empirical Thornthwaite Moisture Index Relation with equilibrium suction (Russam and Coleman, 1961)

Figure 5. Suction Profile with Depth Illustrating the Inferred Presence of a Water Table.

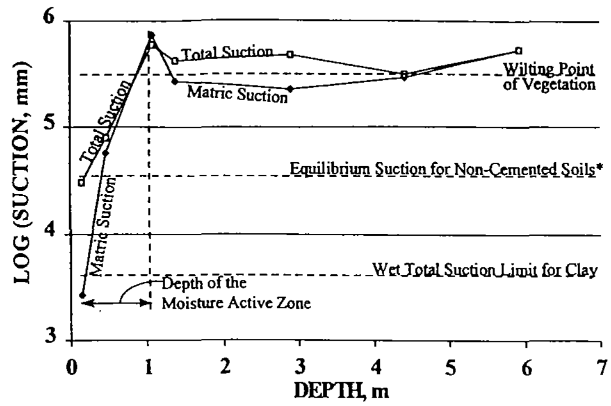
The <u>third</u> criterion is illustrated in Figure 6 which was measured in the root zone of a large oak tree in Texas during a hot, dry summer. The deepest recorded root fiber was at 4.3 m. The total suction, which had been at or slightly above the wilting point down to that point, began below that point to reduce dramatically. The moisture active zone is where moisture can move quickly in and out of the soil in the cracks formed principally by vegetation. Roots can fracture the soil approximately 0.6 m beyond or deeper than the location of the root fiber. The soil moisture beyond that point is influenced by changes of suction in the root zone but at the slower rate for intact soil governed by Gardner's relation.



• From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)

Figure 6. Suction Profile in a Tree Root Zone in Summer

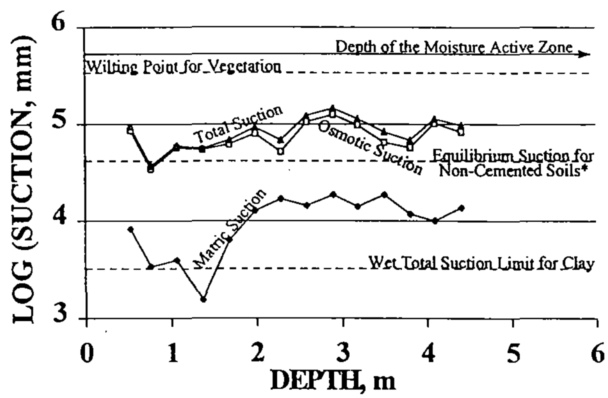
The <u>fourth</u> criterion is illustrated in Figure 7, a suction profile showing a cemented soil which roots cannot penetrate below a depth of 0.8 m. The inference that it is a cemented soil comes both from the boring log comments on the soil being "very stiff" and from the high level of matric suction, nearly equaling the total suction. The soil at this level of suction and higher cannot support vegetation and will not be cracked by it. The soil is intact and marks the limit of the moisture active zone. Frequently, rainwater falling on the ground surface will percolate down to the top of the high suction layer and will accumulate there, forming an intermittent perched water table. The soil in the moisture active zone can, and usually does, undergo large changes of suction between its established wet and dry limits, and consequently large and rapid shrinking and swelling.



* From Empirical Relation of Thornthwaite Moisture Index (Russam and Coleman, 1961)

Figure 7. Suction Profile Showing a Cemented Soil Layer.

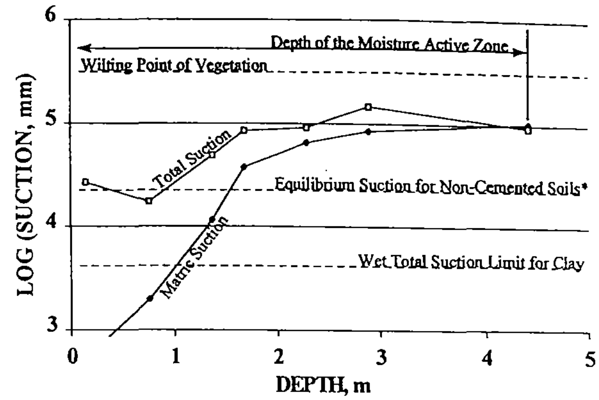
Figure 8 shows a profile of a soil with a high osmotic suction level but one that is not high enough to prevent the penetration of roots. The soil had high concentrations of soluble sulfates and underlay a pavement that had experienced repeated episodes of repeated distress. Consequently, although the borings were carried to a depth of 4.4 m, it did not reach the bottom of the moisture active zone.



*From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russan and Coleman, 1961)

Figure 8. Suction Profile Showing High Osmotic Suction.

Figure 9 illustrates the fifth criterion. The total suction at a depth of 3.4 m had nearly reached the equilibrium suction criterion (0.08 $\log_{10}$ |mm| suction per m) when the matric suction arrived at the same value. The total suction is not high enough to exclude the roots of vegetation.



* From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)

Figure 9. Suction Profile Showing the Total and Matric Suction Values Converging.

The increasing matric suction indicates an increase of cementation of the soil, while the small suction gradients indicate a low level of vertical moisture velocity. Together, these criteria indicate the bottom of the zone in which water will move at a quickened pace over that in an intact soil.. This criterion is based principally upon a maximum rate of vertical flow criterion according to which water is permitted to flow vertically upward or downward at a rate no greater than 100 mm/yr using the Mitchell hydraulic conductivity relation.

In cases where two or more criteria appear to apply, the more conservative one should be selected.

It is noted that the equilibrium value of soil suction as determined by the empirical relation with the Thornthwaite Moisture Index developed by Russam and Coleman (1961) is shown on each of the graphs in Figures 4 through 9. Although some of the suction values at the bottom of the moisture active zone are close to the suction value derived from the Russan and Coleman graph, notably in Figures 4 and 5, it is commonly observed that the empirical relation does not match the observed equilibrium suction well. This statement does not call into question the value of the empirical relation. Instead, it emphasizes the need to determine the equilibrium suction on a more fundamental basis which includes the desorption suction-versus-volumetric water content characteristic curve of the soil on any given site. Such a relationship was developed by D. A. Gay (Gay, 1993). The desorption characteristic curve for a soil is given by

$$|h| = \left[A \left(\frac{\theta_s - \theta_m}{\theta_m - \theta_r} \right) \right]^{\frac{1}{B}} \tag{13}$$

where θ_s = saturated volumetric water content

θ_r = residual volumetric water content

θ_m = the mean volumetric water content in a particular climate

A, B = coefficients which define the soil-water characteristic curve

The mean volumetric content in a given climate, θ_m from the following equation is substituted into the above equation to give the suction-vs-Thornthwaite Moisture Index relation in closed form.

$$\theta_m = \frac{\theta_{fc} - \theta_{dry}}{\left[1 + \dfrac{d_{am} - d_1}{d_1 \left(\dfrac{T}{T_1} \right)^{\gamma}} \right]} + \theta_{dry} \tag{14}$$

where d_{an} = the available moisture stored in the soil profile. This is normally taken as 300 mm for most clay soil profiles.

d_1 = $0.4949\ d_{am} + 0.305$

γ = $0.0393\ d_{am} + 1.357$

T_1 = $0.00627\ d_{am} + 59.536$

T = the Thornthwaite Moisture Index + 60

θ_{fc} = the volumetric water content at the field capacity moisture condition corresponding to a suction of (-9.8 k Pa or pG of 3.0 or - 1000 mm)

θ_{fc} = $0.88\ \theta_s$ approximately for clay soils

θ_{dry} = the volumetric water content at the controlling suction condition at the ground surface, $|h_{dry}|$

$$\theta_{dry} = \frac{\theta_s + \theta_r}{1 + \dfrac{1}{A} \left| h_{dry} \right|^{B}} \tag{15}$$

The two most common driest suction values found at the ground surface are when it is controlled by the wilting point of vegetation (+ 3100 kPa or -3.16×10^5 mm or pG of 5.5) or by evaporation from the soil surface ($+9.8 \times 10^4$ kPa or -10^7 mm or pG of 7.0). The values of θ_s, θ_r, A, and B define the soil-water characteristic curve. The Thornthwaite Moisture Index defines the long-term climate and the controlling dry suction defines the shape of the curve especially in the negative Thornthwaite Index Range. The available moisture depth, d_{am}, may be taken as 300 mm or it may be estimated from the amount of water stored in the soil between the wettest and driest steady state suction profiles with depth. Typical values of θ_s, θ_r, A, and B used in generating clay soil-water characteristic curves with substantial amounts of fine clay content are

$\theta_s = 0.50$

$\theta_r = 0.04$

A = 475 if $|h|$ is expressed in mm.

B = 0.50

These values, with a controlling dry suction of $+9.8 \times 10^4$ kPa (or -10^7 mm or a pG of 7) will produce larger values of equilibrium suction than can be determined with the empirical relation due to Russam and Coleman (1961). These larger values are closer to the suction values that are observed at depth in Figures 4, 6, 8, and 9. Use of the above equations together with simple methods of estimating the desorptive soil-water characteristic curve will make the determination of an equilibrium suction at depth a routine matter. It will also make the task of identifying those suction profiles which are controlled by a high water table, or a high osmotic suction or a cemented soil a more reliable one.

4.5 Transient Cases

As explained in previous references (Lytton 1992, 1994), design of most engineering structures should be based upon a change of suction between two suction profiles which represent a steady state of flow. The Post-Tensioning Institute design procedure (1980, 1996) is based upon an edge drying (center lift) and an edge heaving (edge lift) differential movement. The edge drying movement occurs between an equilibrium suction profile (vertical velocity is zero) and a profile beneath a covered area with steady upward flow, controlled by a vegetative suction (+3160 kPa, $-10^{5.5}$ mm, or pG 5.5) or an evaporative suction (9.8×10^4 kPa, -10^7mm, or pG 7.0) at the surface. The edge wetting movement occurs between an equilibrium suction profile and a profile with steady downward flow which is controlled by a surface suction at the wet limit for suction (+31 kPa, $-10^{3.5}$ mm, pG 3.5).

There are specific cases in which transient rather than steady state suction profiles will prove to be useful for design purposes. One of these is the equation for the variation of suction with depth caused by a cyclic suction at the surface. That equation was developed by Mitchell (1980) and has been presented earlier. This equation and variations of it can reliably predict the effects of lawn watering and seasonal rainfall and drying.

Other transient cases represent the extreme cases of constant ponding and constant evaporation or transpiration. These cases are rarely seen in the field and should be used sparingly. They, too, were developed by Mitchell (1980). The solution for the ponding case is

$$u(z,t) = u_o + \left[u_{e(z)} - u_o\right] erfc\left(\frac{z}{2\sqrt{\alpha t}}\right) \qquad (16)$$

where u(z,t) = the logarithm of the total suction in mm at the depth, z in mm, and at time, t in seconds.

$u_e(z)$ = the equilibrium logarithm of suction in mm at depth, z.

u_o = the constant logarithm of suction in mm at the surface

α = the unsaturated diffusivity in mm²/s, as defined by Mitchell (1980). The value of α ranges between 10^{-1} and 10^{-3} mm²/s.

The constant evaporation case is

$$u(z,t) = u_e(z) + \left[u_a - u_e(z)\right] erfc\left(\frac{z}{2\sqrt{\alpha t}}\right) - \left[u_a - u_e(z)\right] exp\left(rz + r^2\alpha t\right) erfc\left(\frac{x}{2\sqrt{\alpha t}} + r\sqrt{\alpha t}\right) \qquad (17)$$

where u(z,t) = the logarithm of the total suction in mm at depth, z, and time, t.

$u_e(z)$ = the equilibrium logarithm of suction in mm at depth, mm.

u_a = the logarithm of the suction in mm in the air above the soil.

r = the film coefficient of vapor transfer. This was found experimentally by Mitchell (1980) to be 0.054 mm^{-1}.

Another case of practical interest to design is the change of suction beneath a covered area immediately after construction. The transient equation for this case is

$$u(z,t) = u_e(\ell) + \Delta U \sum_{n=1}^{\infty} \frac{8}{(2n-1)^2\pi^2} * \cos\left[\frac{(2n-1)\pi z}{2\ell}\right] * \exp\left[-\frac{(2n-1)^2\pi^2\alpha t}{4\ell^2}\right] \qquad (18)$$

where $u_{e(l)}$ = the logarithm of the equilibrium suction in mm at the depth of the moisture active zone.

l = the depth of the moisture active zone in mm.

Δu = the change of the logarithm of suction in mm from the bottom of the moisture active zone to the top of it at the time of construction.

With a rising permanent water table, l will decrease with time.

These three cases will apply to most of the transient cases encountered in design.

4.6 Trees

The equations presented above provide a means of estimating the suction within the moisture active zone because they make use of the Mitchell formulation of hydraulic conductivity, which includes, in an approximate way, the effects of the smaller cracks in the soil in assisting the transmission of water.

The actual suction within a tree root zone changes rapidly with the seasons varying from nearly the wet limit of suction (+31 kPa, $-10^{3.5}$ mm, or pG 3.5) to the wilting point (+3100 kPa, $-10^{5.5}$ mm, or pG 5.5). Thus trees can engender both heave and shrinkage at the edge of a foundation or pavement. Another major problem created by trees is when they are cut down or removed prior to construction, leaving their root zones beneath the covered area. Because of construction normally proceeds during warm and dry weather, the severed tree root zone is at or approaching the wilting point. The suction beneath the covered area then approaches its equilibrium value, wetting up the tree root zone and causing heave.

4.7 Drainage

The effects of poor drainage may be represented for design purposes by using either the ponding transient condition or a steady state representation throughout the depth of the affected area of a suction level at the wet limit.

4.8 Slopes

In his Theoretical Soil Mechanics, Terzaghi (1963) used an elastic solution presented by A. E. H. Love (1927) to represent the stress state in an earth dam. The solution was for the stresses, strains, and displacements in an elastic wedge acting under its own weight. The solutions for displacements, translated to use elastic material properties that are more familiar are as follows:

$$u(x,z) = -\frac{(1+\nu)}{E}p + \frac{(1-\nu^2)}{E}q$$

$$v(x,z) = -\frac{(1+\nu)}{E}r + \frac{(1-\nu^2)}{E}s \qquad (19)$$

where

u(x, z), v(x, z) = the horizontal and vertical displacements

E, ν = the Youngs modulus and Poisson's ratio

and

$$p = 3ax^2 + 2bxz + cz^2$$
$$q = 3ax^2 + 2bxz + cx^2 + 6dxz$$
$$r = bx^2 + 2cxz + 3dz^2$$
$$s = 6axz + bz^2 + 2cxz + 3dz^2.$$

Referring to Figure 10, the coefficients a, b, c, and d are further defined as:

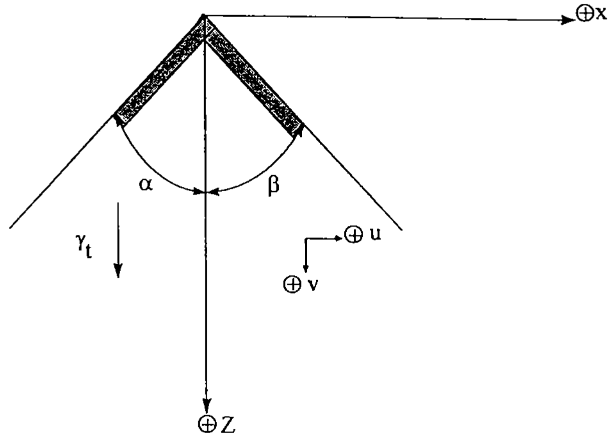


Figure 10. Sign Conventions for Love's Solution for an Elastic Wedge

$$a = \frac{\gamma_t}{6} \cdot \frac{\tan\beta - \tan\alpha}{(\tan\alpha + \tan\beta)^3}$$

$$b = \frac{1}{4}\left[\gamma_t - 6a(\tan\beta - \tan\alpha)\right]$$

$$c = -3a\tan\alpha\tan\beta$$

$$d = \frac{\tan^2\beta}{12}\left[-\gamma_t + 6a(3\tan\alpha + \tan\beta)\right] \qquad (20)$$

Setting the angle α to equal (β + π/2) gives a slope with a slope angle of (π/2 - β). Making use of the viscoelastic correspondence principle and of Schapery's approximate inverse LaPlace transform (Schapery, 1962, 1965) gives the equation for down hill creep displacements of these soils. The equations are as follows:

$$u(x,z,t) = \frac{1}{E_\alpha + E_1\Gamma(1-m)(2t)^{-m}} * \left[-(1+\nu)p + (1-\nu^2)q\right]$$

$$v(x,z,t) = \frac{1}{E_\alpha + E_1\Gamma(1-m)(2t)^{-m}} * \left[-(1+\nu)r + (1-\nu^2)s\right] \qquad (21)$$

where

E_α, E_1, m = the coefficients and exponent of the power law relaxation modulus of the soil

Γ (1 - m) = the Gamma function with the argument (1 - m).

ν = the Poison's ratio which is assumed to be constant

The slope as defined here is illustrated in Figure 11.

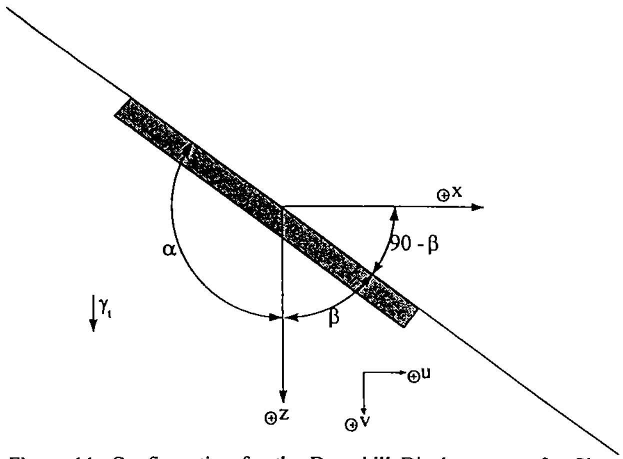


Figure 11. Configuration for the Downhill Displacement of a Slope with a Slope Angle of $(90-\beta)°$.

For this special case, the coefficients a, b, c, and d are as follows:

$$a = \frac{\gamma_t}{6}\ \frac{g^2}{n^3}$$

$$b = \frac{\gamma_t}{4}\left(1 - \frac{g}{n^3}\right)$$

$$c = \frac{\gamma_t}{2}\ \frac{g^2}{n^3}$$

$$d = \frac{\gamma_t}{12}\ \ell^2\left[\frac{g}{n^3}\left(\sin^2\beta - 3\cos^2\beta\right)\ 1\right]$$

$$\ell = \tan^2\beta$$

$$g = \sin\beta\ \ \cos\beta$$

$$n = \sin^2\beta - \cos^2\beta \tag{22}$$

The values of E_α, E_1, and m depend upon the level of suction in the soil and can be measured simply in a relaxation modulus test on the soil. Typical values of m, the exponent are between 0.10 and 0.50. The exponent can never be above 1.0. The displacement of the slope in the downhill direction is given by

$$w(t) = u(x, z, t)\ \sin\beta + v(x, z, t)\ \cos\beta \tag{23}$$

Downhill creep has caused serious problems to foundations and pavements and these equations provide a relatively straight forward way of estimating the down hill movement prior to construction. The equations are set up so that the origin does not move. Thus it should be set at the bottom of the slope and the displacements calculated for values of x and z which are both negative. An example of such calculations is shown in Figure 12.

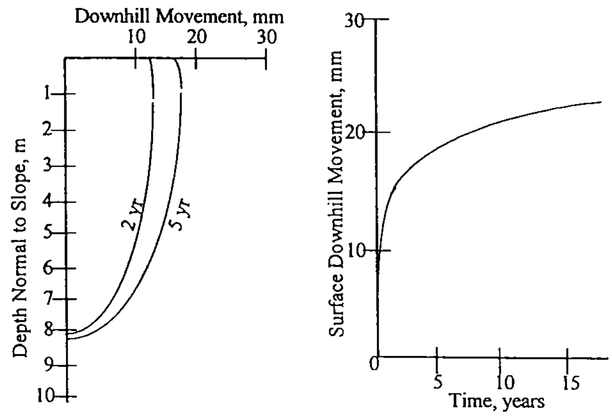


Figure 12. Typical Computed Downhill Creep Movements.
Soil Properties used in making these computation were measured in the laboratory and are as follows: E_α=37 kPa; E_1=3.8 x 10^5 kPa-$(s)^m$; m=0.24; ν=0.4; γ_t=18 kN/m³

5. DESIGN CRITERIA

The performance of engineering structures can be predicted using one or another of the methods outlined in the previous section of this paper. The design criteria that should be met by these structures can be compared with the predicted performance to determine whether the design being considered is adequate. If it is not adequate, another alternative is explored in the same way. This is the design process, one that has not, in general, been used in the design of engineering structures on unsaturated soils because the predictive methods were either unavailable or unverified with actual performance.

All engineering structures on unsaturated soils, are subject to the variations of suction at the soil surface due to weather, evaporation, vegetation drainage, and watering patterns. In designing these structures, recognition must be taken of the length of the time these structures must be in service, and of the severity of the weather patterns that may occur during the expected life of the structure. The return period in hydrologic events is appropriate to use in estimating the design criteria for foundations and pavements on these unsaturated soils.

As an example of this, the edge moisture variation distance e_m, that is used in the design of slabs-on-ground can be estimated using the e_m-versus-Thornthwaite Moisture Index (TMI) chart that was published with the Post-Tensioning Institute design manual (First Edition, 1980: Second Edition, 1996). These e_m-values were derived by back calculation from slabs which were performing successfully in San Antonio, Dallas, and Houston. None of the slabs were more than 10 years old at the time. It can be argued that the design values of e_m represent a 10-year return period. On the other hand, another set of e_m-versus TMI charts were presented by Lytton (1994). These charts were developed by finite element simulation using suction conditions inferred from weather data that covered up to 50 years. The finite element program, a coupled non-linear elastic and unsaturated diffusion transient flow program, had been calibrated to several years of field observations beside and beneath pavements. It can be argued that these charts represent a 50-year return period. The use of the Gumbel probability density function, which is commonly used to represent the probability of weather events, may be used to establish the risk level that is desired for design in accordance with the expected service life. The following table shows the risk level and corresponding return period that can be selected for design.

In the case of the PTI design procedure, these risk levels, as defined by the Gumbel distribution, can be used to interpolate design values of e_m between the 10-year chart in the PTI manual (10 percent risk) and the 50-year chart in the paper by Lytton (1994) (2 percent risk).

The equation relating the e_m-values for the 10-year and 50-year return periods to the design $e_m$value for another return period is given by

$$e_{m_r} = e_{m_{10}} + (e_{m_{50}} - e_{m_{10}}) \left(\frac{z_r - z_{10}}{z_{50} - z_{10}} \right) \tag{24}$$

The z_r scores are computed from the Gumbel cumulative probability distribution curve.

$$1 - \frac{1}{r} = e^{-\left(\frac{\rho}{Z_r}\right)^{\beta}} \tag{25}$$

where r = the return period in years

ρ, β = the scale and shape factors for the Gumbel distribution

Assuming that both ρ and β equal 1.0 in Equation (25), and that the 10-year and 50-year e_m-values are 1.37 m and 2.44 m respectively, the following table gives typical values of e_m for a range of return periods.

Return Period, Years	Risk, %	Z_r	e_m, m
100	1.0	99.5	3.77
80	1.25	79.5	3.24
50	2.0	49.5	2.44
40	2.5	39.5	2.17
25	4.0	24.5	1.77
20	5.0	19.5	1.64
10	10.0	9.49	1.37
5	20.0	4.48	1.24
2	50.0	1.44	1.16

The underlined e_m-values were used to construct the table of risk scores and e_m-values for other risk levels. The value of β will change to meet the probability patterns of local drought and rainfall occurrences. This illustrates how these two sets of design charts may be used to account for return periods in weather events. A common design period for residential and pavement construction is 20 years (5 percent risk). Similar procedures can be established for the other types of engineering structures.

Design requires a reasonable estimate of the maximum movements or pressures that can be expected during the expected service life of the engineering structure. In some cases, such as with vertical membranes that are used as moisture barriers or root barriers, the maximum movements or pressures that are exerted by an active soil can be reduced dramatically if the vertical membrane is extended deeply enough. A membrane depth of 1.25 m has been found to be a minimum practical depth to assure at least a 50 percent reduction in differential movements, when the source of the moisture or drying influence is at or near the ground surface (vegetation and drainage). In pavements, the annual total movement in any given wheel path has been found by field observations to result in an accumulation of roughness in that wheel path over time. Vertical barriers assist in reducing the rate of roughness increase in all wheel paths but their effectiveness depends upon how deep they are relative to the depth of the moisture active zone. It has been found that a vertical membrane (not an injected slurry) should be as deep as the moisture active zone until that zone becomes deeper than 2.5 meters. Vertical membranes deeper than that will continue to be more effective with increasing depth, but the increase will be at a diminishing rate.

Design does not need to be based upon precise transient solutions to the unsaturated moisture flow and movement problems although these solutions give the clearest understanding of what must be designed against. Instead, the transient solutions are always bounded by steady state envelopes with the appropriate wet and dry limits of suction applied as the controlling boundary conditions. These steady state solutions are easier to compute and being envelope values, are generally more useful in design than the transient results. The steady state computations are based upon a steady velocity of moisture flow both into or out of the soil, flowing between the steady suction at the base of the moisture active zone and the controlling wet and dry limit suction values at the surface.

The obvious exception to this general approach is the steady accumulation of water mounding above a permanent water table and below an extended covered area, a condition that is common in residual soils, and is commonly encountered in South Africa. In such cases, the controlling suction is at the water table (around +31 kPa or $-10^{3.5}$ mm or pG 3.5) and is at a rising elevation. At the surface, the controlling boundary condition is zero flow beneath an impervious boundary. The solution to the changing suction and movement patterns is transient and should not be based upon the erroneous assumption that the accumulating water above the permanent water table is somehow changing the Thornthwaite Moisture Index. The solution to this transient problem is provided earlier in this paper (Equation 18) and is due to Mitchell (1980).

6. NEEDED RESEARCH

There is beginning to be a broad-scale recognition that there are serious questions in the analysis and design of engineering structures on unsaturated soils that can only be answered with equally serious research. Practitioners, analysts, and designers should encourage the needed research and welcome the results as they are brought out. One of the reasons that such research would have been premature earlier is that previously there has not been a reasonably well-defined framework within which to systematically answer the questions. The international conferences on expansive soils and unsaturated soils since 1965 have contributed much to the formulation of this framework.

Briefly listed here are some of the subjects that fit within that framework and the questions that need definitive answers. The subjects are volume change, shear strength, lateral earth pressure, hydraulic conductivity, effects of viscoelastic properties of soils and particularly the effects of composition and compaction upon these properties, and the effects of weather return periods upon risk and reliability of engineering structures built on or in these unsaturated soils.

Volume change behavior of unsaturated soils needs to establish when the large strain and the small strain formulation should be used and how to account for the formation and presence of cracks in the soil mass. The effects of the change of osmotic suction needs to be explored systematically. Shear strength research needs to establish the mechanics basis for its relation to both matric and osmotic suction and the effects of cracks in the soil on shear strength. This is particularly the case with the case of shallow slope failures in which the transmission and storage of low suction water by cracks is a known major contributor. Lateral earth pressure formulations must be developed to account for the cracks, the transient suction in soil masses, and the effect of the viscoelastic mature of the soil on the lateral earth pressure coefficient. Hydraulic conductivity formulations need to be made to account for the effects of distributed cracks in the soil mass and of osmotic suction and dissolved inorganic salts and organic compound on the rate of flow, both in liquid and vapor form. Constitutive equations of unsaturated soils that take into account the composition (percent water, solids, and air) and the effects compaction on its viscoelastic properties need to be developed. Weather patterns for both drought and rainfall are already known and the characteristic values of ρ and β in the Gumbel distribution may already be catalogued by meteorologists or hydrologists. If so, this information needs to be made available to designers in practice. Finally, nondestructive or small aperture testing instruments need to be developed to permit more rapid and precise determinations in the field of these important characteristics of unsaturated soils: the components of suction, water content and unit weight, and the density of the crack fabric, the stiffness and viscoelastic properties of these soils.

7. CONCLUSIONS

The culmination of successful research is the formulation of a sound mechanics framework for the behavior of unsaturated soils in the field, laboratory and field, instruments to measure the relevant properties, an accurate understanding of and an ability to use with confidence the important relationships by designers and practitioners, rational design criteria that are attuned to this overall framework, and finally, successful application of these to achieve predictable performance at desired levels of reliability. Measured by this description, we have more to do in many areas of unsaturated soil testing, analysis, and design. The importance of the problems that are encountered by engineering structures on unsaturated soils in all climates, and the need for rational and achievable design criteria and for methods of accurately predicting future performance are becoming clearer with time and experience. Conferences such as this will assist greatly in moving toward the culmination described above.

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Estimating Soil Swelling Behavior
Using Soil Classification Properties

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Abstract

There exists a need for a method of estimating soil swelling behavior for the design and practitioner community. Desirable method attributes include; low cost, reasonable accuracy and technical soundness. The method should be usable for a wide range of soil types. Practitioners should be able to use local soils testing services to get the data required by the method. The NRCS Soil Survey Laboratory (SSL) soils database is a large, quality controlled resource that provides the data for the development of such a method. The presented method allows for the estimation of the suction compression index using Atterberg limits, particle size classification, and the coefficient of linear extensibility (COLE) values as contained in the SSL database. Using a 6500 sample subset of the SSL database, a series of charts are presented yielding suction compression index values for various mineralogically based groups. The proposed method provides quick and stable prediction of an important soil property using low cost and commonly available soil test procedures.

Introduction

There exists a need for a method that provides geotechnical practitioners with an estimate of soil swell characteristics as a part of the design process for building slab and mat foundations, piers and other support structures. Desirable attributes of such a methods include; low cost, straightforward to use, technically supportable, and reasonably accurate. In addition the method should provide results for a broad

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geographical coverage and provide answers using analyses within the capability of most conventional geotechnical laboratories.

Presented herein are the results of an examination of the use of Atterberg limits, particle size information, and cation exchange activity to produce such a predictive method.

Earlier Predictive Methods

The use of Atterberg limits as predictors of soil behavior has been common since their development. The Testing is relatively inexpensive, re-producible, and fast compared to many other tests. Given this long history, there is wide geographic coverage for test results.

Holtz and Gibbs (1956) developed a soil swell classification shown in Table 1 using index tests. Pearring (1963) used cation exchange capacity, CEC, and plasticity as two parameters to classify soils as to a predominant mineral type. Pearring normalized these two parameters based on the percent fine clay content. This normalization yielded two new parameters, the activity ratio (Ac) and the cation exchange activity (CEAc) as follows;

$$\mathrm{Ac} = \frac{\mathrm{PI\%}}{\dfrac{(\% - \mathrm{2micron})}{(\% - \mathrm{No.200\ sieve})} \mathrm{x100}}$$

$$\mathrm{CEAc} = \frac{\mathrm{CEC}\dfrac{\mathrm{millieq.}}{\mathrm{100gm\ of\ dry\ soil}}}{\dfrac{(\% - \mathrm{2micron})}{(\% - \mathrm{No.200sieve})}\ \mathrm{x\ 100}}$$

Figure 1 illustrates the classification developed by Pearring (1963).

Seed et. al. (1962) developed a chart based on Ac and the percent clay fraction. The chart is shown as Fig. 2. Snethen et al. (1977) re-evaluated criteria for predicting soil swell and found that the soils liquid limit, plasticity index and soil suction at natural moisture content were the best indicators of potential swell. The resulting classification system is shown in Table 2. Lytton (1977, 1994) presented an expression relating the volumetric change in a soil sample due to changes in water potential. The relationship includes both water potential and stress terms as follows;

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)$$

where;

h_i, h_f are initial and final water potential,

$_i$, $_f$ are normal stress terms, and

$^a{}_h$ and ^{a} are the matric suction compression index and the mean principal stress compression index respectively.

McKeen (1981) used a mineralogical classification similar to that of Pearring (1963), defining regions charted against Ac and CEAc axes. For each of the identified zones, a value for the corresponding suction compression index is given after being adjusted to a 100% fine clay fraction. The actual suction compression index for the soil portion finer than the No. 200 sieve is proportional to the 100% fine clay index by the following equation;

$$\gamma_h = \gamma_{100}\left[\frac{\% - 2\text{ micron}}{\% - \text{No.200 sieve}}\right]$$

The COLE test represents the fractional change in a clod sample resulting from changes in moisture content. The classification chart, including predicted COLE values, is shown in Fig. 3. Hamberg (1985) updated the classification chart and included an adjustment for clay percentage as shown in Fig. 4. The approach continued to be refined in Nelson and Miller (1992) that produced a more simple general classification scheme using CEAc and Ac axes as shown in Fig. 5.

The NRCS Database

The United States Department of Agriculture, Natural Resources Conservation Service has created a national database of soil samples and test results. At the present time there are data for more than 25,000 soil pedons from all 50 states, other U.S. territories and several foreign countries. The stated objective of the program is to obtain representative samples of soils throughout the United States and its territories to provide a consistent analytical view of these soils chemical, mineralogical, and physical characteristics. Each soil sample was collected by experienced soil scientists at each horizon for future analyses. Different methods of sampling were used depending on the specific tests to be used. Clod samples were collected at each horizon for COLE determinations, for example.

Data from these analyses are compiled by the Soil Survey Laboratory (SSL) of the National Soil Survey Center. Most of the data in the present database were obtained over the last 40 years with approximately 75% of the data being obtained in the last 25 years. The SSL database may be accessed online at http://www.statlab.iastate.edu/soils/ssl. The database is also available on CD-ROM from the SSL. The CD-ROM version is in the form of a relational database approximately 200 MB in size containing the results of analyses on approximately 130,000 soil samples.

Data Analysis

The SSL database as selected as the basis for a re-examination of soil swell behavior as it is affected by certain index properties. SSL database features that

Table 1. Expansive Soil Classification from Holtz and Gibbs (1956)

Colloid Content (% minus .0001 mm)	Plasticity Index	Shrinkage Limit	Probable Expansion (%Vol)	Degree of Expansion
>28	>35	<11	>30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
<15	<18	>15	<10	Low

Table 2. Expansive Soil Classification based on Atterberg Limits from Snethen et al. (1977)

LL (%)	PI (%)	Natural Soil Suction	Potential Swell %	Potential Swell Classification
>60	>35	>4	>1.5	High
50-60	25-35	1.5-4	0.5-1.5	Marginal
<50	<25	<1.5	<0.5	Low

were important include; wide geographic coverage, the quantity of analyses available and extensive quality control in sampling, analysis and reporting. For this study, the SSL database was filtered to retain only those records that contained non-null results for the following tests;

Liquid limit
Plasticity index
Plastic limit
Cation exchange capacity
Coefficient of linear extensibility
% passing 2 micron
% passing No. 200 sieve

This data filtering produced a subset of the data containing approximately 6400 records. These data are shown in Fig. 6. The figure illustrates the broad distribution of the data. Next, the data records were partitioned according to Fig. 7 based on Casagrande (1948) and the Holtz and Kovacs (1981) mineral classification chart.

This partitioning step resulted in eight separate data groups, each representing a group with some mineralogical similarity. For each record a matric suction index was calculated according to the following expressions,

$$\gamma_h = \left(\frac{\gamma(\text{swellingcase}) + \gamma(\text{shrinkingcase})}{2} \right)$$

$$\gamma(\text{swellingcase}) = \left[\left(\frac{\text{COLE}}{100}+1\right)^3 - 1\right]$$

$$\gamma(\text{shrinkingcase}) = \left[1 - \frac{1}{\left(\frac{\text{COLE}}{100}+1\right)^3}\right]$$

The calculated (average) suction compression index was then adjusted to a 100% fine clay content.

Data for each of the eight separate mineralogical groups was then plotted as contoured surfaces on Ac-LL/%fine clay axes as shown in Fig. 8 through 15. These contoured data were created using a kriging algorithm per Cressie (1990). No explicit smoothing interpolation was used in creating the plotted surfaces. Table 3 provides summary statistics for each of the eight data groups. More simple versions of these surfaces have been prepared for Groups I though IV, TBPE (2000) and are shown as Fig. 16a to 16d.

Conclusions

The SSL database provides a rich source of analytical data for researchers and practitioners alike. The dataset may well be one of the largest available for the examination of wide ranging soil characteristics.

The method developed herein represents a refinement of earlier methods. The method builds on these earlier methods in that it is consistent in the use of low cost and easily available testing methods (Atterberg limits and soil particle size distributions) to predict soils properties and behavior. The method is stable in the sense that each mineralogical zone or group is explicitly defined, thus no arbitrary distinctions can affect the results. Within each group, the practitioner needs only the liquid limit, plasticity index and the fine clay fraction (%) to get an estimate of the suction compression index. The suction compression index can then be explicitly to calculate shrink and swell behavior using previously published methods as Lytton (1977, 1994). Small changes in soil index properties result in small changes in the derived suction compression index within each mineralogically based group. The proposed method, therefore, provides quick and stable prediction of an important soil property using low cost and commonly available soil test procedures.

Table 3. Summary Statistics for Suction Compression Indices; Minimum, Maximum and Percentile Values

Zone	1	2	3	4	5	6	7	8
minimum	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.00
25%	0.05	0.04	0.03	0.03	0.03	0.02	0.04	0.03
50%	0.09	0.08	0.07	0.06	0.05	0.05	0.08	0.07
75%	0.14	0.12	0.12	0.10	0.09	0.07	0.14	0.14
95%	0.21	0.20	0.20	0.20	0.19	0.17	0.32	0.31
maximum	0.44	0.46	0.48	0.44	0.48	0.48	0.49	0.47
No. of Data Pts.	523	1328	2534	991	266	166	266	302

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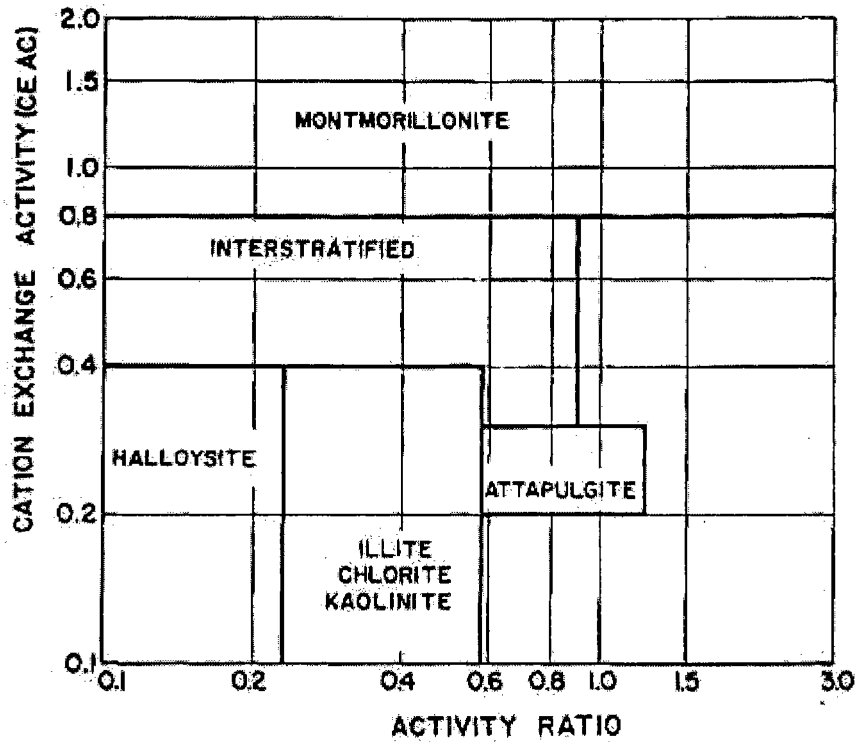


Figure 1. Mineralogical Classification From Pearring (1963)

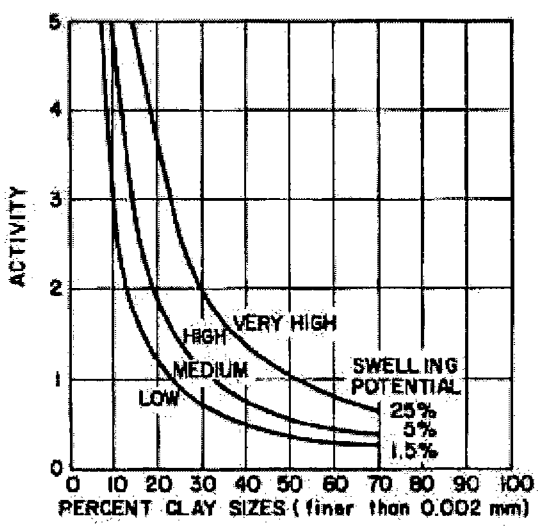


Figure 2. Soil Swell Potential Based on Size Fraction and Activity from Seed (1962)

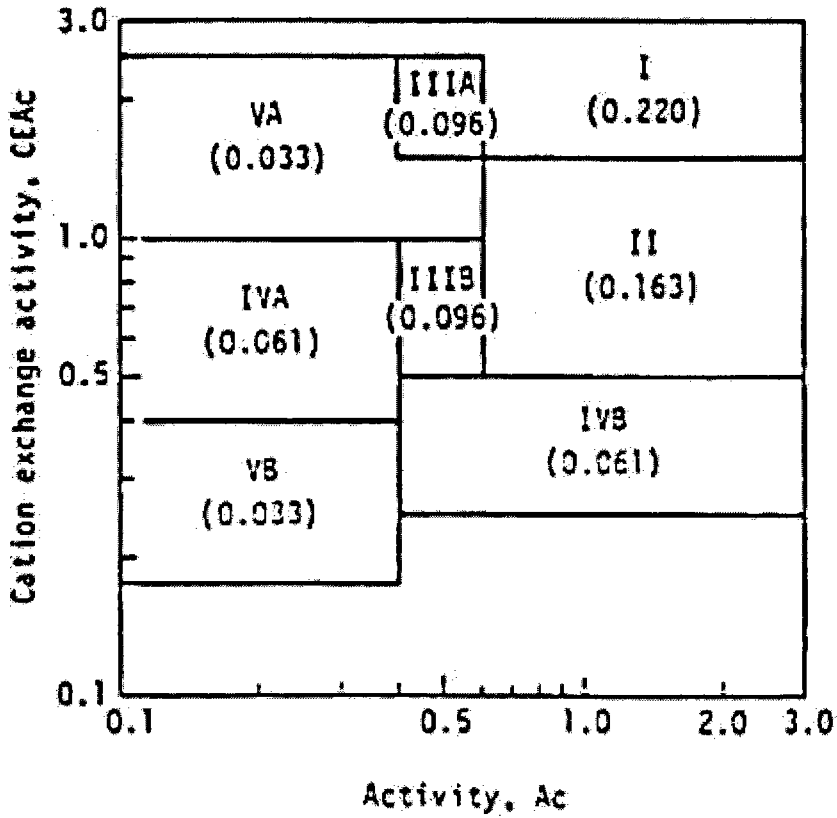
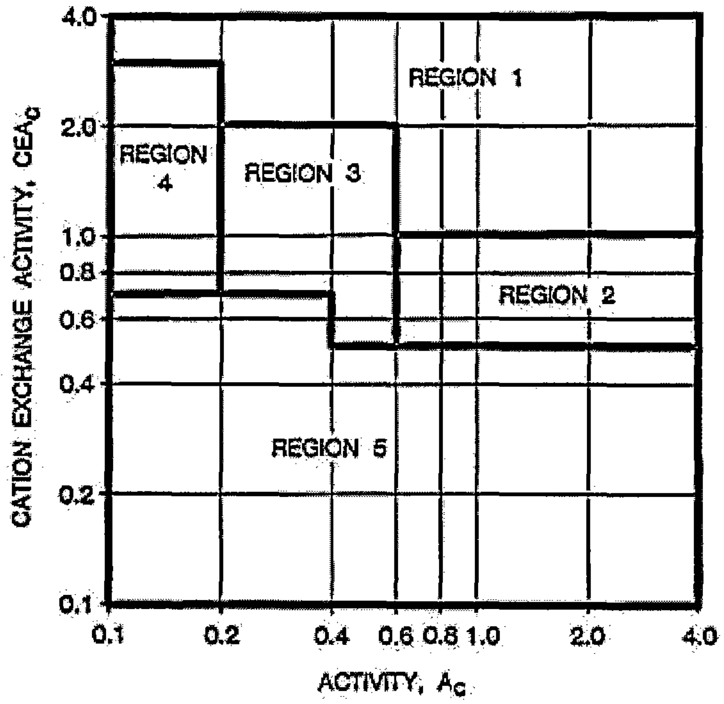


Figure 3. Classification Chart for COLE Values from McKeen and Hamberg (1981)



Region	A	B	n	r^2
1	0.23	-1.46	20	0.79
2	0.20	-1.14	45	0.77
3	0.14	-0.71	93	0.72
4	0.13	-0.31	21	0.51
5	0.09	-0.96	20	0.87

Figure 4. COLE Values bAsed on CEAc, Ac and Fine Clay from Hamberg (1985)

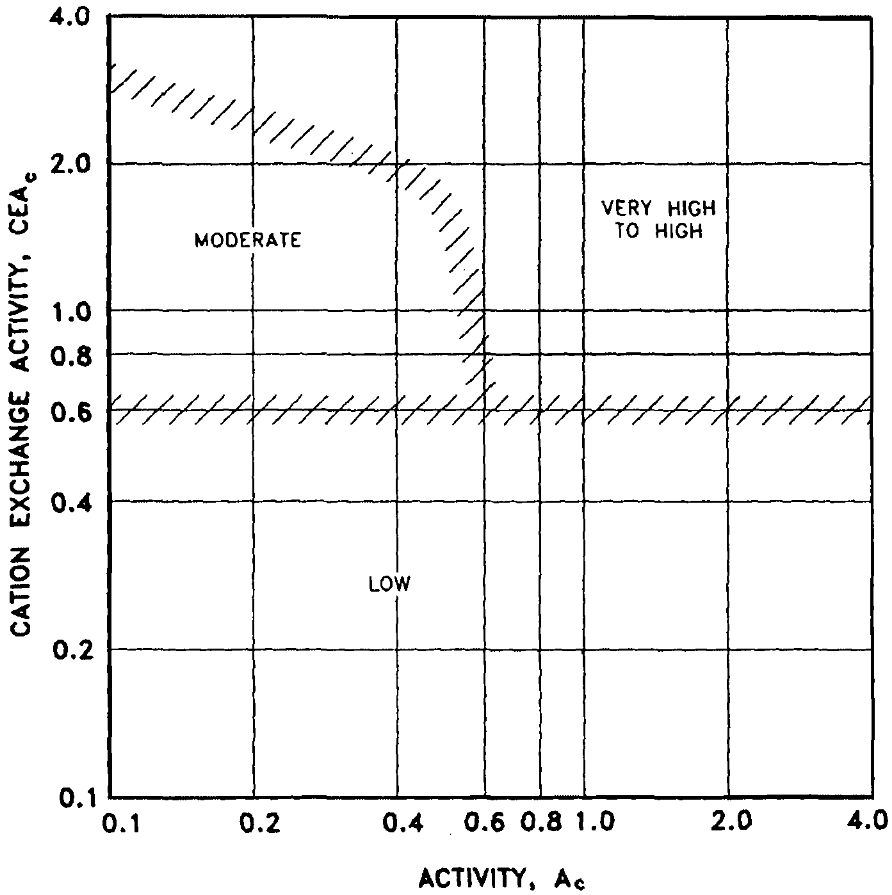


Figure 5. Expansion Potential as a function of CEAc and Ac from Nelson (1992)

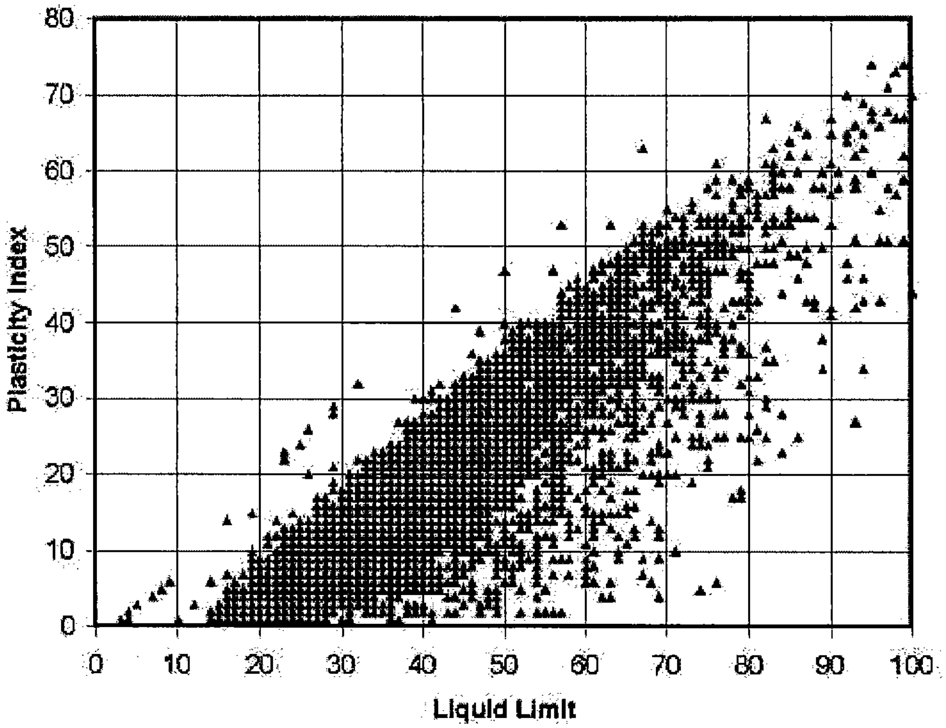


Figure 6. Data Set (6500 records) for Soil Compression Index Calculations

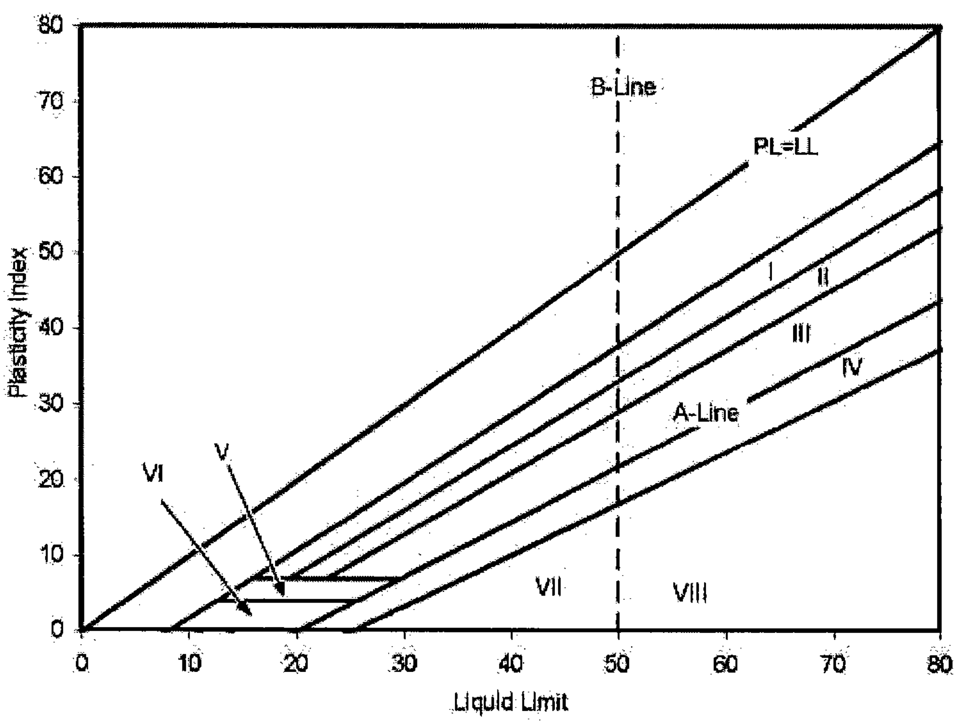


Figure 7. Data Filter for Partioning Data base on Mineralogical Types after Casagrande (1948) and Holz and Kovac (1981)

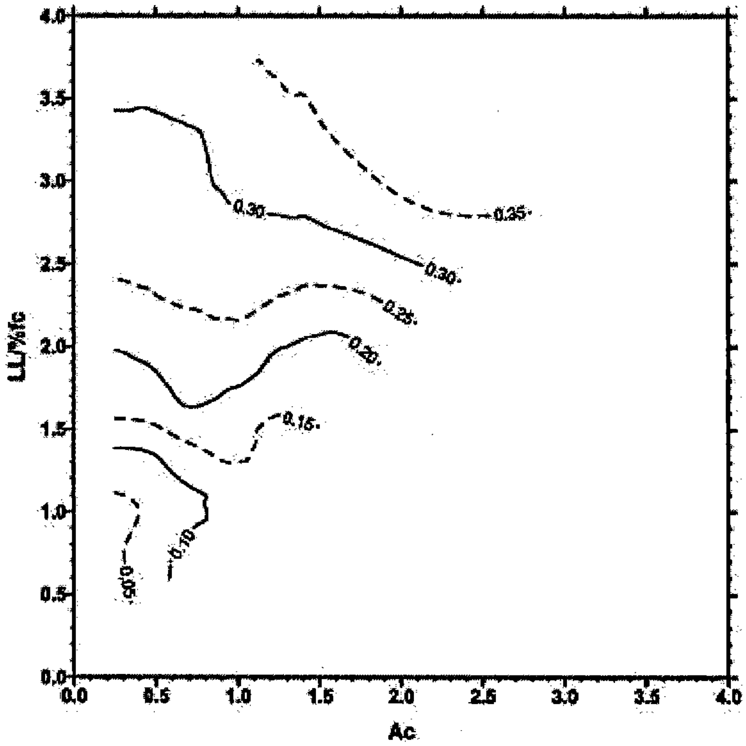


Figure 8. Predicted Soil Compression Index Values for Zone I

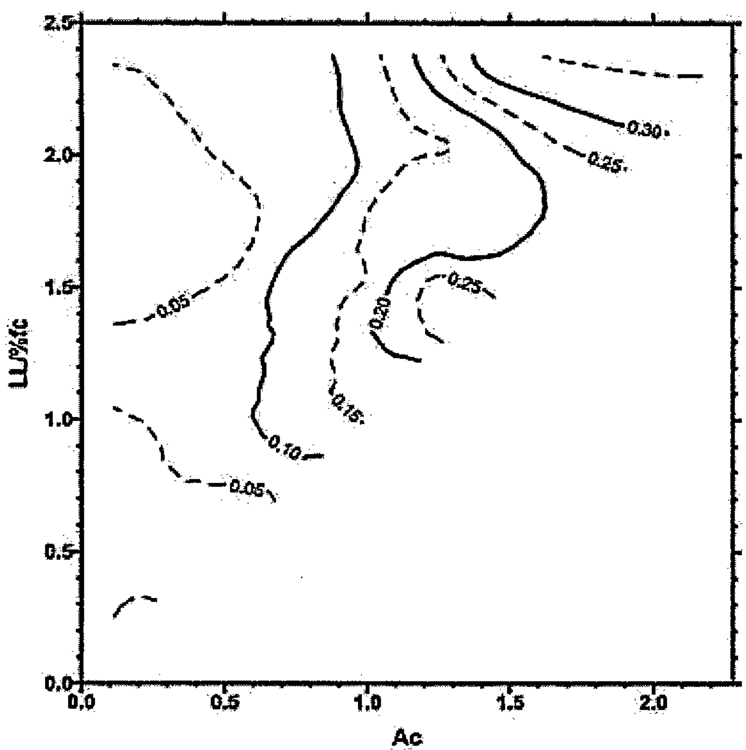


Figure 9. Predicted Soil Compression Index Values for Zone II

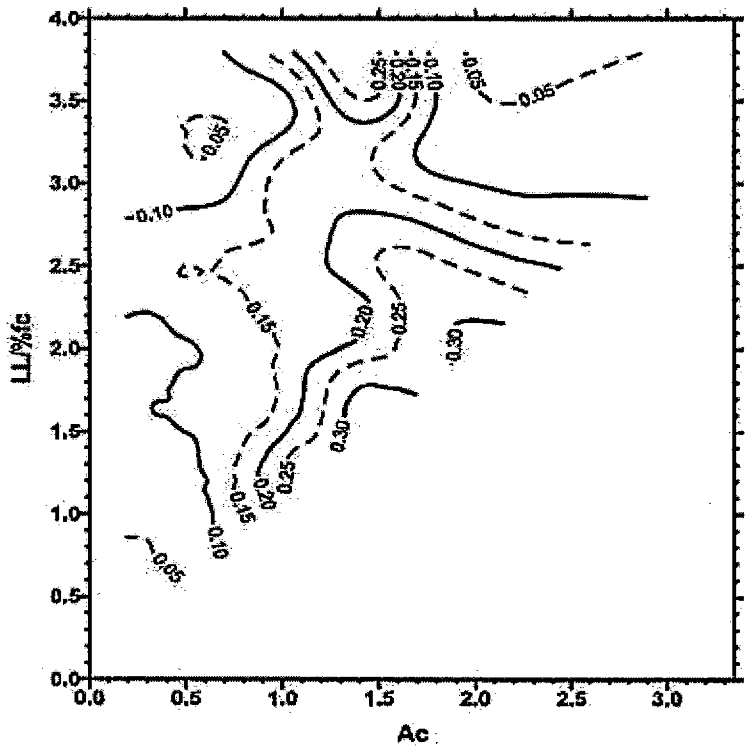


Figure 10. Predicted Soil Compression Index Values for Zone III

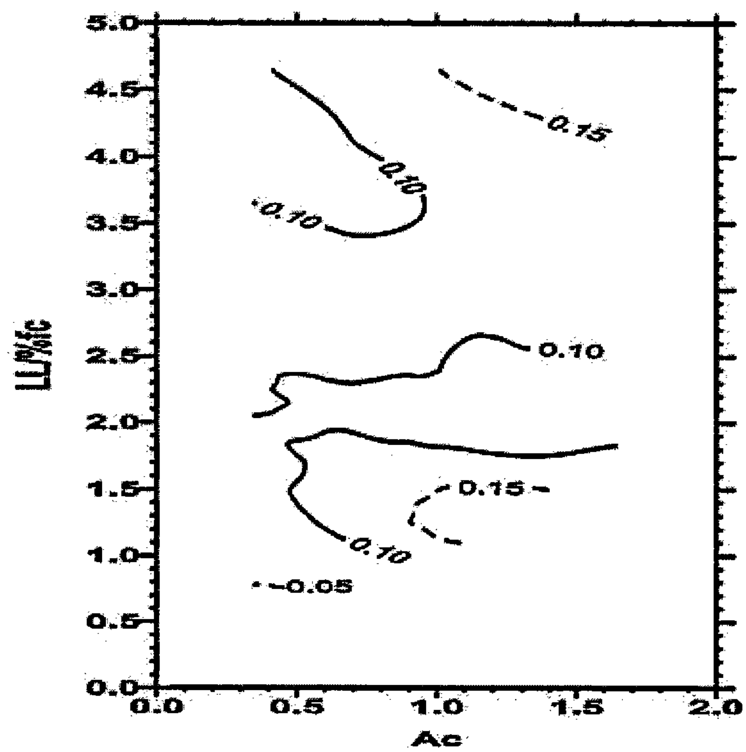


Figure 11. Predicted Soil Compression Index Values for Zone IV

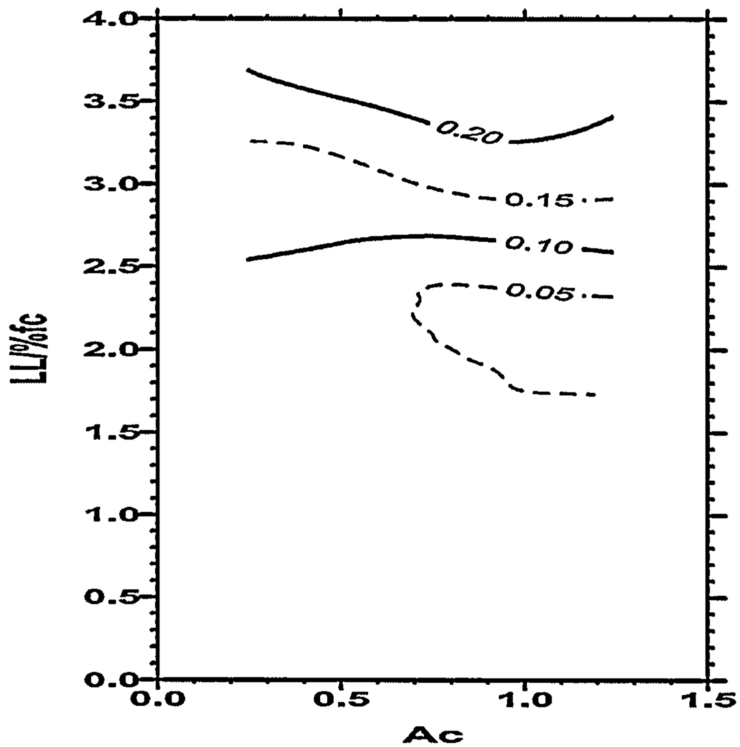


Figure 12. Predicted Soil Compression Index Values for Zone V

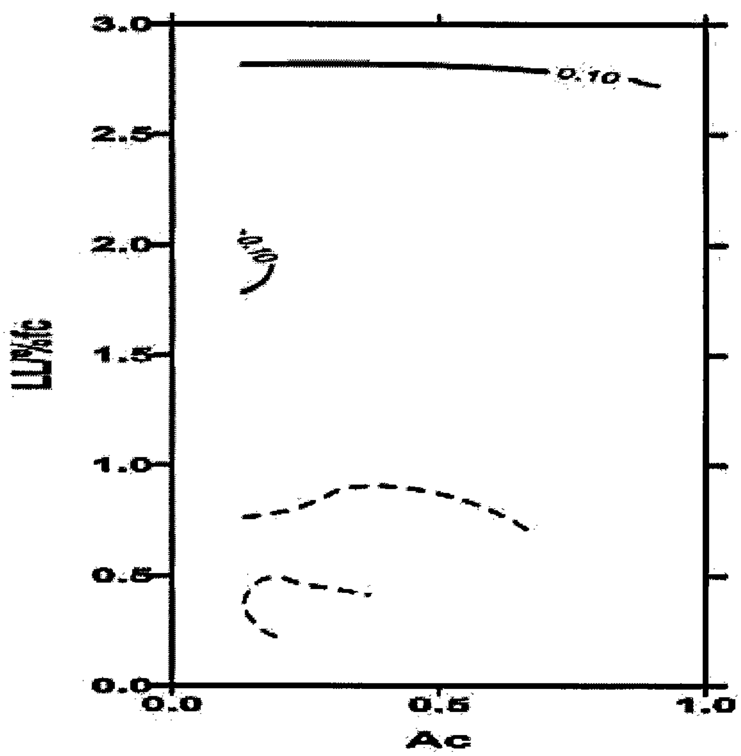


Figure 13. Predicted Soil Compression Index Values for Zone VI

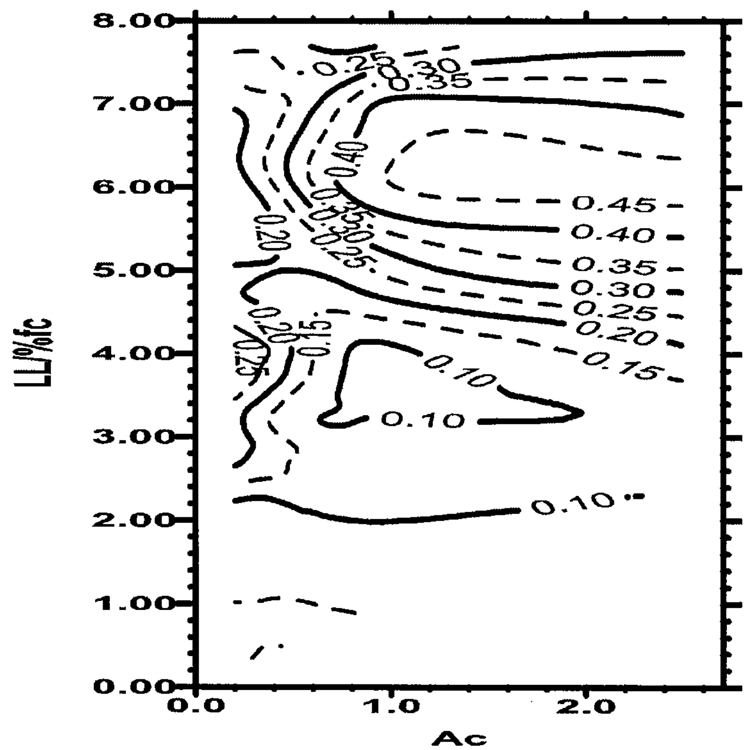


Figure 14. Predicted Soil Compression Index Values for Zone VII

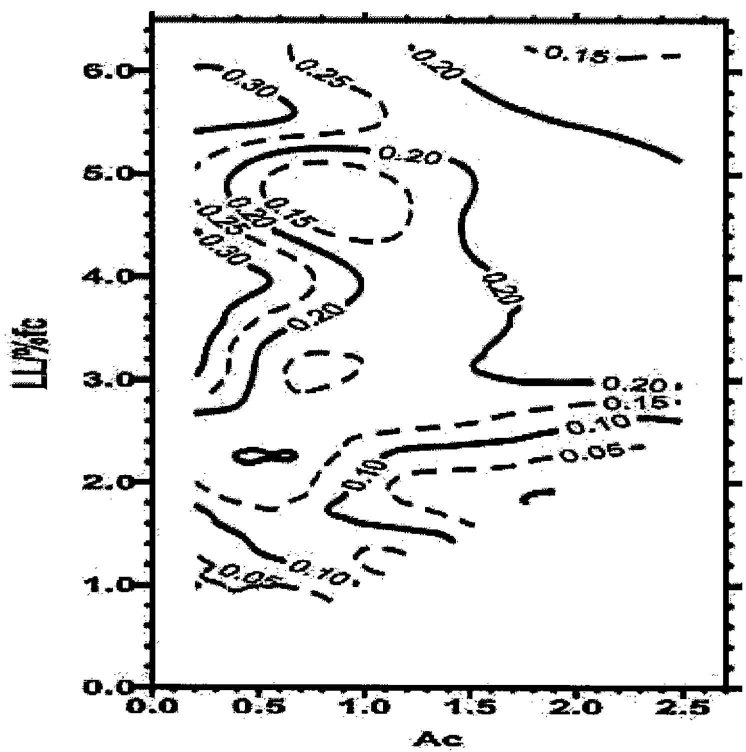


Figure 15 Predicted Soil Compression Index Values for Zone VIII

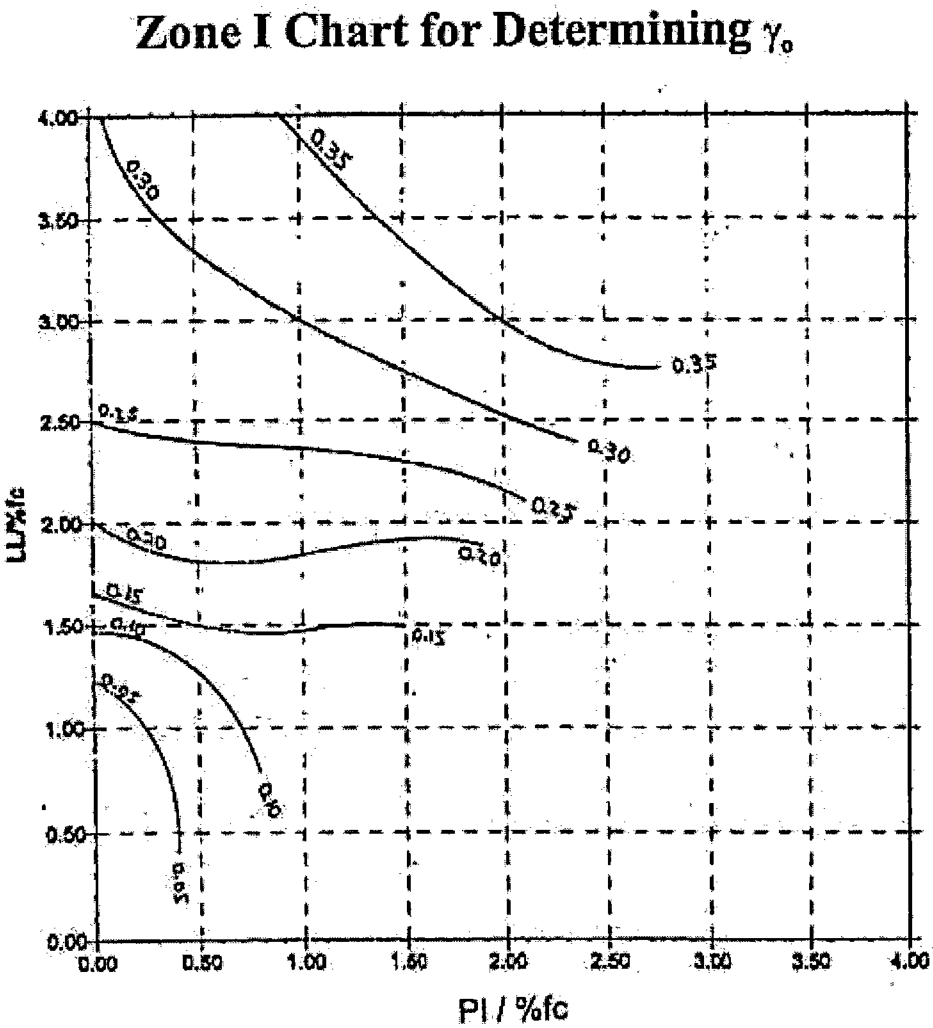


Figure 16a. Modified Soil Compression Index Surfaces from TBPE (2000)

Zone II Chart for Determining γ_o

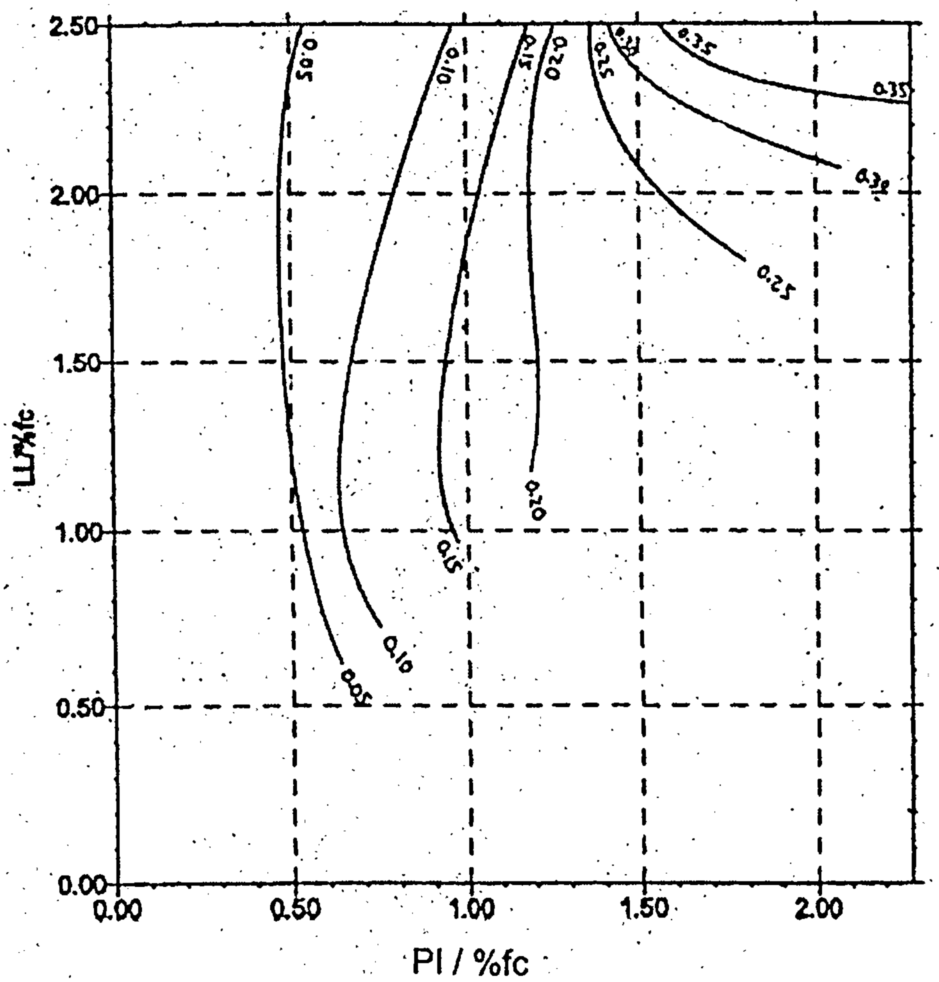


Figure 16b. Modified Soil Compression Index Surfaces from TBPE (2000)

Zone III Chart for Determining γ_o

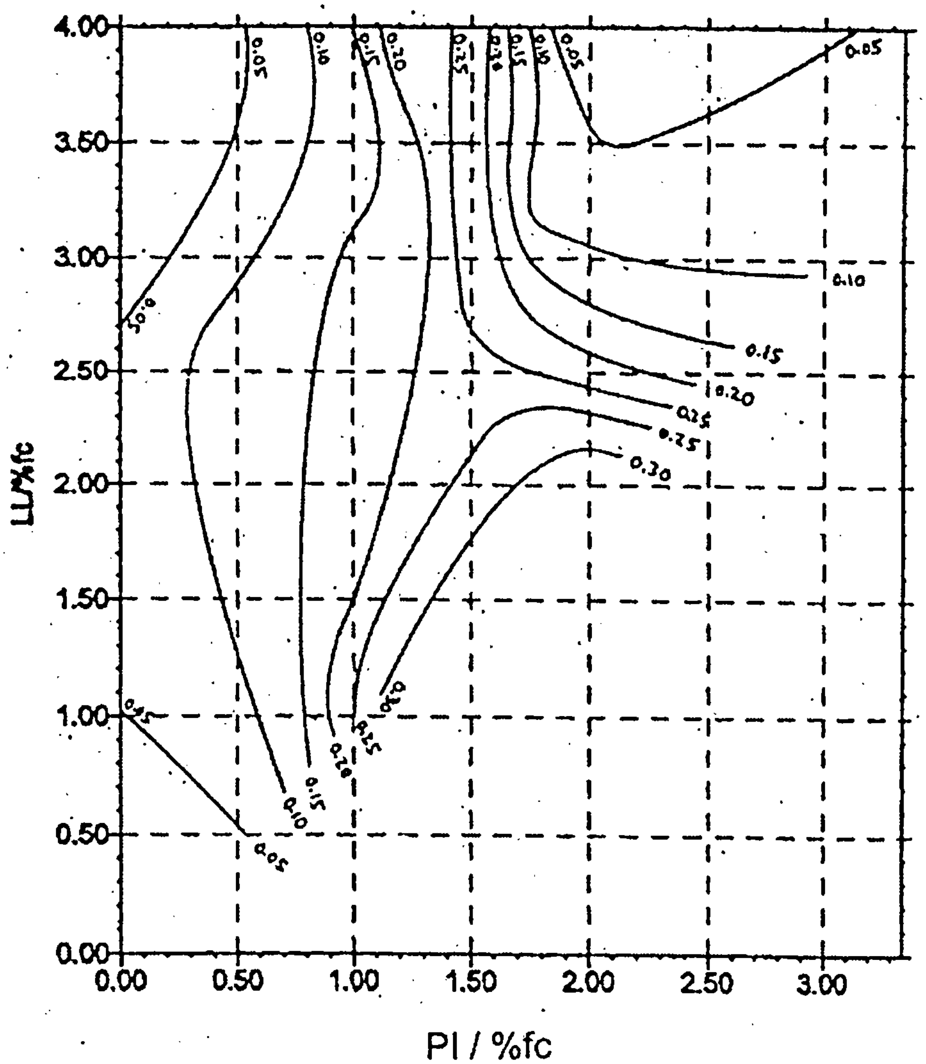


Figure 16c. Modified Soil Compression Index Surfaces from TBPE (2000)

Zone IV Chart for Determining γ_o

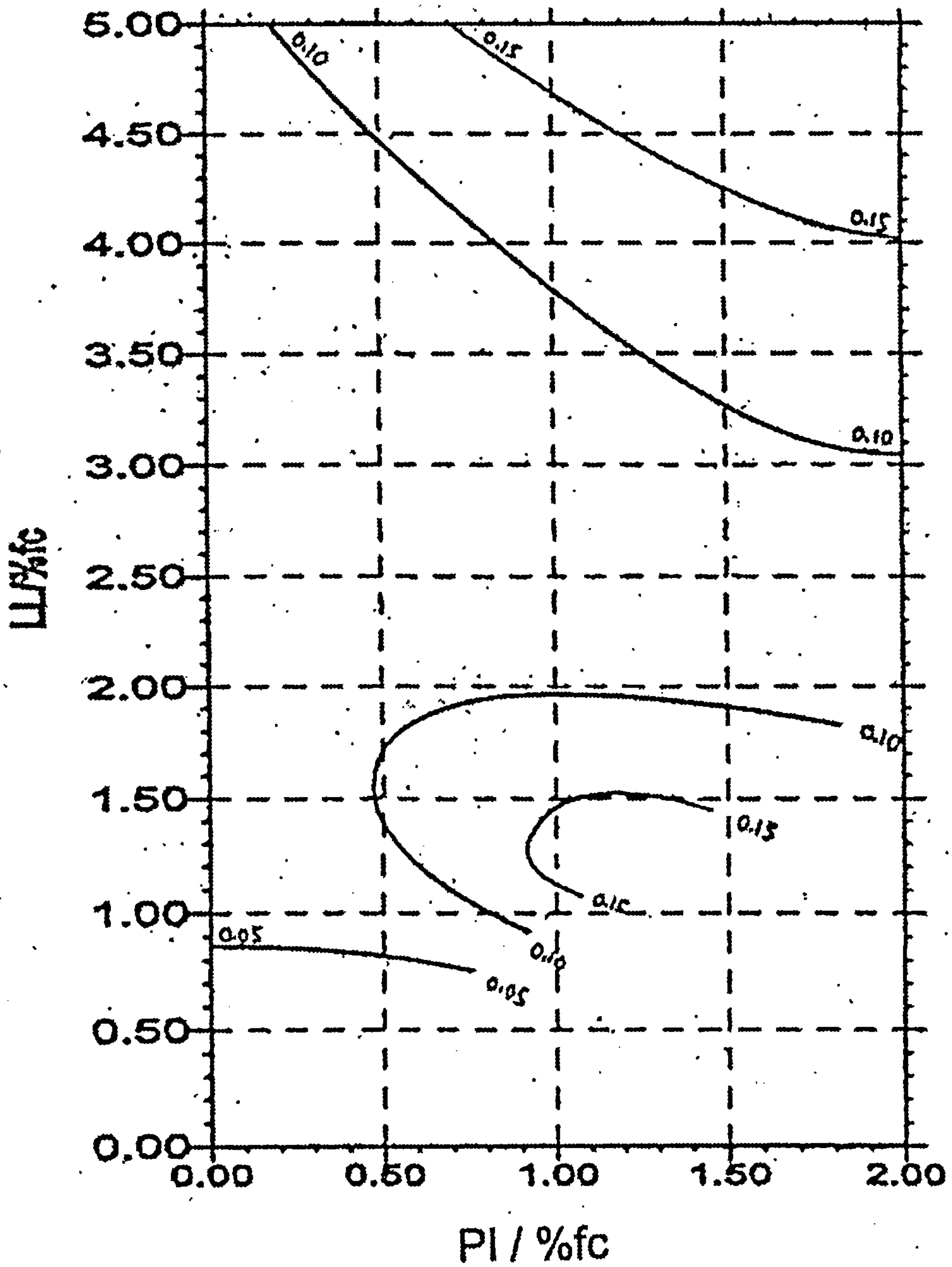
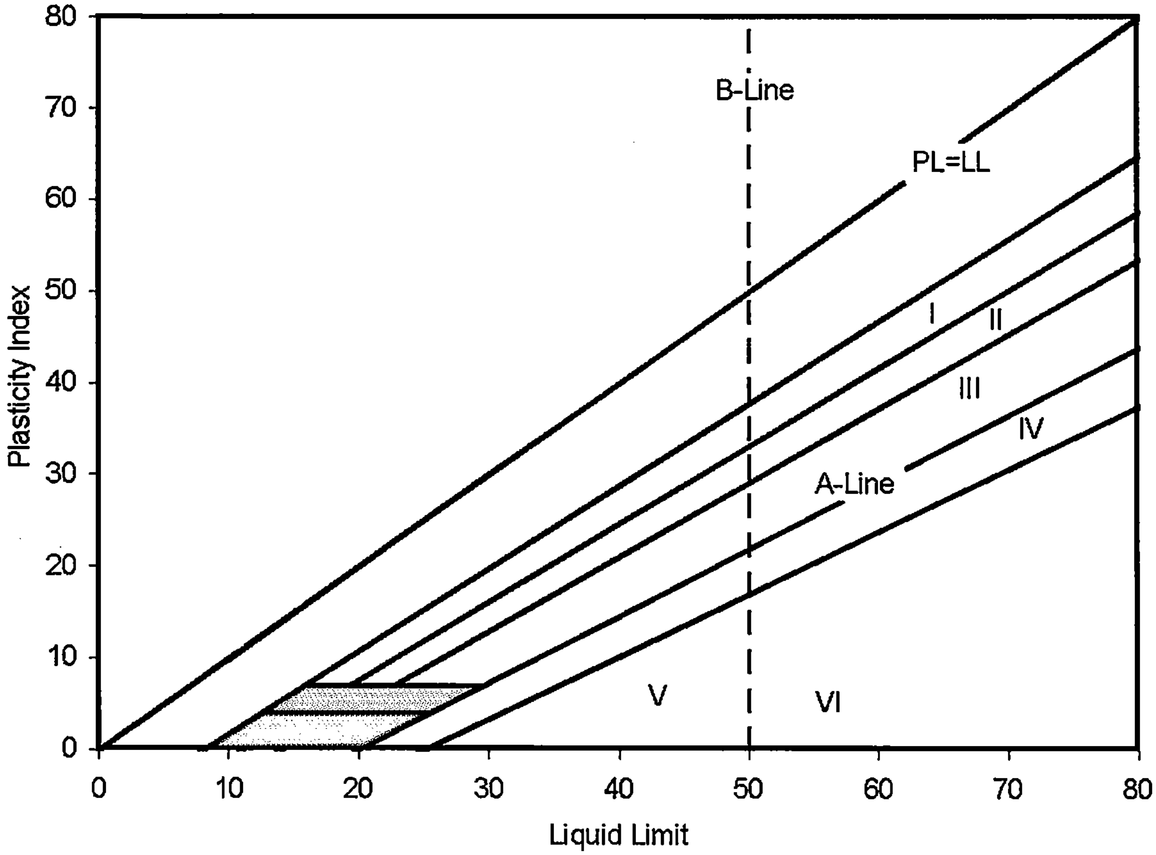


Figure 16d. Modified Soil Compression Index Surfaces from TBPE (2000)



B-Line
PL=LL
I
II
III
IV
A-Line
V
VI
Plasticity Index
Liquid Limit
0
10
20
30
40
50
60
70
80

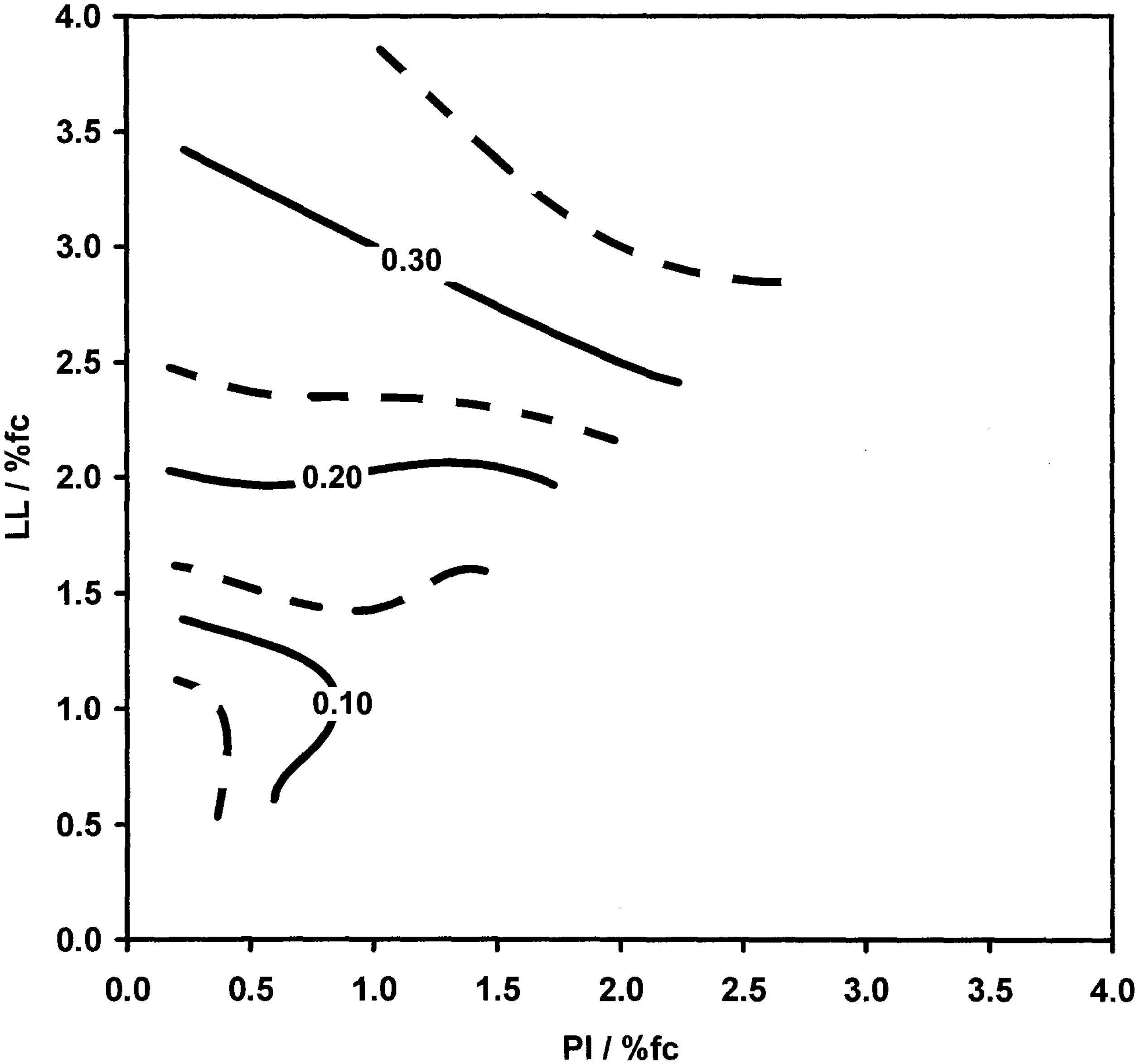


Figure 1. Predicted soil compression index values, Zone 1

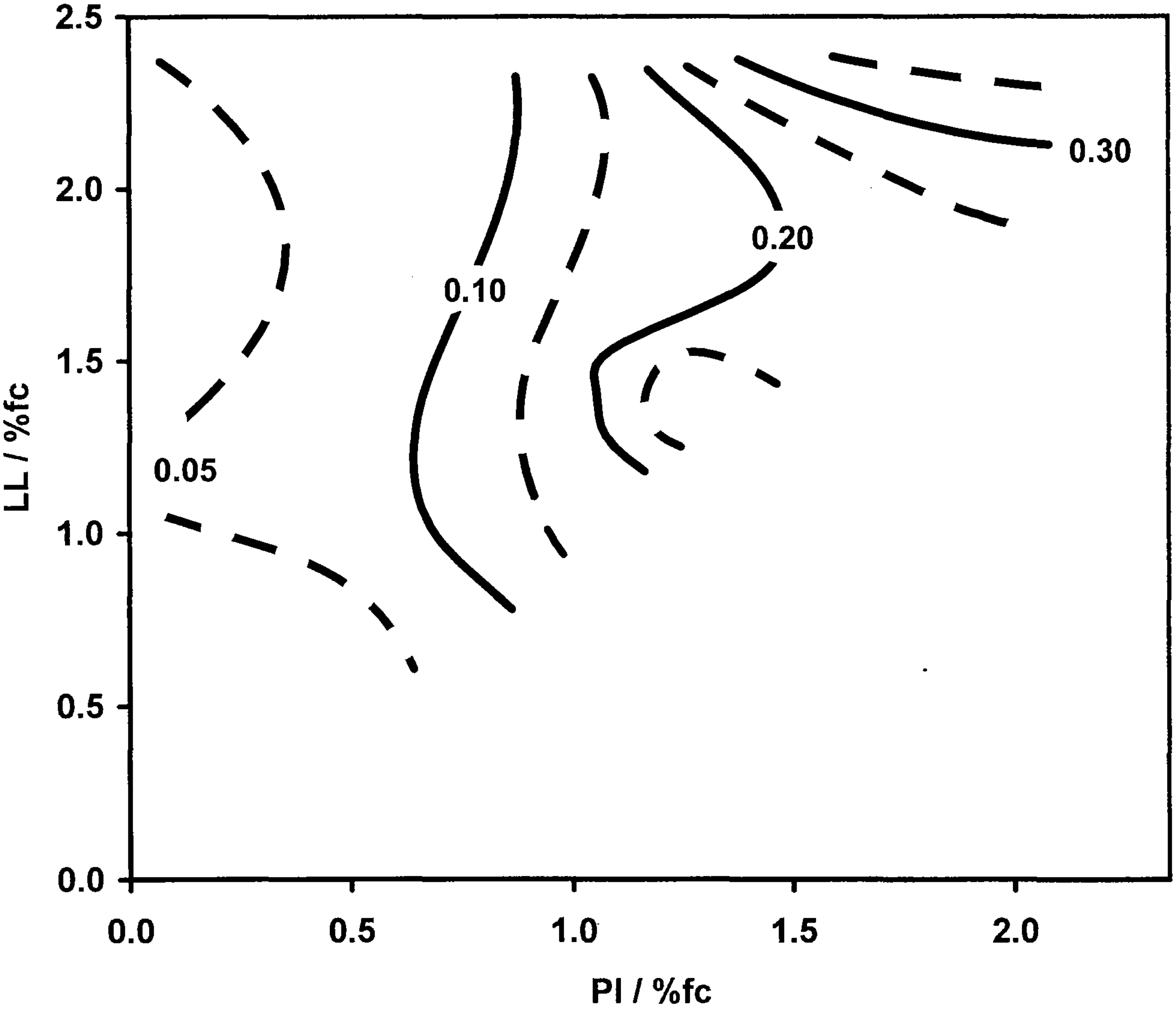


Figure 2. Predicted soil compression index values, Zone 2

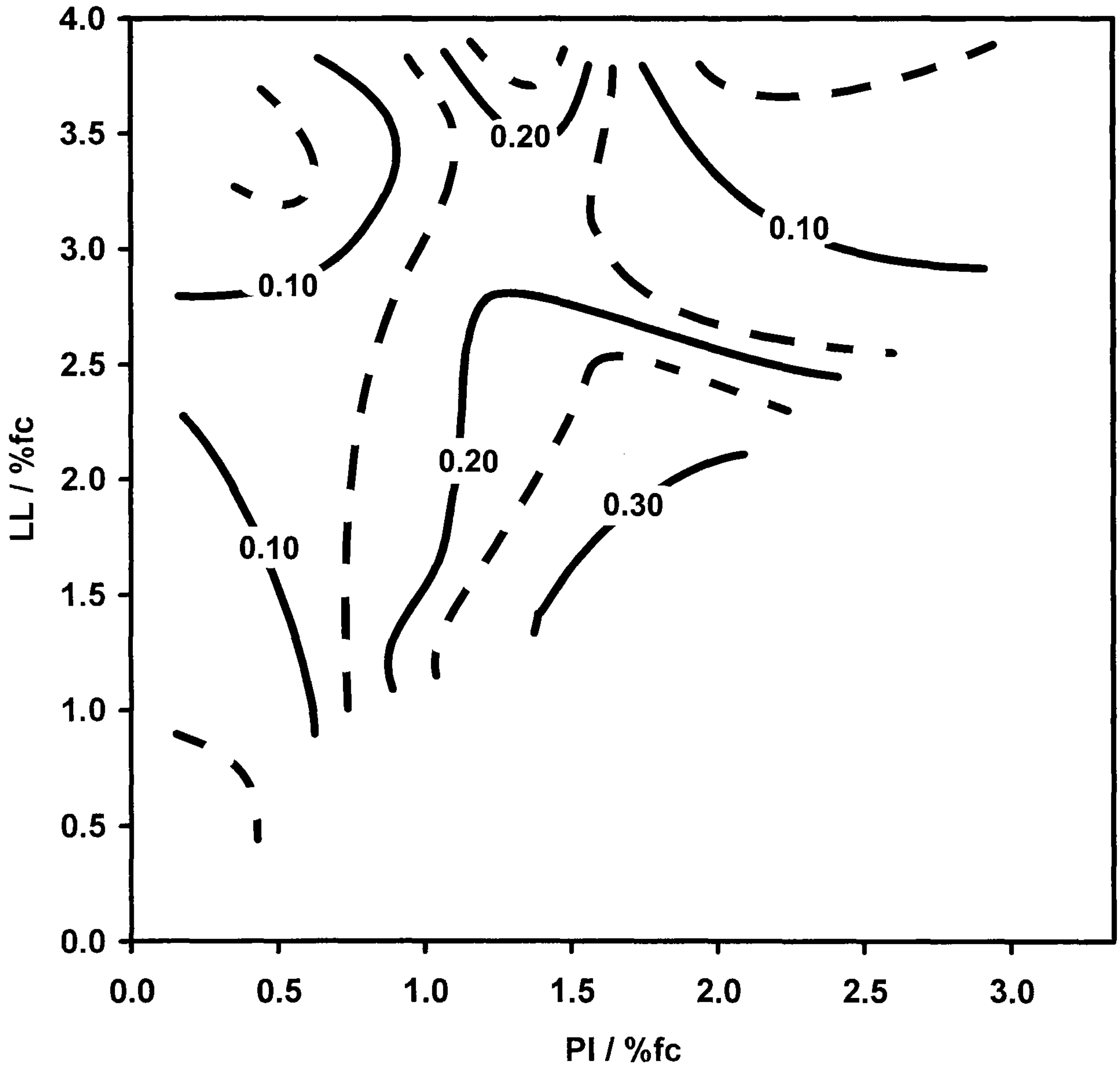


Figure 3. Predicted soil compression index values, Zone 3

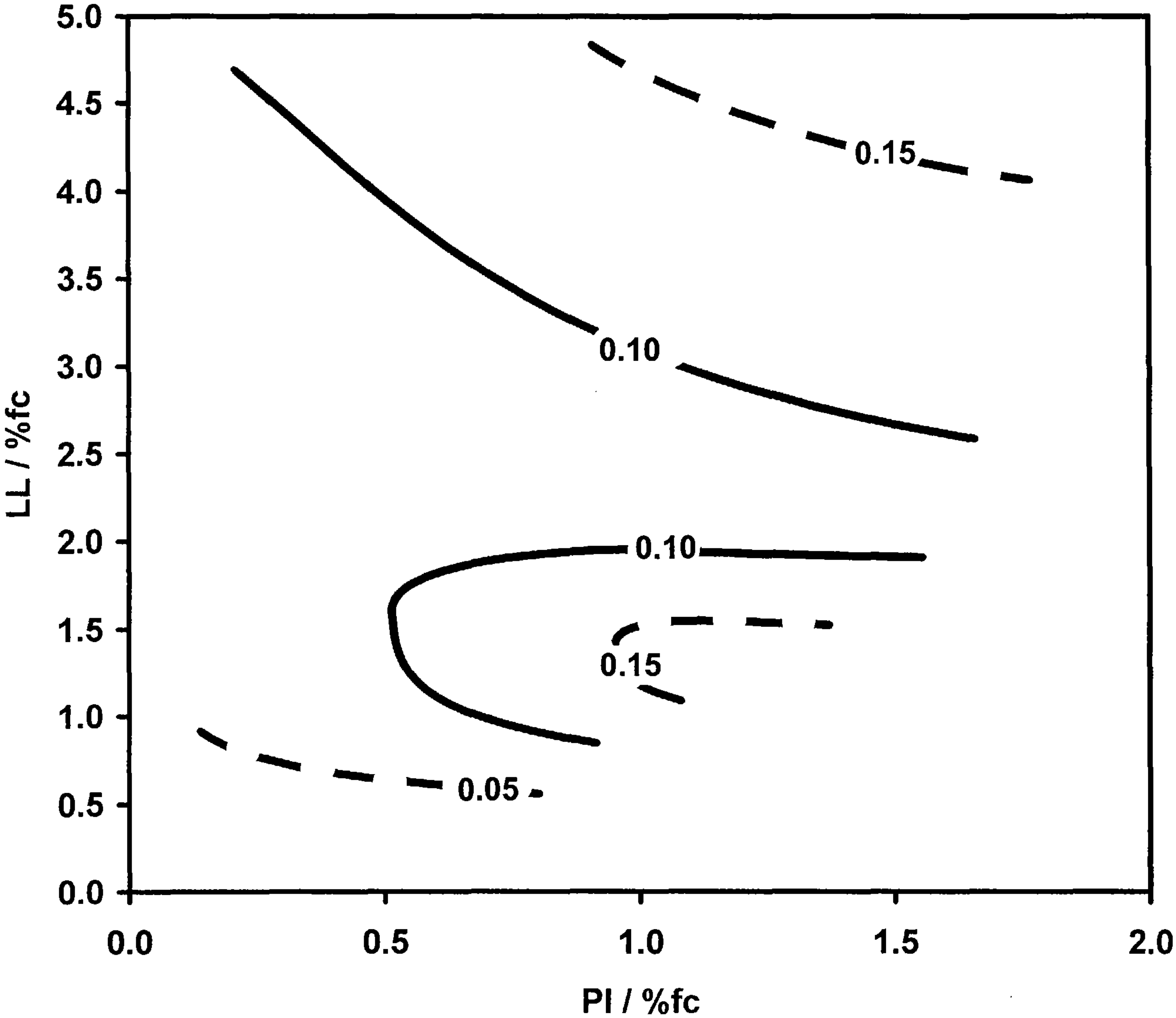


Figure 4. Predicted soil compression index values, Zone 4

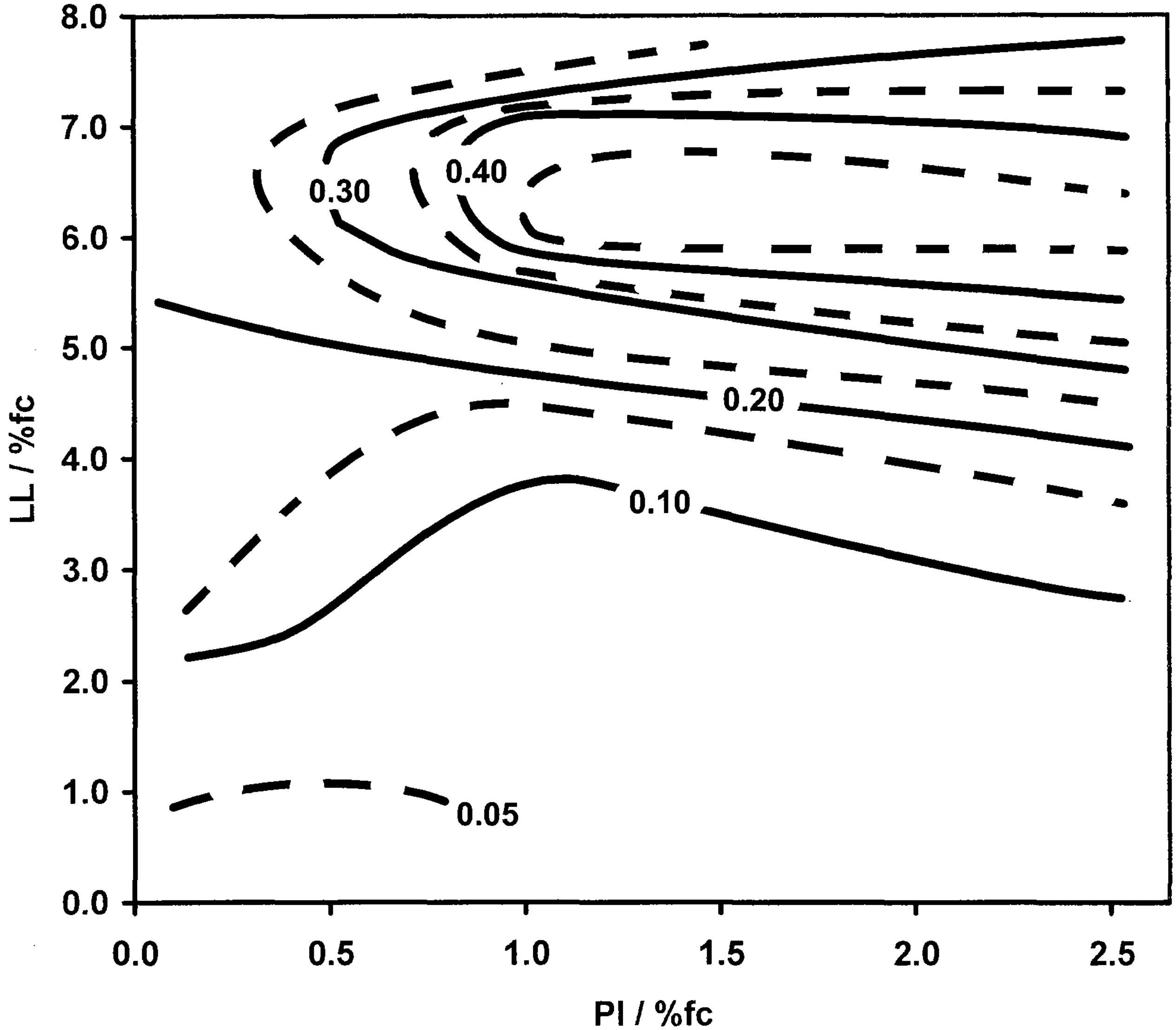


Figure 5. Predicted soil compression index values, Zone 5

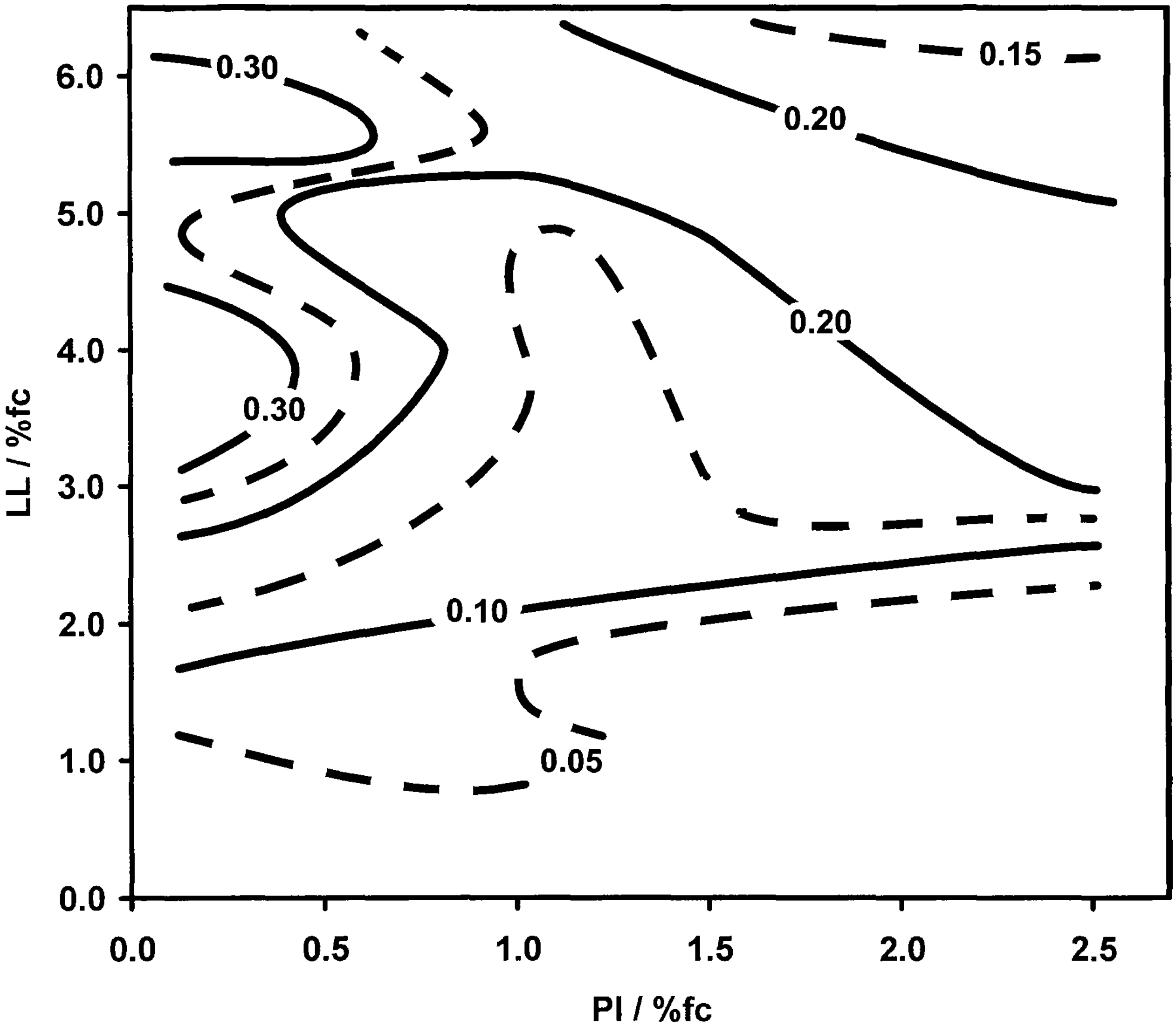


Figure 6. Predicted soil compression index values, Zone 6

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FOUNDATIONS AND PAVEMENTS ON UNSATURATED SOILS

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ABSTRACT: This paper defines briefly the functions of both foundations and pavements and states the measures of their performance that are affected by the properties of unsaturated soils. The paper then notes eight areas in which further development is needed to improve the analysis, design and performance of both foundations and pavements. Six examples of these developments are given in the areas of theory and constitutive equations. Several new concepts from thermodynamics, micromechanics, and other principles are illustrated as they apply to shear strength, volume change, lateral pressure, suction, and plasticity. In addition, future needed developments in testing methods and analysis methods used in design are described.

1.1. INTRODUCTION

This paper will present some reflections on what has been achieved in the engineering of foundations and pavements on unsaturated soils and suggest some of the directions that may be taken in the future.

Foundations are used for residential, commercial and industrial, public infrastructure, and high rise construction types of foundations include slab-on-ground, drilled pier and structural slab, retaining walls, canal linings, pipelines, landfill linings and caps, and earth structures such as dams and cut and fill slopes. Pavements are used for highways, roads, airports, and guideways. Types of pavements include Portland cement concrete, asphalt concrete, aggregate surfaced, and unpaved surfaces.

Each of these, foundations and pavements, are judged to have been designed and built successfully if they perform their intended function reliably and economically over their life cycle. Measures of performance differ between foundation types and pavement types. Regardless of the measure, a foundation or pavement must be designed taking into account the effect of the soil on which it rests. Table 1 indicates the measures of performance that are affected by unsaturated soils beneath the different types of foundation. These foundations require reasonably accurate predictions of the expected movements, pressures, and flows of the unsaturated soils to be made in order for the foundations to be designed successfully.

The same may be said of pavements on unsaturated soils. Table 2 shows the measures of the performance of pavements that are affected directly by their supporting unsaturated soils. Example of amplitude spectra are shown in Figures 1 through 4 (Velasco and Lytton, 1981). Figure 1 shows an amplitude versus frequency plot taken from a measured right wheel path profile. Figure 2 shows an amplitude versus frequency spectrum derived from a Fast Fourier Transform of the same measured profile. Figure 3 shows a collection of spectra from a number of pavements which range from rough to smooth. In this figure, the amplitude is plotted against the wave length, which is the reciprocal of the frequency. Figure 4 shows a typical probability density function of right and left wheel path wavelengths. These four figures are typical soil mass properties of expansive soils.

Table 1: Measures of Foundation Performance Affected by Unsaturated Soil.

FOUNDATION TYPE	MEASURES OF PERFORMANCE
Slab - on - Ground Drilled Pier	Differential Movement Total Movement · Heave, shrinkage · Collapse
Retaining Wall	Lateral Pressure · Movement · Creep
Canal Linings	Differential Movement
Pipeline	Differential Movement
Landfill Liners and Caps	Fracture Leakage of Leachate Moisture Balance
Earth Structures	Slope Failure Shallow Slope Failure Downhill Creep

Table 2: Measures of Pavement Performance Affected by Unsaturated Soils.

PAVEMENT USES	MEASURES OF PERFORMANCE
Airport	Amplitude Spectrum Acceleration Distress
Highway, Road, Guideway	Amplitude Spectrum International Roughness Index Bump Height Distress

The important types of unsaturated soils for foundations are those which are volumetrically active and those which are stress-responsive. The categories are not exclusive of one another. Volumetrically active soils include expansive soils, collapsing soils, frozen soils and cemented soils. Stress-responsive soils are both fine and coarse grained. Important types of unsaturated soils in pavements include the volumetrically active and load-responsive soils in the subgrade and base courses, and asphaltic concrete and Portland cement concrete in the surface courses. The latter two may be surprise additions to the list of unsaturated soils. However, asphalt concrete differs from unsaturated coarse grained soils only in the fluid which binds the particles together. Both fluids, asphalt and water, are normally in a state of tension in the unsaturated state. Portland cement concrete has particles cemented together but also has water in tension in its normal state.

All of this means that well-designed foundations and pavements require a knowledge of

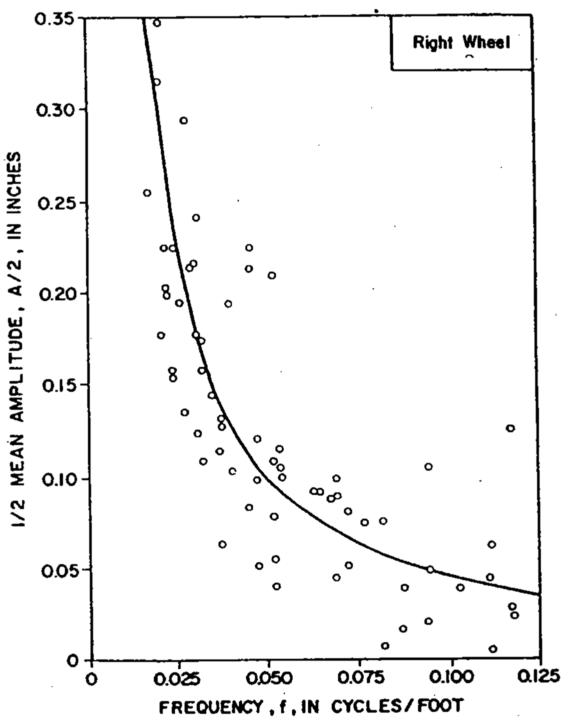


Figure 1: Amplitude versus Frequency Spectrum of a Pavement on Expansive Soil.

the properties of unsaturated soils. Soil properties come in two sizes: test sample size and soil mass size. Properties measured on a test sample are the mechanical properties of the soil. Properties of a soil mass include the variability of these properties and spectra of various characteristics of the soil mass such as crack spacing, wave length, roughness amplitude, and so on.

Mechanical properties of unsaturated soils include the stress-strain, plasticity, water and vapor conductivity, fracture, interface, and special properties. Among the stress-strain properties of unsaturated soils are

- volume response
- deviatoric response
- large and small strain properties
- resilient dilatancy and
- work potential.

Plasticity properties include

- limiting equilibrium
- tensile, compressive, and shear strength
- yield function, and
- plastic potential for non-associative permanent dilatancy.

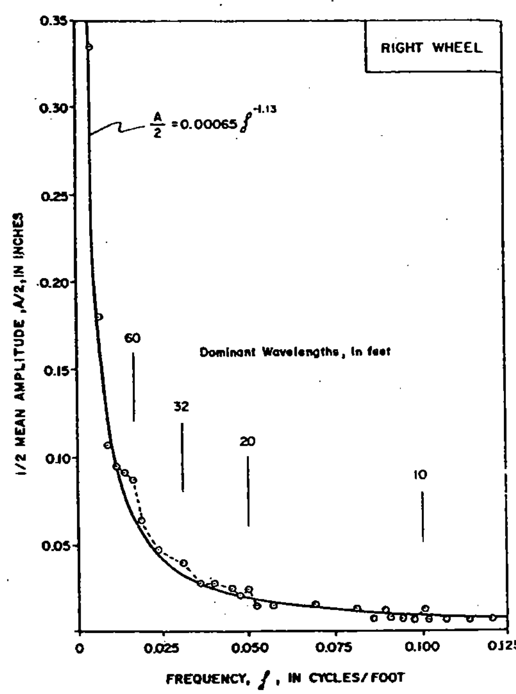


Figure 2: Fast Fourier Transform Amplitude versus Frequency Spectrum of a Pavement on Expansive Soil.

Water and vapor conductivity occurs on different scales. Fluids flow in soils in

- macrocracks (largely by gravity)
- microcracks (along suction gradients)
- intact soil.

The hydraulic conductivity gets progressively smaller as the flow passes from macrocracks to microcracks to the intact soil. Solutes in the fluid (usually water) can greatly increase the conductivity.

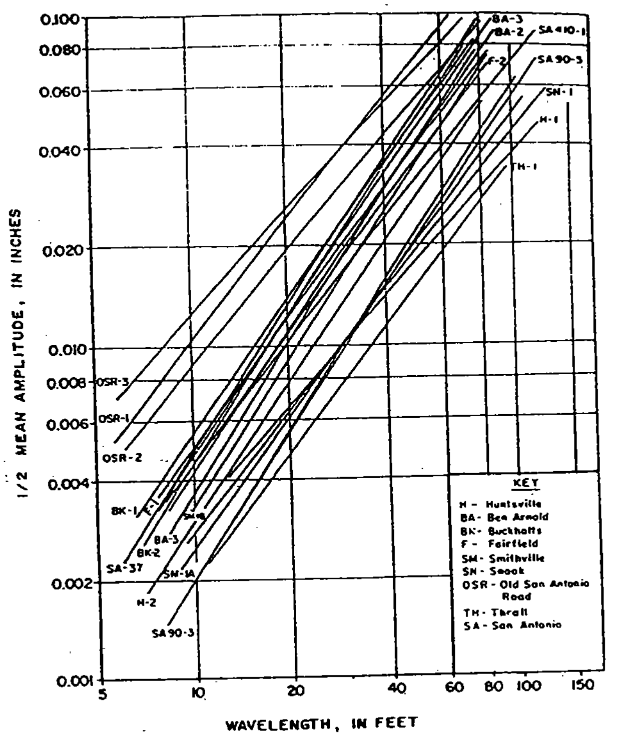


Figure 3: A Collection of Fast Fourier Transform Amplitude versus Wavelength Spectra of Pavements on Expansive Soils.

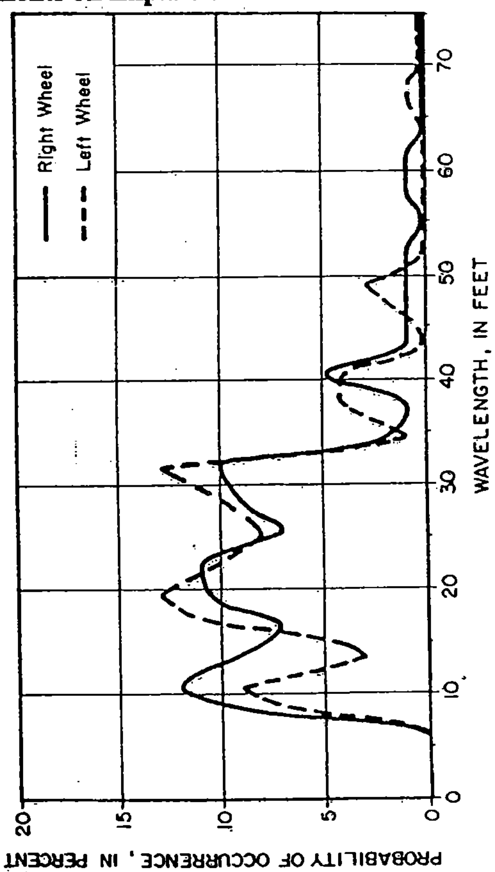


Figure 4: Probability Density of Right and Left Wheel Path Wavelengths of a Pavement on Expansive Soil

Fracture properties of unsaturated soils have not been used much in the past but will probably see much more use in the future. These include the tensile and shear compliances and the surface energies of the water and soil particle surfaces.

Interfaces between air and soil, soil and water, and soil and structure all have transmission and interaction properties. The air-soil interface has a coefficient of vapor transfer which according to Wilson, Barbour, and Fredlund (1995) is 1.0 for ranges of soil suction up to 1000 kPa (pF = 4) and reduces below that at higher suction levels. The soil-water interface is where erosion occurs and fluid shear transfer must be known. The soil-structure interface requires a knowledge of interface shear characteristics.

Special properties of unsaturated soils are the Poisson's ratio and lateral earth pressure coefficients. Poisson's ratio of expansive and collapsing soils are of interest for lateral earth pressure estimates. Poisson's ratios of granular stress-responsive soils well greater than 0.5 are common and explain the build-up of confining pressures in such soils under load. Lateral earth pressures at rest are different for volumetrically inert than for active soils such as swelling, shrinking, and collapsing soils. Adequate design of all retaining structures and drilled piers in active soils requires an accurate estimate of the range of lateral pressures both under shrinking and swelling conditions.

Other special properties include the interaction of an unsaturated soil with its environment, i.e., with the weather, water tables, vegetation, solutes and other osmotic effects, and cementation.

Examples of properties of unsaturated soil masses have been given above, including variability and various spectra. The hydraulic conductivity of a soil mass, both in its natural state and as it is compacted, depends upon the relative distribution the sizes of connected voids in the soils mass in the macrocrack, microcrack, and intact soil size ranges. The opening of these cracks has a directly bearing on lateral earth pressure as well. As a general rule, the macrocracks can be considered to be closed when the soil suction around the crack is lower than the suction level at depth. The suction level at depth is controlled either by a high water table or the climatic moisture balance between rainfall and evapotranspiration.

This is a summary of where we are now in using the mechanics of unsaturated soils in the analysis and design of foundations and pavements. There are obvious needs for future developments in eight areas.

1. Further development of theory: the continuum theory of mixtures and in micromechanics.
2. Development of constitutive equations for all mechanical materials properties of unsaturated soils.
3. Development of test methods for determining these material properties both at research and production testing levels.
4. Computational methods for analyzing foundations and pavements need to be developed in the areas of coupled flow of water, vapor, and heat, elasticity, plasticity, fracture, and interactions at interfaces.
5. Analysis methods for use in design need to be developed, and a particular need is to estimate envelope values of design quantities.
6. Design methods for foundation elements and pavements need to be developed incorporating the properties of the supporting unsaturated soils and using the prediction of performance measures as a basis for design.
7. Use of a reliability approach in the design of foundations and pavements taking into account the variability of material properties, geometry, and loading, and using a rationally selected level of reliability.
8. Comprehensive use of nondestructive testing methods in site investigation, construction quality assurance and quality control, and field performance monitoring.

With the rapid improvements in computers and instrumentation that are currently under way, the greatest practical barriers are being overcome to the realization of these developments in the near future.

There are no specifics in the list of eight needs for future development. Some specific examples of these needed developments are presented here in the areas of theory, constitutive equations, testing methods, and analysis methods used in design.

1.2. Example Development No. 1. Theory

The stress that is generated on the unsaturated soil mineral skeleton due to tension in the pore water has been determined by use of reversible thermodynamics principles by Lamborn (1986):

$$\overline{\sigma_{ij}} = \theta \frac{\partial F_w}{\partial \left(\overline{\epsilon_{ij}}\right)_w} \tag{1}$$

where

F_w = the Helmholtz free energy in the water

$(\overline{\epsilon}_{ij})_w$ = the strain in the water

θ = volumetric water content

$\overline{\sigma}_{ij}$ = stress on the soil mineral skeleton due to the water.

The formulation was made for "moist" soil that is substantially drier than the saturated condition. This is the soil moisture condition in which the air in the soil is continuous open channels. In terms that are somewhat more familiar,

$$\overline{\sigma_w} = -\theta\ h_m \tag{2}$$

where $\bar{\sigma}_w$ = the stress on the soil mineral skeleton due to the water

θ = the volumetric water content

h_m = the matric suction, a negative number, corresponding to tensile stress in the pore water. The symbol u_w is also used to denote this matric suction.

The first fact to note is that matric suction is a derivative of the Helmholtz free energy of the water with respect to the strain in the water. The second notable fact is that this formulation has applications in estimating the shear strength of the soil. The theoretical relation between shear strength, mechanical stress, matric suction, and their respective friction angles is given by Fredlund, et. al. (1978) as

$$s = c' + (\sigma - u_a) tan\phi' + (u_a - u_w) tan\phi^b \tag{3}$$

When air pressure is different from atmospheric, the term $(u_a - u_w)$ represents the combined effect of the air pressure and the matric suction applied to the soil mineral skeleton.

Using Lamborn's formulation for moist soils this becomes

$$s = c' + [(\sigma - u_a) + \theta\ (u_a - u_w)]\ tan\ \phi' \tag{4}$$

since

$$tan\ \phi^b = \theta\ tan\ \phi' \tag{5}$$

Empirical confirmation of this relation is found in the data of Lam (1980) and Peterson (1992). Lam's measurements were made on decomposed rhyolite and Peterson's measurements were made on the Vicksburg Buckshot clay. A graph of the measured strength due to suction versus the product of volumetric water content, θ, and matric suction, h_m, in Figure 5 shows that if anything, the product $|h_m\ \theta|$ overestimates the strength of the soil.

As the soil becomes saturated and air exists in the soil only in the form of occluded

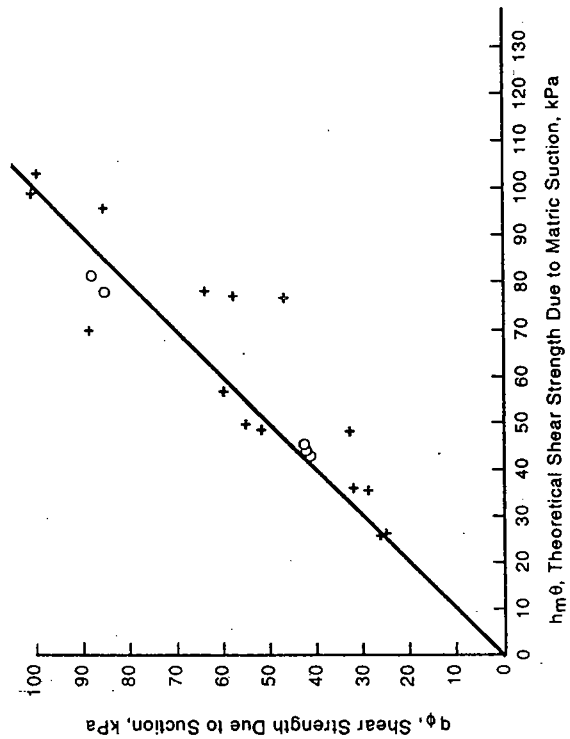


Figure 5: Measured versus Predicted Shear Strength of Soil Due to Matric Suction (Lam, 1980).

bubbles, the tangent of the friction angle due to mechanical stress is the same as the friction angle due to matric suction.

$$tan\phi^b = 1.0\, tan\phi' \tag{6}$$

The change of multiplying factor of these friction angles from 1.0 for wet soils with occluded bubbles to θ for moist soils with continuous open air channels undergoes a transition as illustrated schematically in Figure 6.

The transition zone occurs between the air entry point suction value and the unsaturation point suction value. The air entry point is where open air channels begin to appear in the soil. These channels begin to open with the larger pore spaces and then, as the suction level increases, the open air channels extend into the smaller pore spaces. A measure of the volume of pore spaces that is evacuated between the air entry suction and the unsaturated suction levels is given in Equation (7).

$$\theta_a - \theta_u = \int_{\theta_u}^{\theta_a} d\theta = \int_{h_{mu}}^{h_{ma}} \left(\frac{\partial \theta}{\partial h_m}\right) dh_m \qquad (7)$$

where

θ_a , θ_u = the volumetric water content of the soil at air entry and unsaturation,

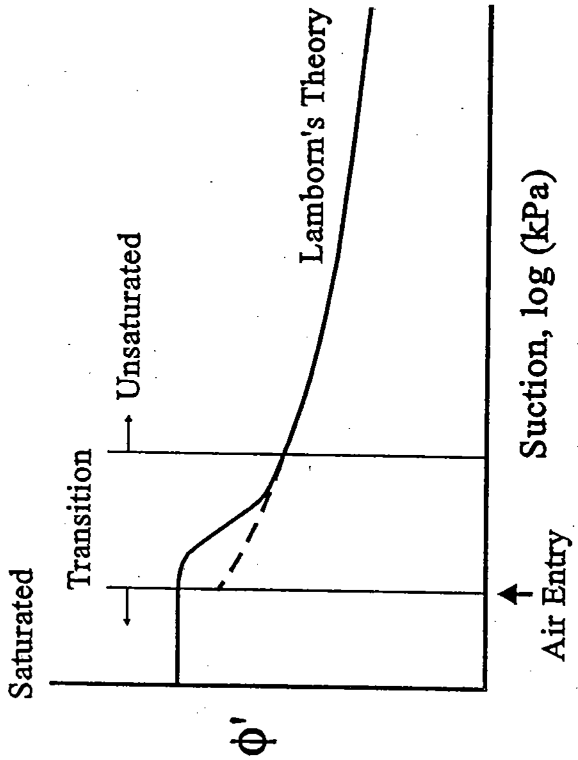


Figure 6: Transition of Friction Angle due to Matric Suction from the Saturated to Unsaturated State.

respectively

h_{ma} , h_{mu} = the matric suction values at air entry and unsaturation, respectively

$\left(\frac{\partial \theta}{\partial h_m}\right)$ = the slope of the matric suction-volumetric water content characteristic curve for the soil

According to micromechanics theory (Modern Composite Materials, 1967), the upper and lower bounds of the stress on the soil mineral skeleton due to water at a suction level, h_m, is given by the expressions

$$\overline{\sigma_w} = -h_m \theta \left[\left(\frac{\theta_a - \theta}{\theta_a - \theta_u}\right) + \frac{1}{\theta}\left(\frac{\theta - \theta_u}{\theta_a - \theta_u}\right)\right] \qquad (8)$$

and

$$\overline{\sigma_w} = -h_m \theta \left[\frac{1}{\left(\frac{\theta_a - \theta}{\theta_a - \theta_u}\right) + \theta \left(\frac{\theta - \theta_u}{\theta_w - \theta_u}\right)} \right] \qquad (9)$$

The three differences in volumetric water content in Equations (8) and (9) may be found using the slope of the suction - vs. - volumetric water content characteristic curve:

$$\left(\theta_a - \theta_u\right) = \int_{h_{mu}}^{h_{ma}} \left(\frac{d\theta}{dh_m}\right) dh_m \qquad (10)$$

as in Equation (7), and the other two differences are

$$\left(\theta_a - \theta\right) = \int_{h_m}^{h_{ma}} \left(\frac{d\theta}{dh_m}\right) dh_m \qquad (11)$$

$$\left(\theta - \theta_u\right) = \int_{h_{mu}}^{h_m} \left(\frac{d\theta}{dh_m}\right) dh_m \qquad (12)$$

The expression for the suction-related friction angle, $\tan \phi^b$, is bounded by the product of $\theta \tan\phi'$ and the two functions in brackets in Equation (8) and (9). There are several observations that may be made of this result:

1. Lamborn's theory constitutes a lower bound solution for the entire range of water content.
2. In the transition zone, the stress on the mineral skeleton due to the water is equal to the matric suction multiplied by a

number between θ and 1, and more specifically in the transition zone, is bounded by $|h_m\theta|$ multiplied by the two functions in brackets in Equations (8) and (9).

3. The suction-related friction angle $\tan\phi^{b'}$ is equal to $\tan\phi'$ multiplied by a number between θ and 1, and more specifically in the transition zone, is bounded by $\theta\tan\phi'$ multiplied by the two functions in brackets in Equations (8) and (9).
4. The functions in brackets will be the same at a given volumetric water content regardless of whether the soil is wetting or drying. The value of matric suction, h_m, corresponding to that volumetric water content will be smaller during wetting than during drying and because of this, the soil is weaker on wetting at the same water content.
5. In estimating the strength of soil in the field, accurate measurement of the volumetric water content, θ, either with nuclear moisture or ground penetrating radar equipment will lead to a lower bound estimate using Lamborn's theory and, in the transition zone, to an accurately bounded estimate using the bracketed functions in Equations (8) and (9).

More recent developments in micromechanics such as the "method of cells" (Aboudi, 1991) are capable of providing the exact relation between $\tan\phi^b$ and $\tan\phi'$, instead of upper and lower bounds.

This is an example of how the development of theory in unsaturated soil can provide practical benefits to the areas of foundations and pavements.

1.3. Example Development No. 2. Constitutive Equations

This example development makes use of the previous one and adds another in developing the constitutive equation for both the resilient modulus and the Poisson's Ratio of an unsaturated soil. The original development is based upon empirical observations of a dry granular soil (Uzan, 1985) that the resilient modulus is given by the power law form:

$$E = k_1 p_a \left(\frac{I_1}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{13}$$

where

I_1 = the sum of all principal mechanical stresses

τ_{oct} = the octahedral shear stress

p_a = atmospheric pressure in the same units as the resilient modulus

k_1, k_2, k_3 = material properties of the dry granular soil.

When water is added to a soil to make it an unsaturated soil, the effect of suction is added to the above formulation to give:

$$E = k_1 p_a \left(\frac{I_1 - 3\theta f h_m}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{14}$$

where

$\theta\, h_m$ = the lower bound term from Lamborn's theory

f = the function of volumetric water content presented in the previous example development

The value of f is 1 at all water contents greater than θ_a; is equal to θ at all water contents less than θ_u; and is bounded by the bracketed terms in Equations (8) and (9) in the transition zone between saturated and unsaturated behavior. The volume change of this soil is governed by an elastic work potential when it is loaded and unloaded. For the properties k_1, k_2, and k_3 to be stress path independent, the following integral must be equal to zero when the integral is taken around a closed stress path (Lade and Nelson, 1987).

$$\int \left(\frac{I_1 dI_1}{9K} + \frac{dJ_2}{2G}\right) = 0 \tag{15}$$

where

K = the bulk modulus

G = the shear modulus

I_1 = The first invariant of the stress tensor which is equal to the sum of the principal stresses

J_2 = the second invariant of the deviatoric stress tensor which is related to the octahedral shear stress

Expressing the bulk modulus, K, and shear modulus, G, in terms of the Young's modulus, E, and Poisson's ratio, ν, and substituting these into the elastic work potential equation produces the following differential equation:

$$-\frac{1+\nu}{I_1}\frac{\partial(\ell nE)}{\partial I_1}+\frac{1}{I_1}\left(\frac{\partial \nu}{\partial I_1}\right)+\frac{1-2\nu}{3}\frac{\partial(\ell nE)}{\partial J_2}+\frac{2}{3}\frac{\partial \nu}{\partial J_2}=0 \tag{16}$$

Taking the natural logarithm of Equation (13) and substituting the result into Equation (16) produces the following partial differential equation for the Poisson's ratio (Lytton, Uzan, et. al., 1993):

$$\frac{2}{3}\left(\frac{\partial \nu}{\partial J_2}\right)+\frac{1}{I_1}=\nu\left[\frac{2}{3}\frac{k_3}{2J_2}+\frac{k_2}{I_1^2}\right]+\left[-\frac{1}{3}\frac{k_3}{2J_2}+\frac{k_2}{I_1^2}\right] \tag{17}$$

when the soil is unsaturated and suction is present in the soil, the term I_1 in Equation (17) must be replaced by the term, I_{1u}, as in Equation (18):

$$I_{1u} = I_1 - 3\theta f h_m \tag{18}$$

The solution of the partial differential equation in Equation (17) for the Poisson's ratio is given by Zachmanoglou and Thoe (1976) as

$$= \alpha k_4 (u_1)^{k_5} + \frac{\alpha 3^{\delta}}{2(u_1)^{\beta}}\left[-k_2 B_v(\beta,-\delta) + k_3 B_v(\beta,-\delta\right. \tag{19}$$

where

$u_1 = I^2_{1u} - 3J_2$

$\alpha = I_{1u}^{k_2} J_2^{½k_3}$

$$\beta = \frac{k_2 + k_3}{2}$$

$$\delta = \frac{k_3}{2}$$

Bv (,) = the incomplete Beta function

k_4, k_5 = two additional constants that are required to meet the loading and unloading initial and final conditions.

An example of how well this predicted value of the Poisson's ratio fits the observed data is shown in Figure 7

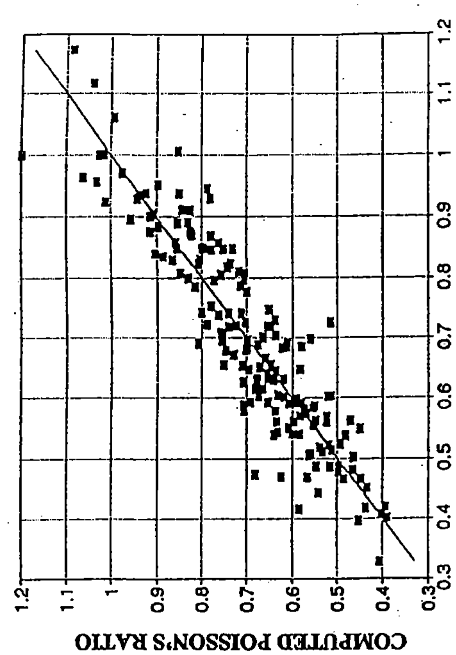


Figure 7: Measured versus Predicted Poisson's Ratio for Granular Materials.

Typical values of k_1, k_2, and k_3 for a variety of base course and subgrade materials commonly used in Texas are given in Tables 3, 4, and 5(Titus - Glover, 1995). The values of k_1, k_2, and k_3 differ between the dry of optimum, optimum, and wet of optimum cases because the composition of the soil (density and water content) are different in each case. In principle, there is no need to determine values of k_4 and

k_5 if the partial differential equation in Equation (17) is solved incrementally by numerical methods.

As seen in Figure 7, the value of Poisson's ratio rises well above 0.5, indicating an increase in volume under certain stress conditions such as high shearing stresses combined with low sums of principal stresses. When the same unsaturated soil is confined, it builds up large additional confining pressures and becomes much stiffer under load. It is this ability which explains how base courses never experience tensile stresses while carrying heavy loads. This "resilient dilatancy" of the base course material is what allows that material to develop its own added strength and stiffness to resist the effects of the load.

Table 3: Resilient Modulus (Dry of Optimum)

MATERIAL	k_1	k_2	k_3
Limestone	1500	0.90	-0.33
Iron Ore Gravel	2820	0.60	0.00
Sandy Gravel	11,300	0.63	-0.10
Caliche	1,440	1.18	0.00
Shell	830	1.10	0.00
Sand	3,120	0.44	0.00
Silt	820	1.20	-0.11
CL Clay	4,100	0.00	-0.27
CH Clay	200	0.66	-1.47

Table 4: Resilient Modulus (Optimum)

MATERIAL	k_1	k_2	k_3
Limestone	1,660	0.90	-0.33
Iron Ore Gravel	1,270	0.49	0.00
Sandy Gravel	1,570	0.67	-0.28
Caliche	890	0.83	-0.01
Shell	820	0.60	0.00
Sand	6,430	0.51	0.00
Silt	1,170	0.52	-0.20
CL Clay	110	0.32	-0.10
CH Clay	260	1.25	-0.50

Table 5: Resilient Modulus (Wet of Optimum)

MATERIAL	k_1	k_2	k_3
Limestone	3,850	0.43	-0.02
Iron Ore Gravel	210	0.56	0.00
Caliche	480	0.19	0.00
Shell	750	0.78	0.00
Sand	6,320	0.40	-0.03
Silt	1,000	0.50	-0.10
CL Clay	780	0.10	-0.55
CH Clay	440	0.66	-0.17

It is the same "resilient dilatancy" that explains most of the resistance of asphalt concrete at high temperatures to plastic deformation or rutting. This is an unusual example of an unsaturated soil in which the fluid is asphalt and not water, but the principle is the same. At high temperatures above 45°C, the asphalt exerts little restraint on the aggregates of the mix. This means that unless the aggregates are well graded enough to develop sufficient "resilient dilatancy" they will not successfully resist permanent lateral shearing displacement. This is why gradation is so important in asphalt mix design, and for that matter, in the specifications for base course aggregates. It is also why determining the coefficients k_1, k_2, and k_3 and the function, f, are a key to the sound practical use of unsaturated soils.

1.4. Example Development No. 3 Constitutive Equations

This third example is an illustration of how a constitutive equation for the volume change of expansive soil (or any soil which undergoes large strains or displacements may be constructed using the methods of Juarez-Badillo, whose work deserves much more attention that it has received in the past(Juarez-Badillo, 1983, 1985, 1986, 1987+).

Juarez-Badillo first determines the natural limits of any process (mean principal stress, suction, and volume, in this case). This is illustrated in Figure 8a. Under conditions of zero mechanical pressure and suction, the soil reaches its maximum volume, V_o. Under conditions of zero suction and infinite mechanical mean principal stress, the soil volume compresses to the volume of the solids alone, V_s. Under conditions of zero mean principal stress and infinite suction, the soil volume compresses to the dry volume, V_d, in which the dry

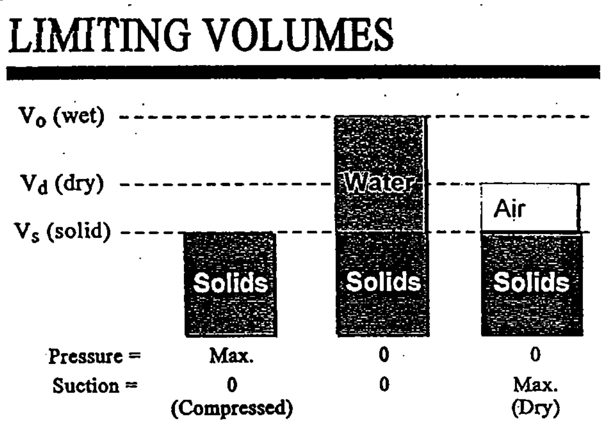


Figure 8a: Natural Limits of the Volume Change Process in Unsaturated Soils.

soil contains a volume of air-filled voids. Plotting volume, mean principal stress and suction along independent axes shows how this surface appears over the full range of the three variables. This is shown in Figure 8b. The method of Juarez-Badillo now operates upon this information.

At a zero mean principal stress level, the range of suction is between 0 and ∞, and the corresponding range of volume is between V_o and V_d. The Juarez-Badillo method establishes a function of volume that has the same limits as suction. The function is found to be

$$f(V) = \frac{1}{V - V_d} - \frac{1}{V_o - V_d} \tag{20}$$

The method now states that the rates of change of the two processes, change of suction and change of f (V), which have the same limits ($|h| = 0$, $f(V_o) = 0$; $|h| = \infty$, $f(V) = \infty$) must be

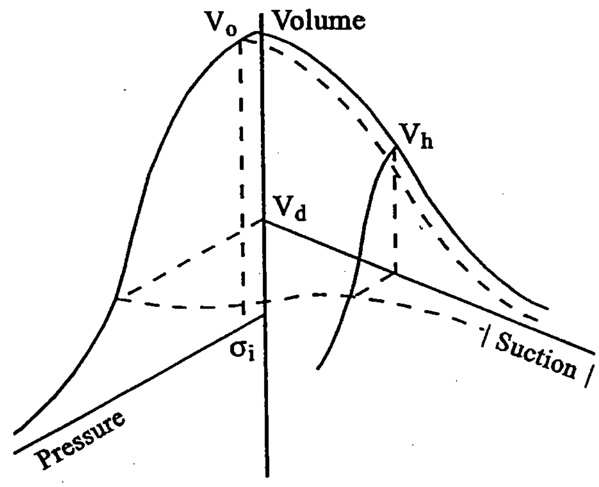


Figure 8b: The Volume - Mean Principal Stress - Suction Surface.

proportional to one another. The constant of proportionality is γ_h.

$$-\gamma_h \frac{d|h|}{|h|} = \frac{df(V)}{f(V)} \tag{21}$$

The use of the symbol γ for this gas law constant is consistent with the use of the same symbol by Juarez-Badillo in all of his original work on large strains in consolidating clays (e.g. Juarez-Badillo, 1983, 1985). Integrating the two expressions between the limits of (V_h, $|h|$) and (V_1, $|h_1|$) which are two generic points on the curve, leads to the expression for the volume of the soil at zero mean principal stress and varying suction levels:

$$V_h = \frac{V_o + a\, V_d\, |h|^{\gamma_h}}{1 + a\, |h|^{\gamma_h}} \tag{22}$$

A similar process establishes the volume of the soil at a constant suction level. The volume changes between the limits V_h (from Equation 22) and V_s at large stress levels. The equation is

$$V = \frac{V_h + b\, V_s\, (\sigma - \sigma_i)^{\gamma_\sigma}}{1 + b(\sigma - \sigma_i)^{\gamma_\sigma}} \tag{23}$$

where

σ = the level of mean principal stress, corresponding to the volume, V.

σ_i = the level of mean principal stress above which the soil volume begins to decrease

a, b = constants to be determined from the measured volume - suction - mean principal stress surface

γ_h, γ_σ = gas law constants for volume change due to a change of suction and a change of mean principal stress, respectively.

This formulation gives the large strain relation between volume, suction, and mean principal stress. An approximate relation which applies to smaller areas on this surface comes from integrating the following differential equation

$$\frac{dV}{V} = -\gamma_h \frac{dh_m}{h_m} - \gamma_\sigma \frac{d\sigma}{\sigma} - \gamma_\pi \frac{d\pi}{\pi} \quad (24)$$

This produces the equation

$$\frac{V_f}{V_i} = \left(\frac{h_{mf}}{h_{mi}}\right)^{-\gamma_h} \left(\frac{\sigma_f}{\sigma_i}\right)^{-\gamma_\sigma} \left(\frac{\pi_f}{\pi_i}\right)^{-\gamma_\pi} \quad (25)$$

where

V_f, V_i = the final and initial volumes

h_{mf}, h_{mi} = the final and initial matric suction values

σ_f, σ_i = the final mean principal stress and the initial mean principal stress below which no volume change takes place

π_f, π_i = the final and initial osmotic suction values

$\gamma_h, \gamma_\sigma, \gamma_\pi$ = the gas law constants for volume change due to changes in matric suction, mean principal stress, and osmotic suction. The γ_h and γ_σ constants are the same as in the large strain relation, Equation (23).

A related approximate small strain formulation is given by taking the logarithm of both sides of Equation (25) to obtain the following:

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left(\frac{h_{mf}}{h_{mi}}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_\pi \log_{10}\left(\frac{\pi_f}{\pi_i}\right) \quad (26)$$

This latter is applicable to volume changes in which small strains occur. Thus, Equations (23), (25), and (26) are related expressions of volume change in expansive soils, applicable to large, intermediate, and small strain conditions. All of these use the same gas law constants, γ, as a consistent material property. A familiar engineering relation is found from the small strain formula applied to the pre-consolidated consolidation curve.

$$\gamma_\sigma \cong \frac{C_s}{1 + e_o} \quad (27)$$

where

C_s = the pre-consolidated swelling or compression index

e_o = the initial void ratio

Juarez-Badillo's has applied his method successfully to the large strain consolidation of the Mexico City clays (Juarez-Badillo, 1986, 1987+). In the process, he found that it was unnecessary to separate the consolidation process into primary and secondary consolidation and tertiary creep. Instead, he found that the entire compression curve represents a single process represented by the γ - constant. The parsing of consolidation into separate processes is, in fact, an artifact of the small strain assumption. Thus, it is seen in this case that obtaining a more comprehensive relation actually simplifies the task of characterizing the materials properties of the soil.

1.5. Example Development No. 4 Constitutive Equations

The analysis and design of retaining structures, basement walls, and other laterally loaded elements requires an estimate of lateral earth pressure, which in turn, requires an estimate of the Poisson's ratio. The lateral earth pressure coefficient for static, elastic, and small strain conditions is given by Fredlund and Rahardjo (1993) as

$$k_o = \frac{v}{1 - v} + \frac{1}{1 - v}\left(\frac{E}{H}\right)\left(\frac{h_m}{\sigma_v}\right) \tag{28}$$

where

k_o = the lateral earth pressure coefficient

E = the Young's modulus of the unsaturated soil due to a change of mechanical stress

H = the Young's modulus of the unsaturated soil due to a change of suction

h_m = the matric suction which remains unchanged (a negative value)

σ_v = the static vertical mechanical stress (a positive value)

The lateral earth pressure when the suction changes from an initial to a final condition and the material properties of the soil are sensitive to changes in mechanical stress and suction would be expected to require some interaction between the initial and final states of stress in the soil. Assuming the intermediate volumetric strain formulation as in Equation (25), and also assuming that deviatoric strains follow a hyperbolic stress-strain rule, the Poisson's ratio is

$$v = \frac{\dfrac{m}{\gamma_\sigma} + a}{2\dfrac{m}{\gamma_\sigma} - a} \tag{29}$$

where $$m = \frac{K_{of}\,\sigma_v}{G} \tag{30}$$

K_{of} = the final lateral earth pressure coefficient

G = the shear modulus of the unsaturated soil

and

$$a = \frac{K_{of}}{1 - K_{of}}\left[1 - \frac{1 - K_{of}}{2K_{of}\tau_{uf}}\right]\ell n\left[\frac{1 + 2K_{of}}{3K_{of}}\right] \tag{31}$$

τ_{uf} = the asymptote value of shear stress which is approached by the hyperbolic shear stress - shear strain curve. The value of τ_{uf} is estimated by

$$\tau_{uf} = \left[\sigma_v\left(1 + 2K_{of}\right) - h_{mf}\theta_f f\right]\tan\phi' \tag{32}$$

where

h_{mf} = the final matric suction value (a negative value)

θ_f = the final volumetric water content

f = the shear strength function which is bounded by the bracketed terms in Equation (8) and (9)

All of the equations given above for the Poisson's ratio involve a knowledge of the final value of the lateral earth pressure coefficient. This shows that both the Poisson's ratio and the lateral earth pressure coefficient must be found by a converging iterative process. This is seen in the following expression for the final lateral earth pressure coefficient.

$$K_{of} = \left(\frac{1 + 2K_{oi}}{2}\right)\left\{\frac{3\left(\dfrac{m}{\gamma_\sigma}\right) + 6\left(\dfrac{E}{H}\right)\left(\dfrac{h_{mi}}{\sigma_v}\right)\left[2\left(\dfrac{m}{\gamma_\sigma}\right) - a\right]}{\left(\dfrac{m}{\gamma_\sigma}\right) - 2a}\right\}\left(\frac{h_{mi}}{h_{mf}}\right)^r - 0.5 \tag{33}$$

where

r = the ratio of $\dfrac{\gamma_h}{\gamma_\sigma}$

h_{mi} = the initial value of matric suction (a negative value)

K_{oi} = the initial value of lateral earth pressure coefficient which may be estimated with the Fredlund and Rahardjo formula in Equation (28).

It should be noted at this point that if the soil creeps under pressure, and all do, the ratio (E/H) will be the ratio of the long-term relaxation moduli of the unsaturated soil. This ratio will very likely be unlike the ratio of the relaxation moduli at short loading times.

The lessons to be learned from Equations (29) through (33) are that the Poisson's ratio of unsaturated soil is stress - and - suction - sensitive; that it depends upon the gas law constant of volume change, γ_o; and that it depends upon the initial and final values of the lateral earth coefficient, K_{oi} and K_{of}, and upon the initial and final values of matric suction. All of this means that the lateral earth pressure of an active soil against a retaining structure can be found only by a convergent iterative procedure that correctly represents the interaction between the soil and the retaining structure as the soil attempts to expand under conditions of changing suction and confining pressures. The shear modulus which was used in Equation (30) to define the function, m, is also stress - and - suction - sensitive. In terms of the power law constitutive equation in Equation (14), the shear modulus is

$$G = \frac{k_1 p_a \left[\frac{I_1 - 3\theta f h_m}{p_a}\right]^{k_2} \left[\frac{\tau_{oct}}{p_a}\right]^{k_3} \left[2\frac{m}{\gamma_\sigma} - a\right]}{6\left(\frac{m}{\gamma_\sigma}\right)} \qquad (34)$$

Equation (34) shows that the value of the shear modulus must also be found by a convergent iterative process. This implies the necessary use of non-linear numerical methods in analyzing and in determining the design values for retaining structures in volumetrically active soils.

1.6. Example Development No. 5 Constitutive Equation

It is important at times to stand back from one's work and look at it from a different perspective. The view may provide insights that invite further progress. Such is the case with surface energies by which water, vapor, and soil particle surfaces are attached to one another. The subject of surface energies is being researched intensively by surface chemists studying adhesive and cohesive bonding. An excellent summary of current thinking in this subject has been published by Good and Van Oss (1991). The laboratory equipment that is used for the measurements is simple but very precise and is called the Wilhelmy Plate apparatus. The equation that is used to interpret the data is 190 years old, having been presented by Thomas Young in 1805. It is known as Young's Equation and is illustrated in Figure 9.

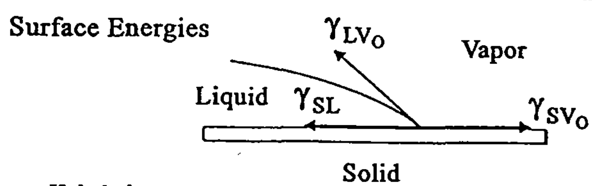
$$\gamma_{SV_o} - \gamma_{SL} = \gamma_{LV_o} \cos\theta \qquad (35)$$

where

- γ_{SV_o} = the surface energy between the solid surface and saturated vapor
- γ_{SL} = the surface energy between the solid and the liquid
- γ_{LV_o} = the surface energy between the liquid and the saturated vapor
- θ = the contact angle

In a 1971 paper, Zisman (1971) noted that

EXAMPLE: CONSTITUTIVE EQUATIONS



Helmholtz Free Energy

$$\gamma_{LV_O} = (\partial F / \partial A_{LV_O})\, T, \mu_i$$
$$\gamma_{SL} = (\partial F / \partial A_{SL})\, T, \mu_i$$
$$\gamma_{SV_O} = (\partial F / \partial A_{SV_O})\, T, \mu_i$$

Figure 9: Vector Diagram Illustrating Young's Equation

these surface energies are in fact derived from the Helmholtz free energy of the solid-liquid-vapor system in thermodynamic equilibrium as follows.

$$\gamma_{SL} = \left(\frac{\partial F}{\partial A_{SL}} \right)_{T, M_i} \tag{36}$$

$$\gamma_{SV_o} = \left(\frac{\partial F}{\partial A_{SV_o}} \right)_{T, M_i} \tag{37}$$

$$\gamma_{LV_o} = \left(\frac{\partial F}{\partial A_{LV_o}} \right)_{T, M_i} \tag{38}$$

where

F	=	the Helmholtz free energy of the system
A_{SL}	=	the area of the solid-liquid interface
A_{SV_o}	=	the area of the solid-saturated vapor interface
A_{LV_o}	=	the area of the liquid-saturated vapor interface

All of the partial derivatives are taken holding temperature and all chemical (osmotic) potentials constant. The surface energies are further broken down into components (Good and Van Oss, 1991).

$$\gamma = \gamma^{LW} + \gamma^{AB} \tag{39}$$

where

γ	=	the surface energy of a liquid or a solid surface
γ^{LW}	=	the apolar surface energy due to Lifshitz-van der Waals forces
γ^{AB}	=	the polar surface energy which is made up of Lewis acid-base interactions

$$\gamma^{AB} = 2\left(\gamma^{\oplus}\gamma^{\ominus}\right)^{1/2} \tag{40}$$

where

$\gamma^{\oplus}$	=	the Lewis acid component
$\gamma^{\ominus}$	=	the Lewis base component

Typical values of the components of surface energies are given in Table 6, for several sizes of river sand, limestone fines, and water (Elfingstone and Li, 1994-95). These values are tabulated here to call attention to the fact that there is a layer of scientific understanding of the attachment of water to soil surfaces that is one level more fundamental than the one used in unsaturated soil engineering. The free energy of adhesion of water to a solid surface is given by

$$\Delta G_{s\ell} = -2\sqrt{\gamma_S^{LW}\gamma_\ell^{LW}} - 2\sqrt{\gamma_S^{\oplus}\gamma_\ell^{\ominus}} - 2\sqrt{\gamma_S^{\ominus}\gamma_\ell^{\oplus}} \tag{41}$$

where

γ_S	=	the surface energy of the solid surface
γ_ℓ	=	the surface energy of the liquid surface.

Table 6: Measured Surface Energies of Soil Particles and Water.

PARTICLE GEOMETRY		SURFACE ENERGIES, mJ/m²					
AGGREGATE	SIZE, µm	SSA* m²/g	Γ	Γ^{LW}	Γ^{AB}	$\Gamma^{\oplus}$	$\Gamma^{\ominus}$
River Sand	500-	0.257	169.6	64.8	104.8	11.0	250.8
River Sand	1000	0.639	199.1	64.6	134.5	21.5	210.0
River Sand	125-250	6.168	223.1	80.3	143.0	35.9	142.1
Limestone	<63	7.031	134.7	68.3	66.4	2.9	378.1
Filler	<63						
Water	-		72.8	21.8	51.0	25.5	25.5

What we call matric suction is the product of $\Delta G_{s\ell}$ and the specific surface area of the particle to which the fluid is bonded (Marquis, et. al., 1982). It is instructive to use Equation (41) together with the values of γ^{LW}, $\gamma^{\oplus}$, and $\gamma^{\ominus}$ for the solids and the liquid in Table 6 to see how matric suction changes with the particle size.

1.7. Example Development No. 6 Constitutive Equations

Another example of needed developments in constitutive equations is in the area of plasticity theory. It has been found by experimentation that most, if not all, soils obey a non-associative flow law in undergoing plastic deformation. The term "non-associative" refers to the fact that the yield function which describes the stress state at which a material yields and the plastic potential which governs the plastic flow are not the same function. It has also been found by experimentation that the Mohr-Coulomb yield function does not accurately represent the actual stress state at which soils yield. It is a conservative criterion, always underpredicting the stresses at which soils yield. It has also been found by experiment (Lytton, et al, 1993) that the plastic yield of asphalt concrete obeys the same yield criteria as unsaturated soils. Figure 10 shows the projection on the octahedral plane of the Mohr-Coulomb and Lade-Duncan yield functions (Lade and Duncan, 1973) which illustrate how well the latter matches the measured data. The Mohr-Coulomb yield function represents a lower bound of practical yield functions. It is

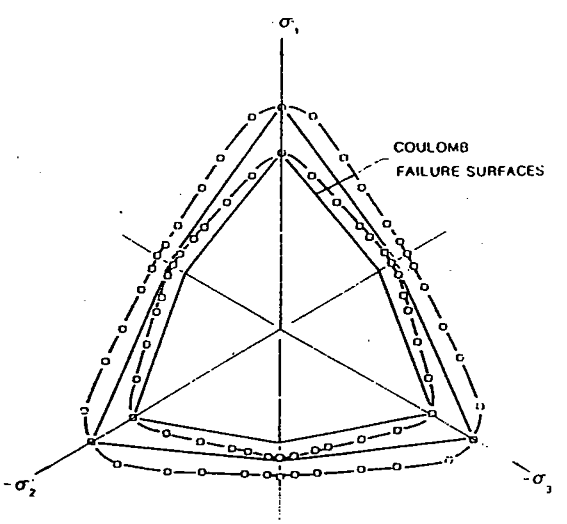


Figure 10: Mohr - Coulomb and Lade Yield Functions Compared with Measured Yield Data (Lade and Duncan, 1973)

sufficient to note here that in estimating the plastic flow of asphalt concrete and base course materials under increasing truck loads and tire inflation pressures, such inaccuracy is a luxury that can ill be afforded.

Similarly, in estimating the conditions of slope failure either by the classic slip surface or the shallow slope failure that is common in unsaturated soils, more accurate yield functions and plastic potential functions are needed.

The more promising functions that have been proposed for these purposes are the Vermeer, Lade, and Desai function (Vermeer, 1984; Lade, 1987; and Desai, et. al., 1991). In all of these, as expected, both the yield functions and plastic potential functions are functions of the first invariant of the stress tensor. This means that in unsaturated soils, including asphalt concrete, the expression for the first invariant of the stress tensor should be as in Equation (18) which is repeated below.

$$I_{1u} = I_1 - 3\theta f h_m \tag{18}$$

This is another use that can be made of the theoretical development in Example No. 1. Of course, in order to take advantage of these new formulations much more use will need to be made of numerical computational methods which incorporate them, and of testing methods which are capable of accurately determining the needed material properties.

1.8 Testing Methods

Testing methods may be divided conveniently into three categories:

- Characterization tests
- Laboratory tests for construction and rehabilitation projects
- In situ tests

Examples of each of these types of tests that need to be developed or to be brought more into practice in the future are listed below. The lists are not exhaustive but are meant to suggest the directions in which future testing needs to develop.

1.8.1 Characterization Tests

The characterization tests are made in the laboratory and are different for foundations and pavements. In foundation applications, it would be very desirable to develop a laboratory test procedure in which both suction and mechanical pressure stress paths could be controlled while both axial and radial strains are measured. This is to make it possible to characterize both volumetric and deviatoric behavior of unsaturated soils, and to determine the resilient dilatancy properties of the soil. In accordance with testing principles that are long established, the test and measurement geometry should be such as to permit all measurements of displacement to be made in a part of the test sample where there is a uniform stress and strain field so that the measurements reflect a pure material response. Thus, the center portion of a triaxial apparatus is acceptable. However, because no part of a direct shear apparatus has either a uniform stress or strain field, it is not usually acceptable. The results from such an apparatus should be regarded as questionable until confirmed by test results from an acceptable apparatus.

In pavement applications, the properties of pavement materials must be known over a wide range of stress, strain rate, and temperature conditions. As a result, asphalt concrete tests should be more widely used which employ triaxial frequency sweep (0.01 Hz to 20 Hz), creep and recovery, and fracture and healing tests at different temperatures and confining pressures. Portland cement concrete tests involving the measurement of total suction, temperature and shrinkage coefficients, and fracture properties should begin to see much more frequent use .

1.8.2. Laboratory Tests for Construction and Rehabilitation Projects

These types of tests should be related to and derived from the characterization tests principally because they produce material properties. And it is material properties that govern the performance of a foundation or a pavement.

In foundation applications, few tests will prove to be more useful than rapid but accurate methods of measuring the suction in unsaturated soils. Methods such as filter paper (slow but simple), transistor psychrometers, and chilled mirror optical dewpoint sensors should prove to be useful for this purpose. In volumetrically active soils, the ability to measure the volume change - versus - suction characteristic of an unsaturated soil under zero or low pressure will be very useful. The same tests will be useful for pavement base courses and subgrade materials.

1.8.3. In Situ Tests

These types of tests should produce rapid and reliable measurements under field conditions. In foundation applications, suction probes using either the transistor psychrometer or the chilled mirror optical dewpoint sensor will be needed to measure total and osmotic suction. Compaction of landfill liners and caps should be controlled by suction probes rather than by the conventional earthwork QA/QC equipment. Lateral earth pressure needs to be measured under conditions of changing suction and fiber optic sensors may be a promising method for this. Ground penetrating radar with the reflected signals filtered for noise and properly analyzed is capable of accurate measurements of stratum thickness, voids, water content and density.

In pavement applications, the measurement of total and osmotic suction in base courses and subgrades can be accomplished with suction probes. Resilient properties, including viscoelastic properties of all pavement layers, can be determined by inverse analysis of the time histories of load and deflections measured in impulse testing. Ground penetrating radar can be used to measure layer thickness, voids, water content or asphalt content and density, and the presence and thickness of ice lenses. Soil mass properties of pavements that can be measured include the profile, roughness spectrum, and variability.

1.9 Analysis Methods Used in Design

In the future, numerical computational methods will be used more widely and for more routine use in design. This will be driven by the availability of inexpensive computers with the required memory and speed and of testing methods that are capable of measuring accurate material properties. The developments that are needed in foundations include analysis - for - design methods for slabs, drilled piers, retaining walls, and downhill creep. In pavements, analysis - for - design methods are needed for Portland cement concrete, asphalt concrete, and unpaved roads.

Slab design methods that are needed include the ability to analyze non-rectangular foundation shapes with and without stiffening beams and for a variety of soil distortion patterns. The effects of water proofing with root and moisture barriers needs to be considered.

Drilled pier design methods will make use of a cylindrical pier acted upon by uplift and down drag forces caused by swelling and shrinking of the surrounding soil. In addition, differential wetting and drying around the pier will generate unsymmetrical lateral earth pressures and moments in the pier. Interface elements must be used to represent the normal and tangential forces imparted by the soil to the drilled pier.

Retaining wall design will need to use a soil - structure interaction analysis employing interface elements and a lateral earth pressure formulation akin to the one considered in Example No. 3, elements and a formulation, which includes the effects of changing suction levels.

Numerous foundation elements including slabs and drilled piers will need to be designed to accommodate the downhill movement due to downhill creep. One of the first design considerations is whether the slope will tend to creep downhill or will undergo a shallow slope failure. Downhill creep will occur if the cohesive shear strength of the soil is larger than the downhill component of the overburden pressure. A lower bound inequality describes the condition in which downhill creep will occur.

$$H \tan\alpha \leq \frac{\theta f |h_m|}{\gamma_t} \tan\phi' \qquad (42)$$

where

- H = the thickness of a layer which is creeping downhill
- $\tan\alpha$ = the tangent of the slope angle
- γ_t = the total unit weight of the soil
- θ = the volumetric water content
- f = the bracketed terms in Equations (8) and (9). The minimum value of f is 1.0
- $|h_m|$ = the absolute value of the matric suction
- $\tan\phi'$ = the friction angle of the material

If the inequality sign is reversed, shallow slope failure may occur if water is trapped in the cracks in the slope so that the saturated effective strength of the soil in the cracks is less than the cohesive shear strength of the intact soil. This effect of cracks in the soil on a sloping site shows the importance of waterproofing the site above the level of a foundation. The lateral pressure applied by the mass of soil creeping downhill against drilled piers becomes a critical part of the analysis and design of those drilled piers.

Asphalt concrete pavements will need to have analysis - for - design methods in use which accurately predict the principal types of distress including rutting, fatigue and thermal cracking, pumping, stripping, raveling, and weathering. The latter three are controlled by the adhesive surface energies of asphalt and aggregates as discussed in Example No. 5.

Portland cement concrete pavements need to have analysis - for - design methods which accurately predict the severity and extent of faulting, spalling, cracking due to warping, curling, and traffic, and pumping.

Unpaved roads constitute large proportions of the transportation networks of all nations. Their proper management will require analysis - for - design methods which make accurate predictions of rutting, surface loss, and corrugations.

Several types of pavement distresses listed above, namely rutting, pumping, surface loss, and corrugations will require the use of numerical computational methods with the capability of allowing the Poisson's ratio to rise well above 0.5.

1.10. Summary

It is apparent that unsaturated soils cover a broad spectrum of the materials of construction including

Expansive soil
Collapsing soil
Frozen soil
Fine and coarse grained soils
Asphalt concrete

Foundations and pavements on these soils must be designed to perform as they are predicted, making use of the characteristics of these unsaturated soils in numerical computational procedures. The materials properties of these soils are stress -and -suction -dependent, and are variable. These soils undergo large strains under service conditions in the field. These characteristics of unsaturated soils made small strain, elastic analyses generally inadequate for the purposes of accurate prediction. Realistic characterization of these soils is necessary for analysis which, in turn, is necessary for design.

Future progress in unsaturated soils requires the development of Theory of mixtures and micromechanics concerning unsaturated soils

- Constitutive equations
- Test method for laboratory and in situ measurements
- Computational methods to include realistic unsaturated soil properties
- Analysis -for -design methods for foundations and pavements
- Design methods that are based upon accurately predicted performance
- Use of the reliability approach in design which accounts for variability and uncertainty
- Nondestructive testing methods to determine in situ properties of unsaturated soils
- Well planned case studies

Rapid improvements in computers and instrumentation are making all of these developments both possible now and practical in the near future.

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SEMINAR

FOUNDATIONS ON EXPANSIVE SOILS

SHERATON – NORTH HOUSTON

HOUSTON, TEXAS

SEPTEMBER 23, 2005

PROGRAM AGENDA

6:45 am Registration

➢ **7:15 am Seminar Opening by David Eastwood, P.E.**

7:25 am Tribute to Professor Michael O'Neill –

Mr. Kenneth Tand, P.E. –

Kenneth Tand and Associates, Inc.

7:35 am Introduction to Unsaturated Soil Mechanics by

Dr. Robert L. Lytton, P.E.

8:00 am Computations of Swell and Shrinkage in Expansive

Soils – Lytton

Development of Volume Change parameters

Use of Soil Suction Concepts

Comments on PVR

8:45 am Break

8:55 am Continuation – Lytton

9:30 am Field Exploration and site Conditions – Meyer

10:00 am Laboratory Testing – Lytton

Swell Test Procedures

Soil Suction Tests

10:30 am Break

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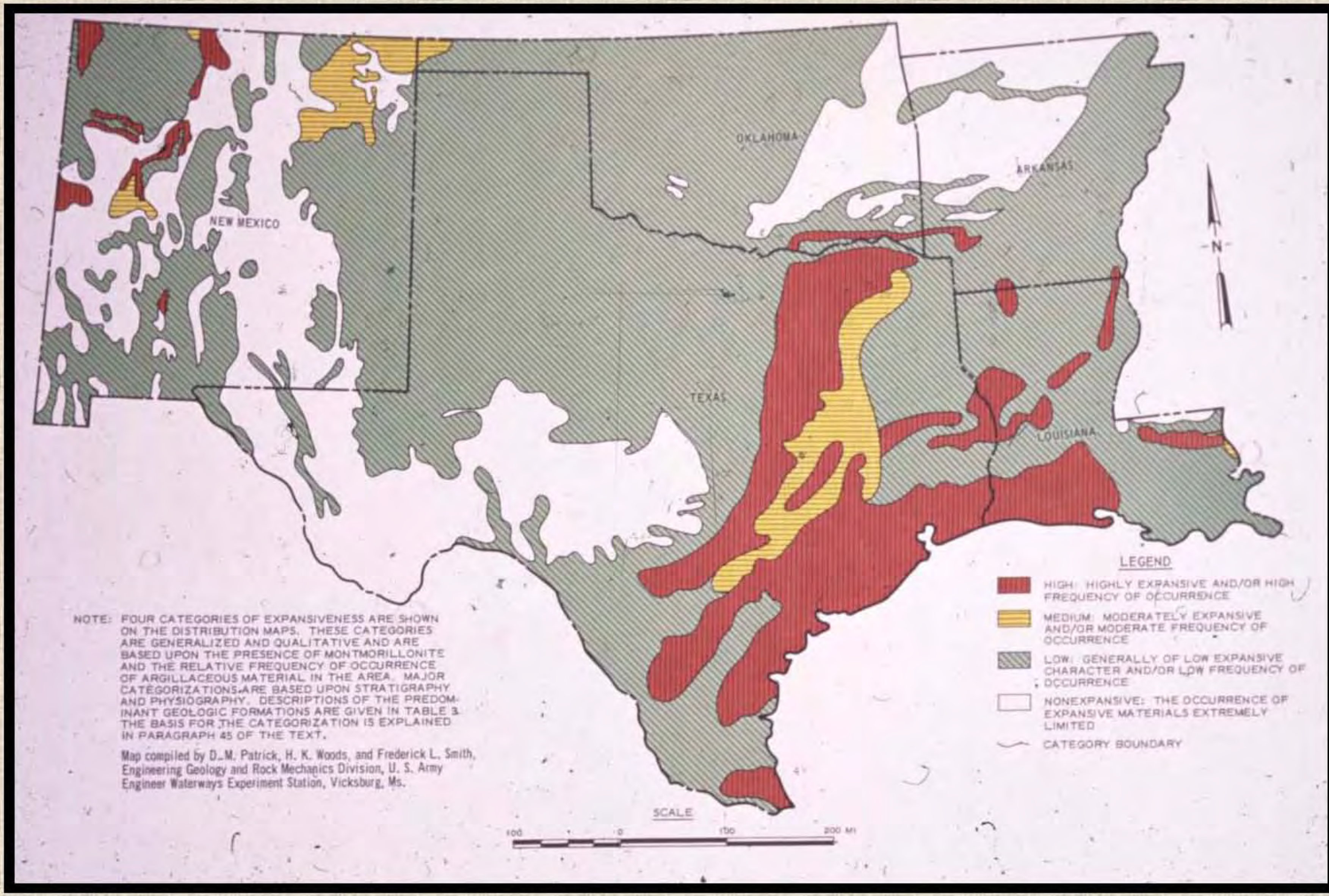
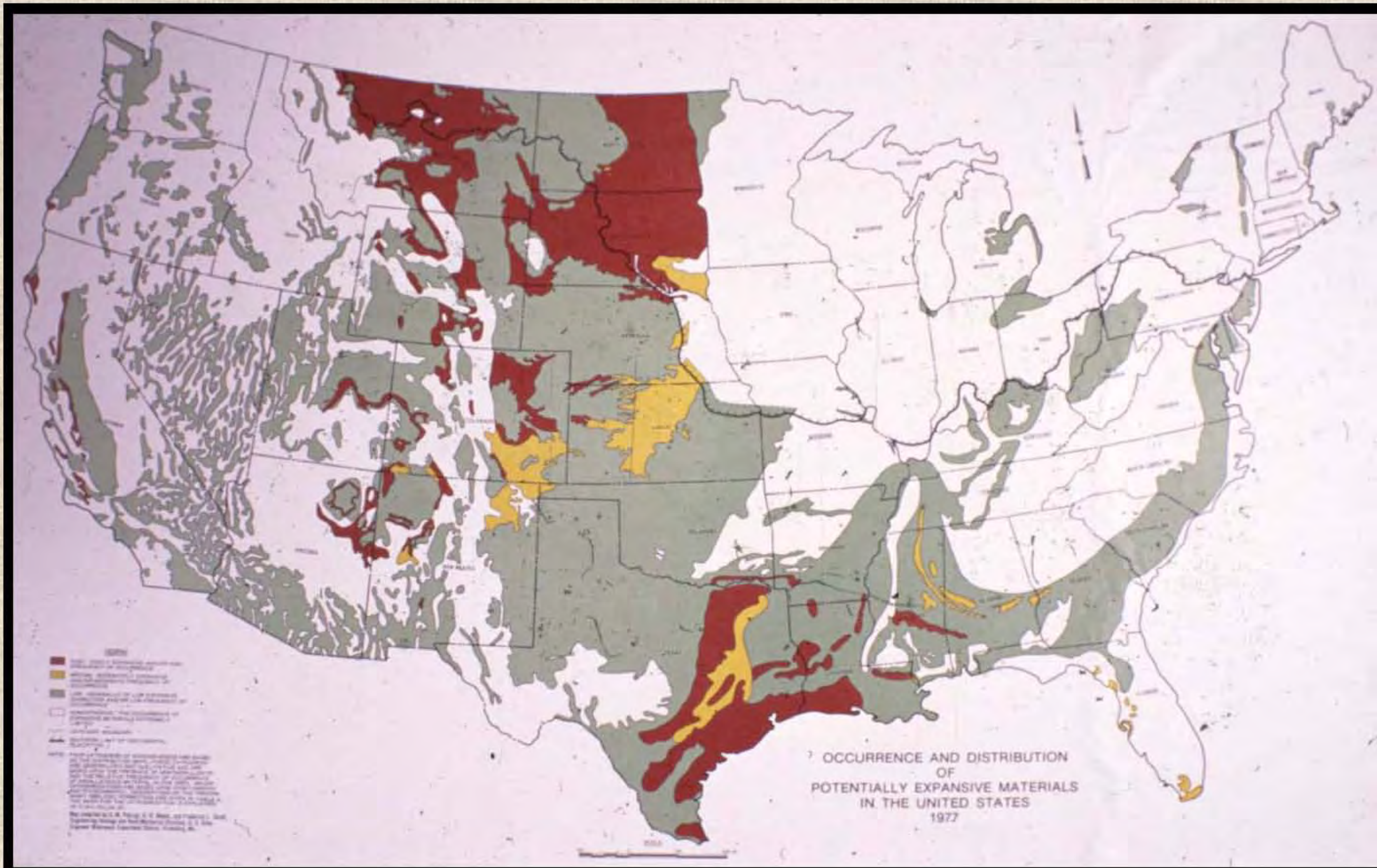
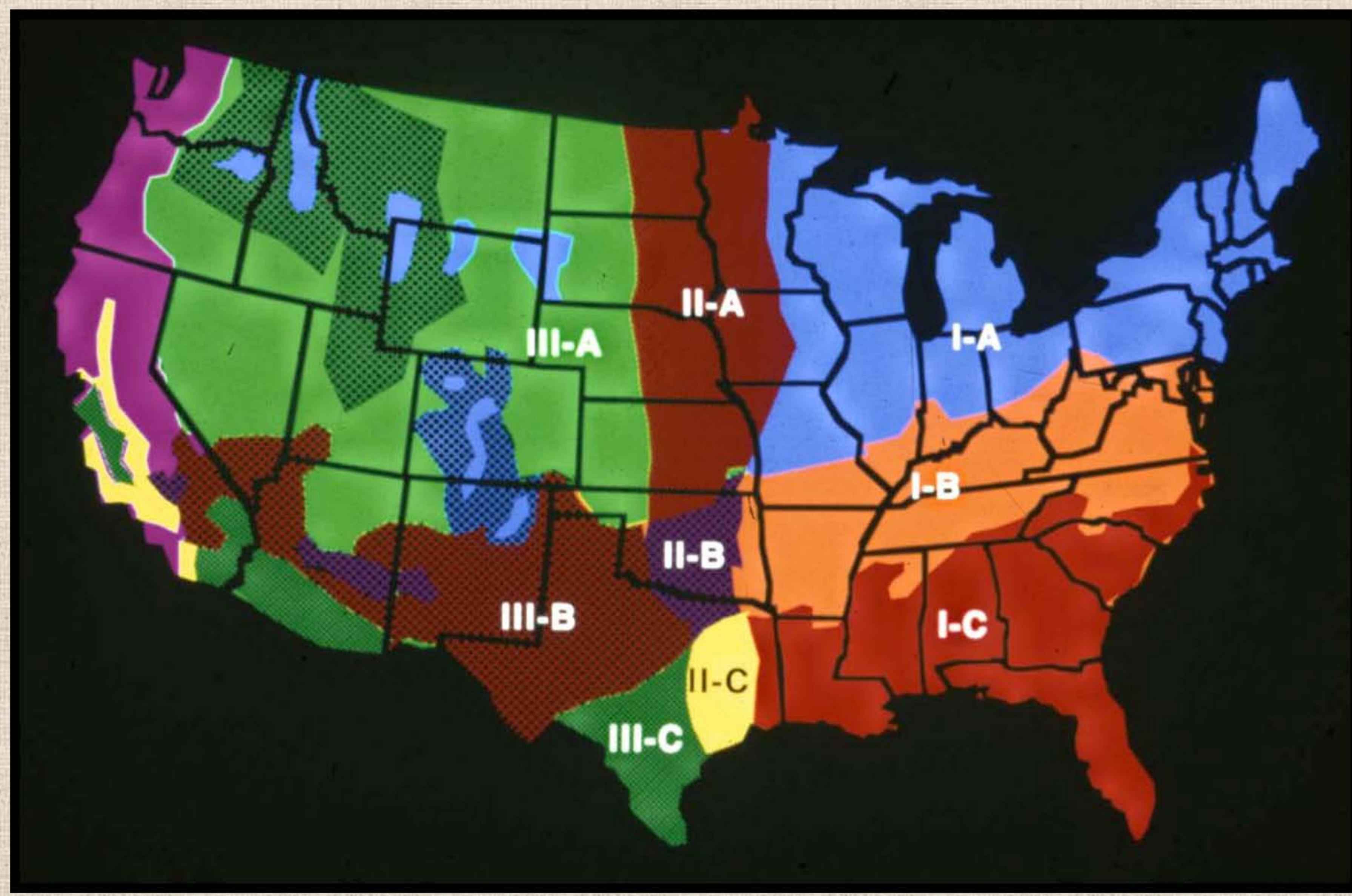
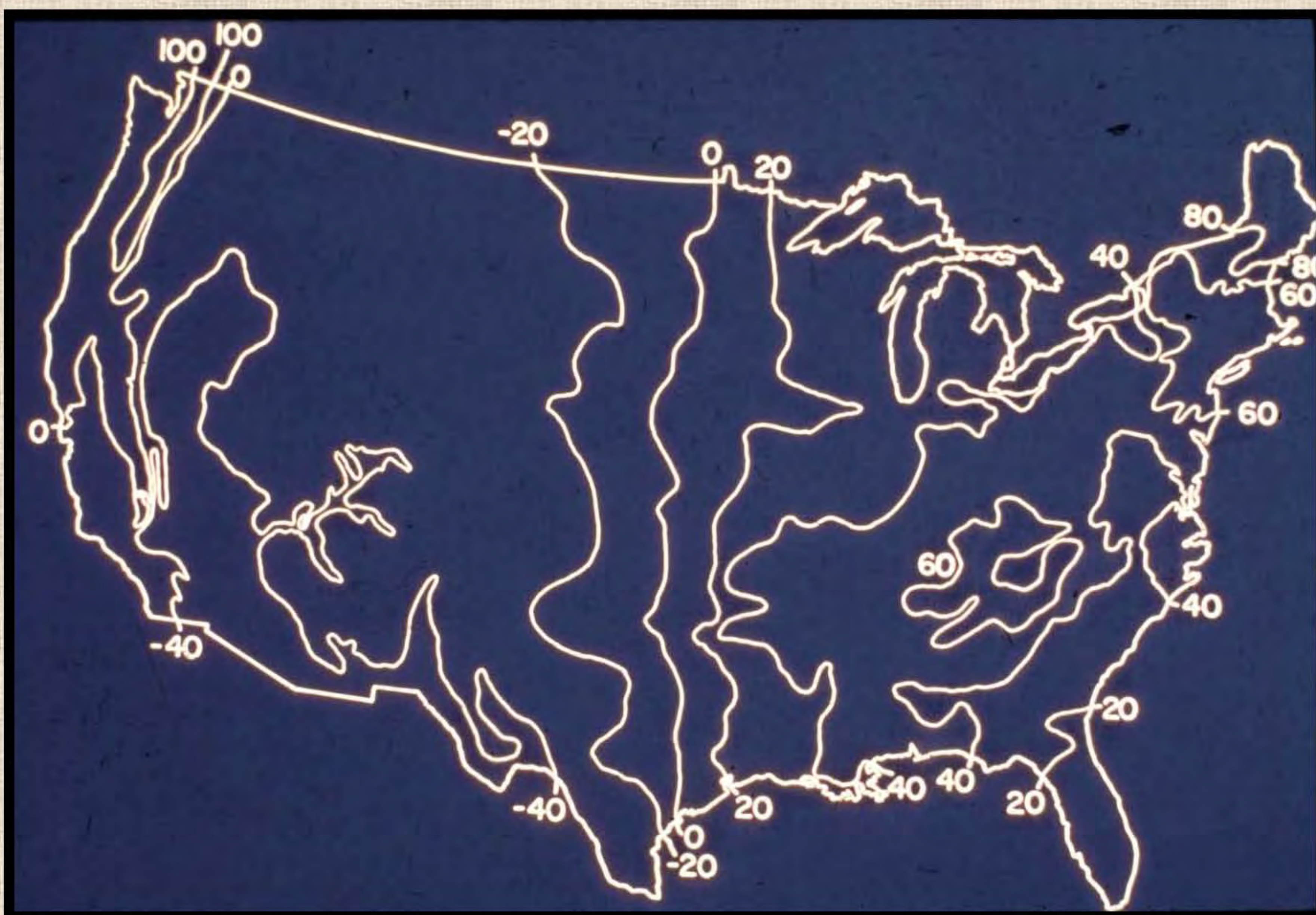
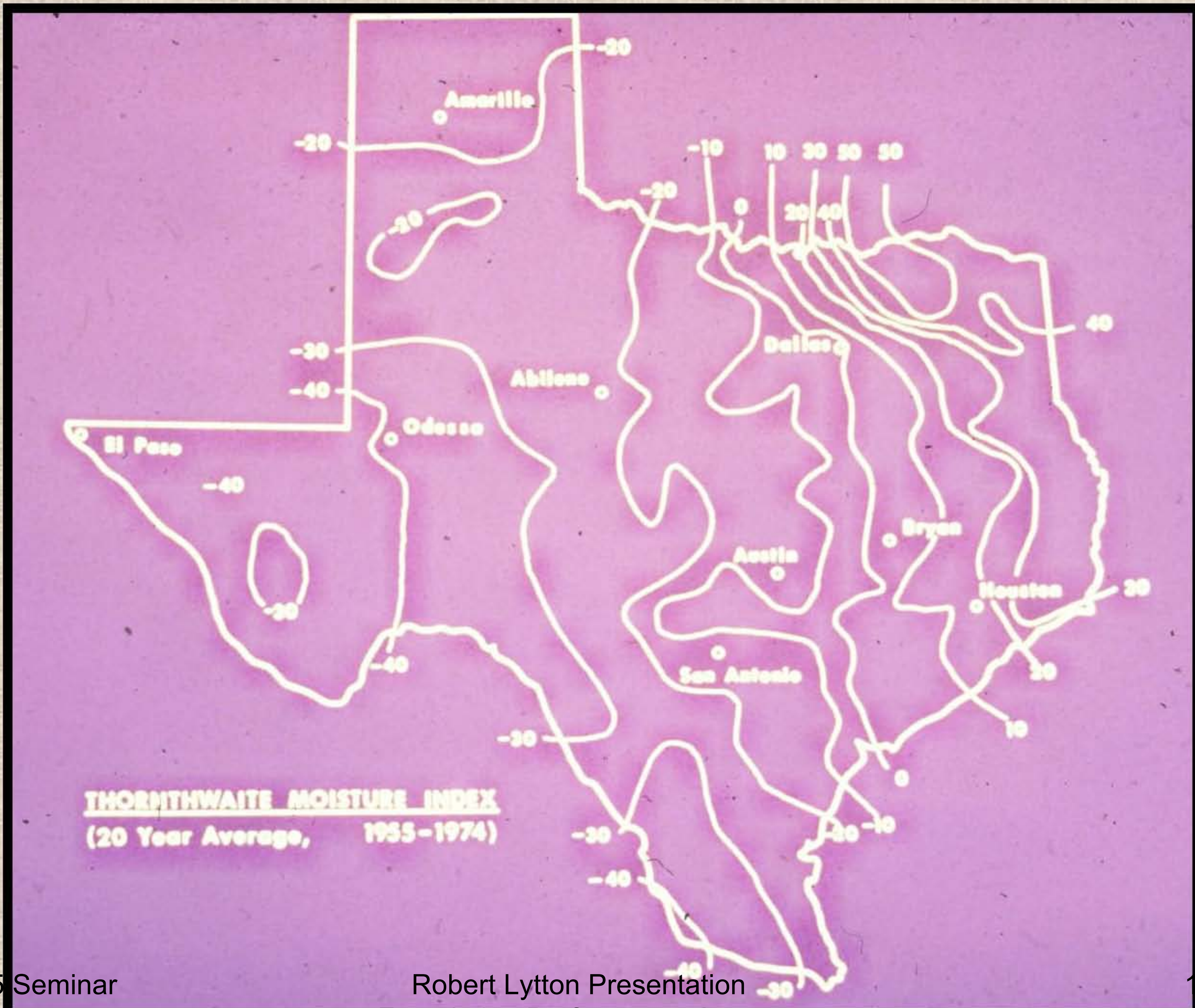
LEGEND
HIGH
MEDIUM
LOW

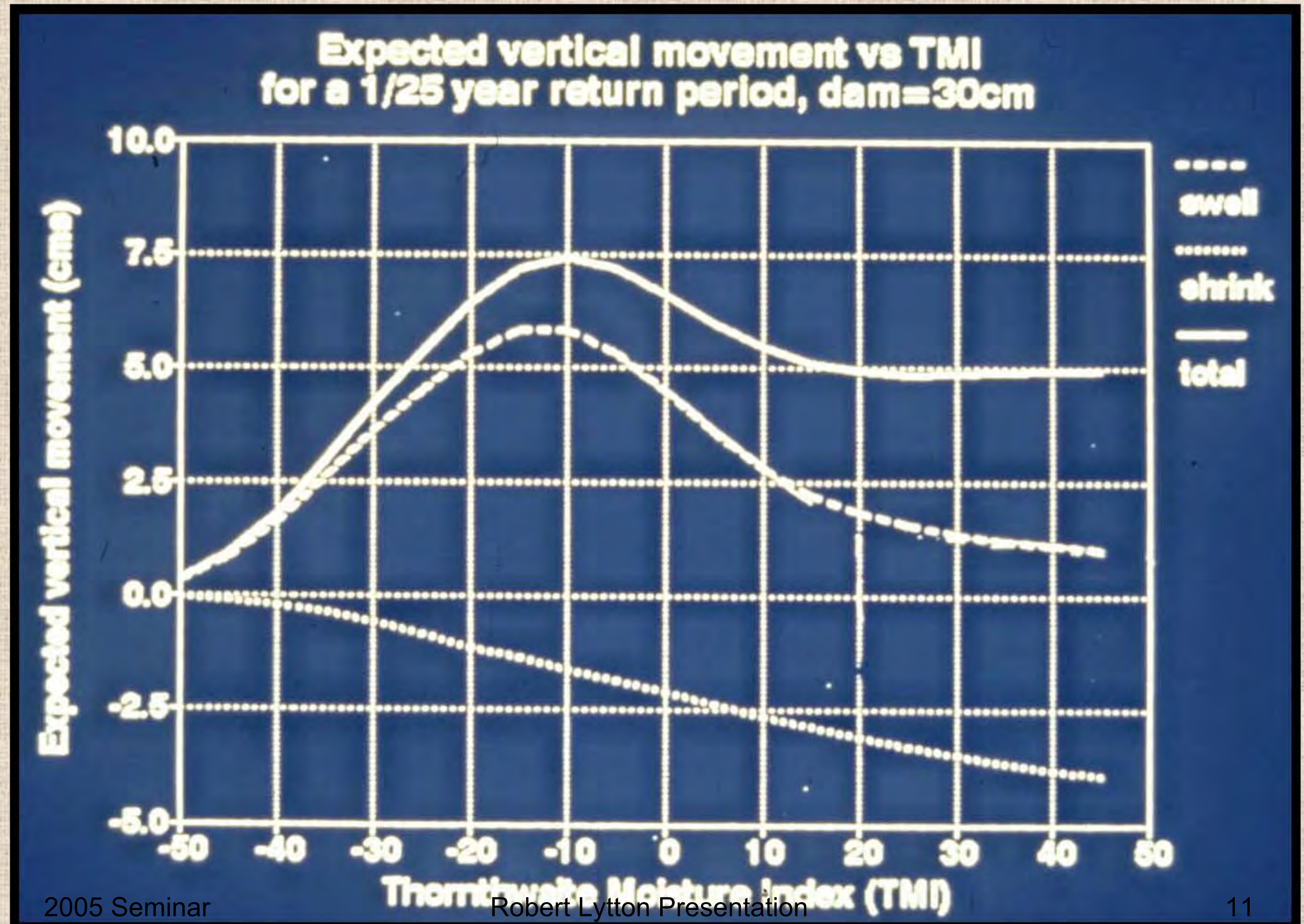

OKLAHOMA
ARKANSAS
NEW MEXICO
TEXAS
LOUISIANA
N
LEGEND
HIGH: HIGHLY EXPANSIVE AND/OR HIGH FREQUENCY OF OCCURRENCE
MEDIUM: MODERATELY EXPANSIVE AND/OR MODERATE FREQUENCY OF OCCURRENCE
LOW: GENERALLY OF LOW EXPANSIVE CHARACTER AND/OR LOW FREQUENCY OF OCCURRENCE
NONEXPANSIVE: THE OCCURRENCE OF EXPANSIVE MATERIALS EXTREMELY LIMITED
CATEGORY BOUNDARY
NOTE: FOUR CATEGORIES OF EXPANSIVENESS ARE SHOWN ON THE DISTRIBUTION MAPS. THESE CATEGORIES ARE GENERALIZED AND QUALITATIVE AND ARE BASED UPON THE PRESENCE OF MONTMORILLONITE AND THE RELATIVE FREQUENCY OF OCCURRENCE OF ARGILLACEOUS MATERIAL IN THE AREA. MAJOR CATEGORIZATIONS ARE BASED UPON STRATIGRAPHY AND PHYSIOGRAPHY. DESCRIPTIONS OF THE PREDOMINANT GEOLOGIC FORMATIONS ARE GIVEN IN TABLE 3. THE BASIS FOR THE CATEGORIZATION IS EXPLAINED IN PARAGRAPH 45 OF THE TEXT.
Map compiled by D. M. Patrick, H. K. Woods, and Frederick L. Smith, Engineering Geology and Rock Mechanics Division, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Ms.
SCALE
100
0
100
200 MI

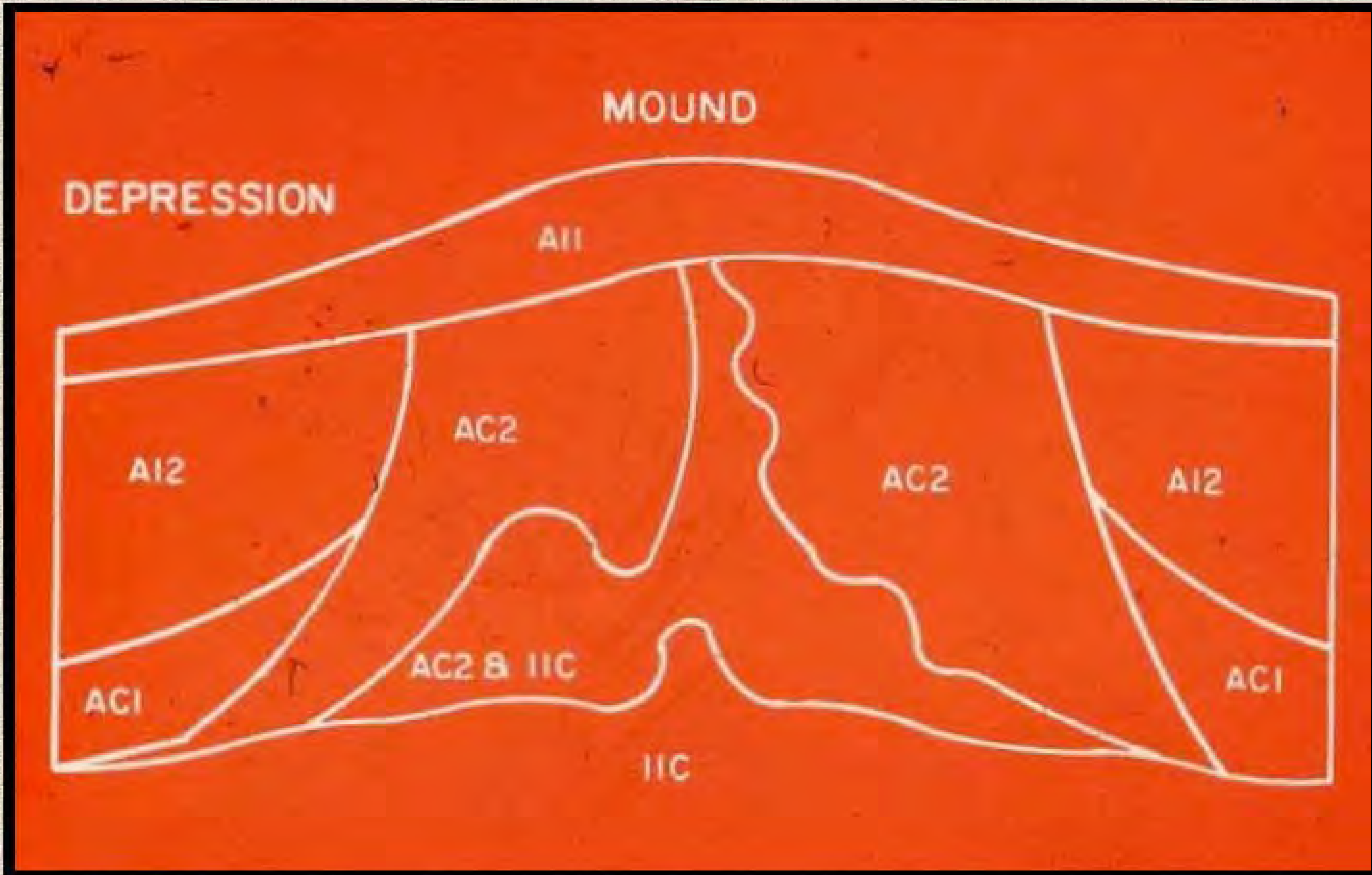

OCCURRENCE AND DISTRIBUTION
OF
POTENTIALLY EXPANSIVE MATERIALS
IN THE UNITED STATES
1977

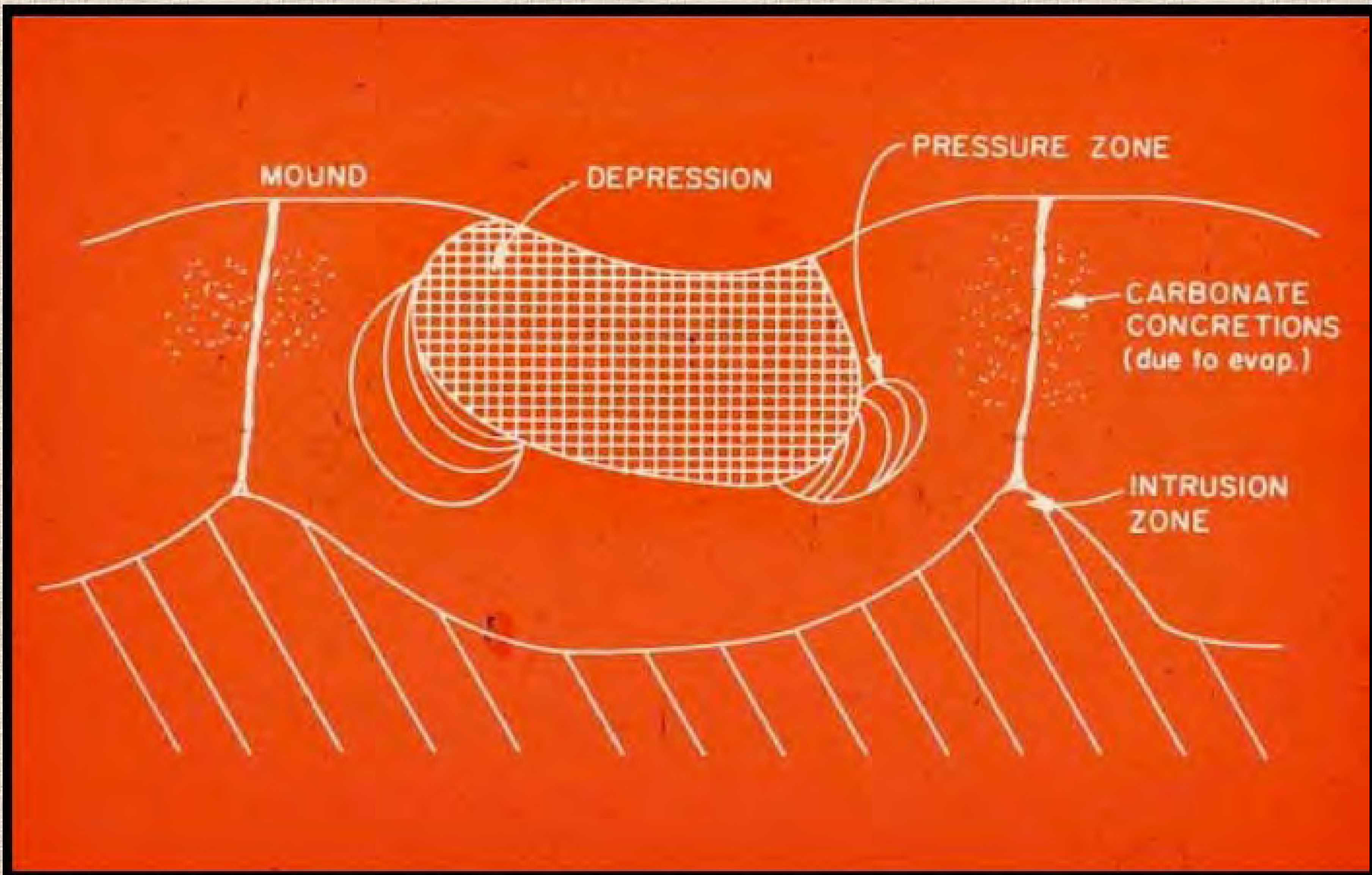

II-A
III-A
I-A
I-B
II-B
III-B
I-C
II-C
III-C

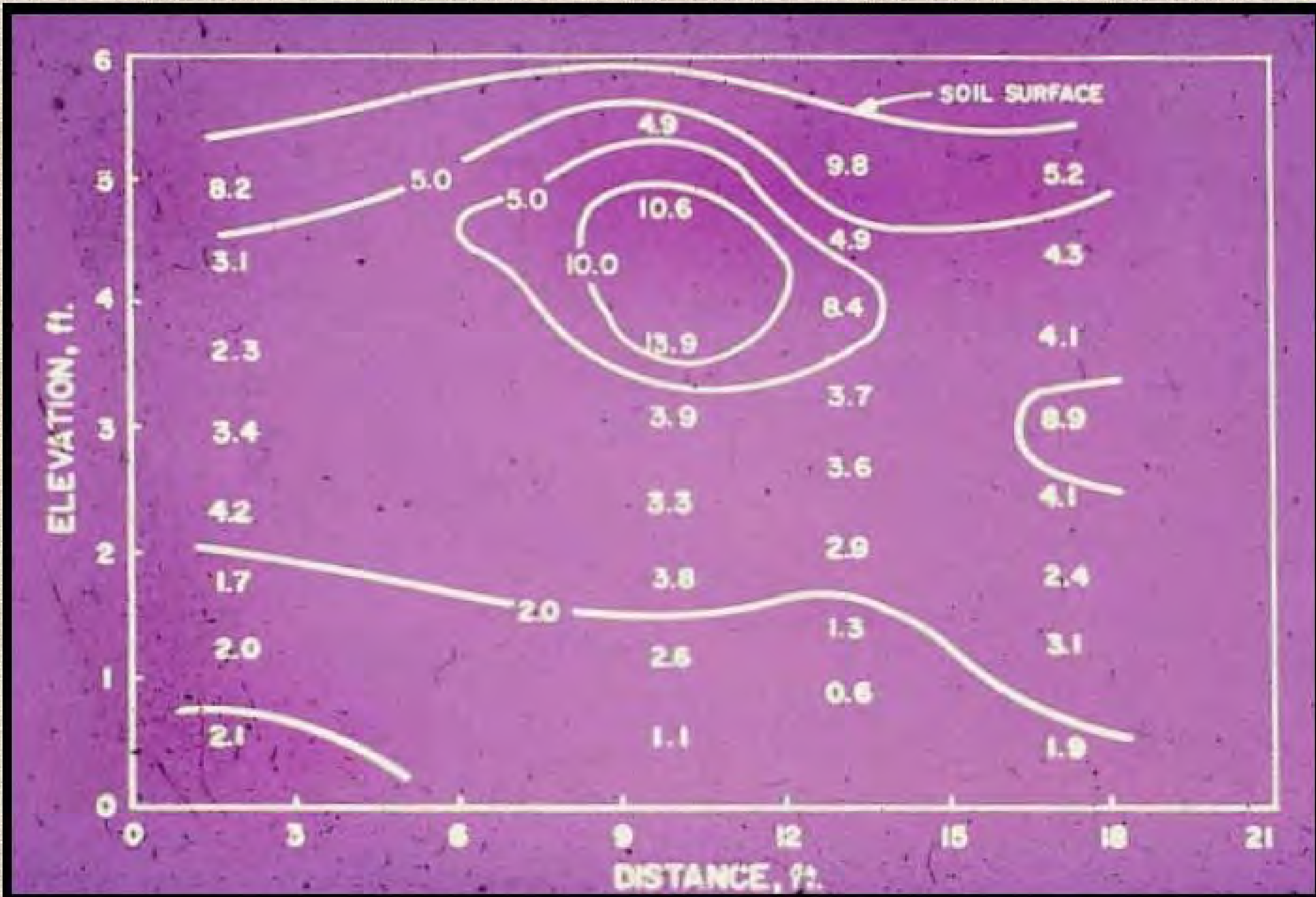

100
100
0
-20
0
20
80
40
60
0
60
60
40
-40
-40
20
40
40
20
20
0
-20

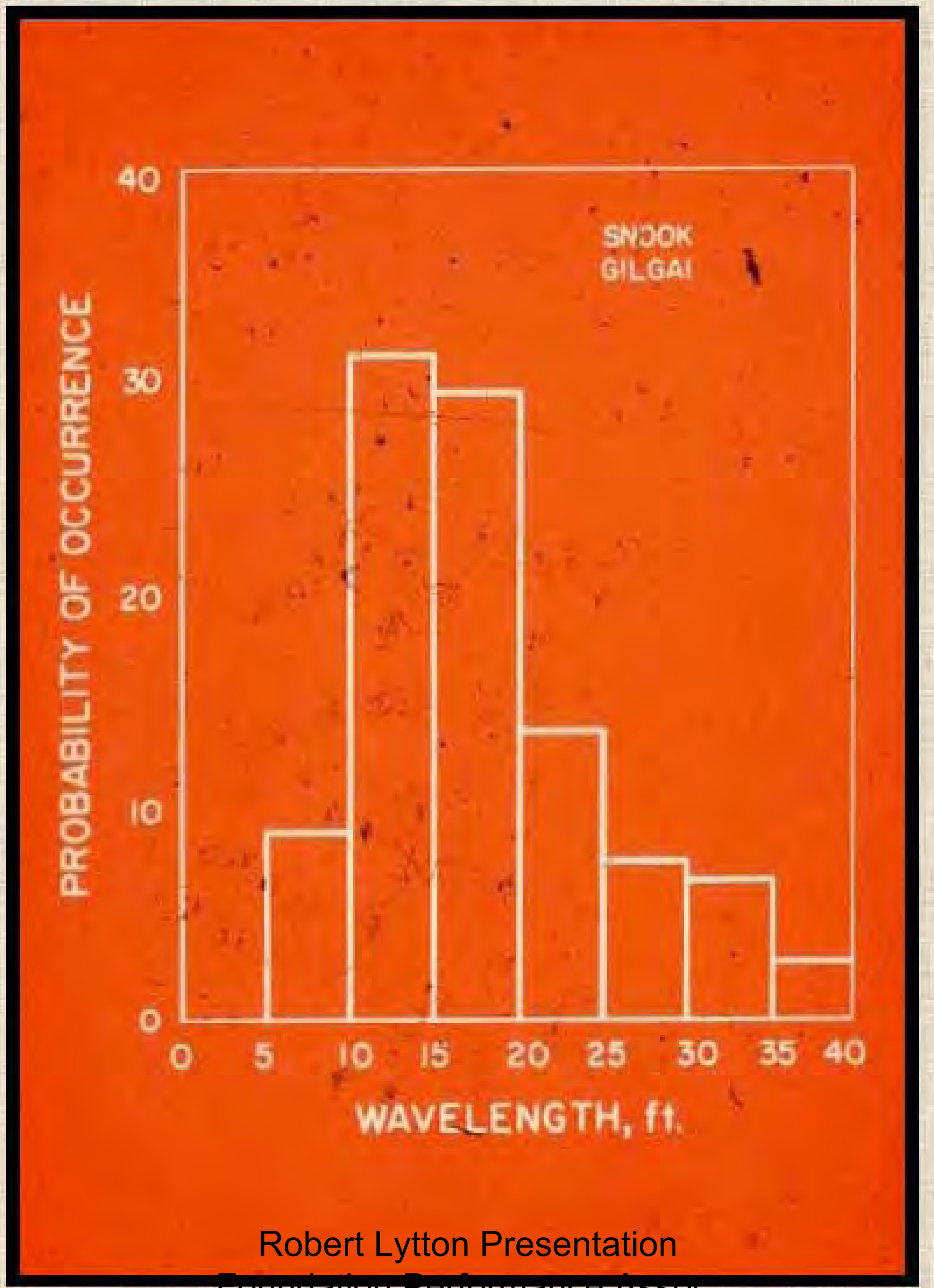

-20
Amarillo
-20
-10
10
30
50
50
-20
0
20
40
-20
40
-30
Dallas
Abilene
-40
Odessa
El Paso
-40
Bryan
Austin
Houston
30
-30
-40
San Antonio
20
-30
10
0
THORNTHWAITE MOISTURE INDEX
(20 Year Average, 1955-1974)
-30
-20
-10
-40
-40
-30

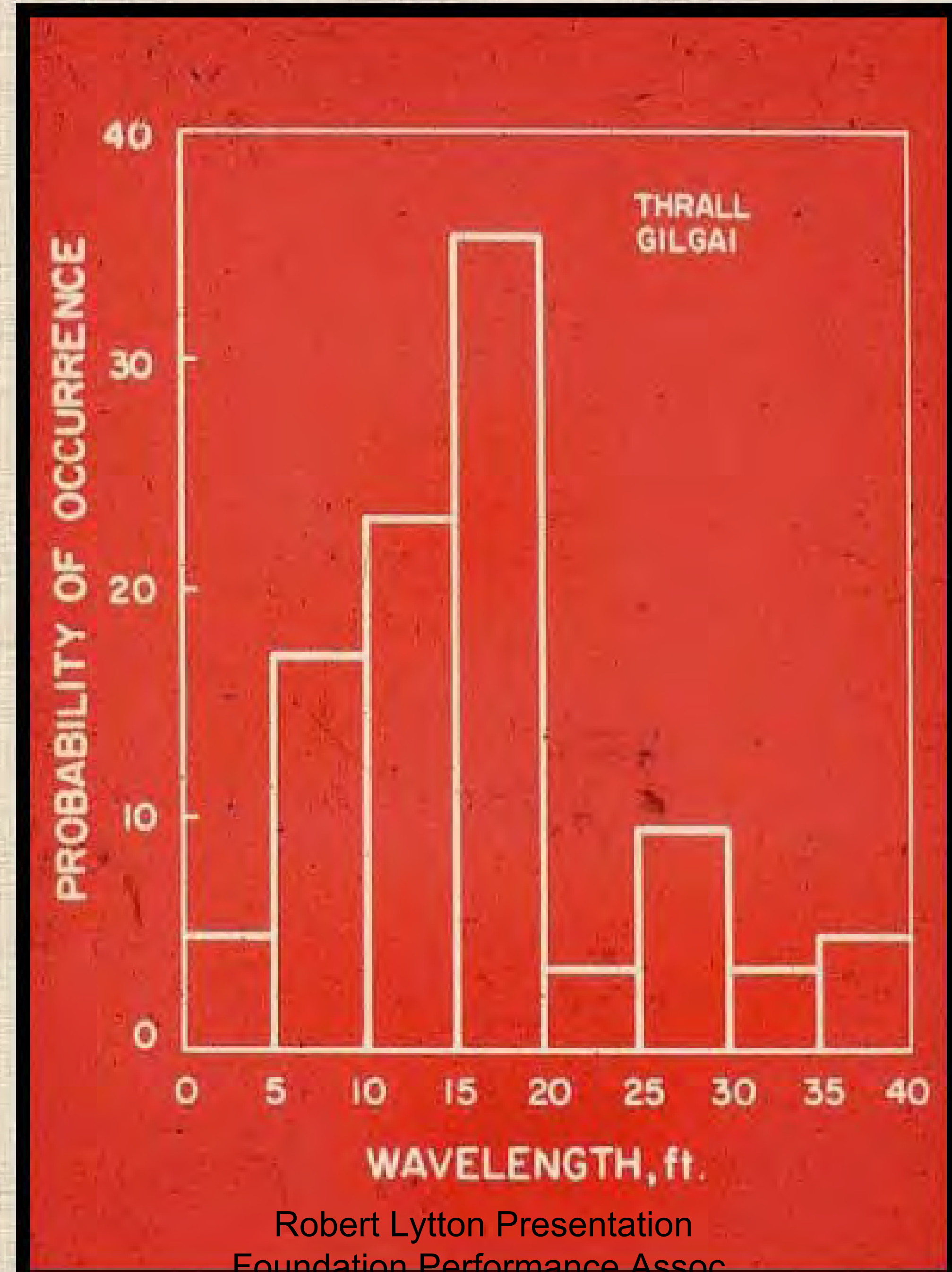

Expected vertical movement vs TMI
for a 1/25 year return period, dam=30cm
Expected vertical movement (cms)
10.0
7.5
5.0
2.5
0.0
-2.5
-5.0
-50
-40
-30
-20
-10
0
10
20
30
40
50
Thornthwaite Moisture Index (TMI)
swell
shrink
total

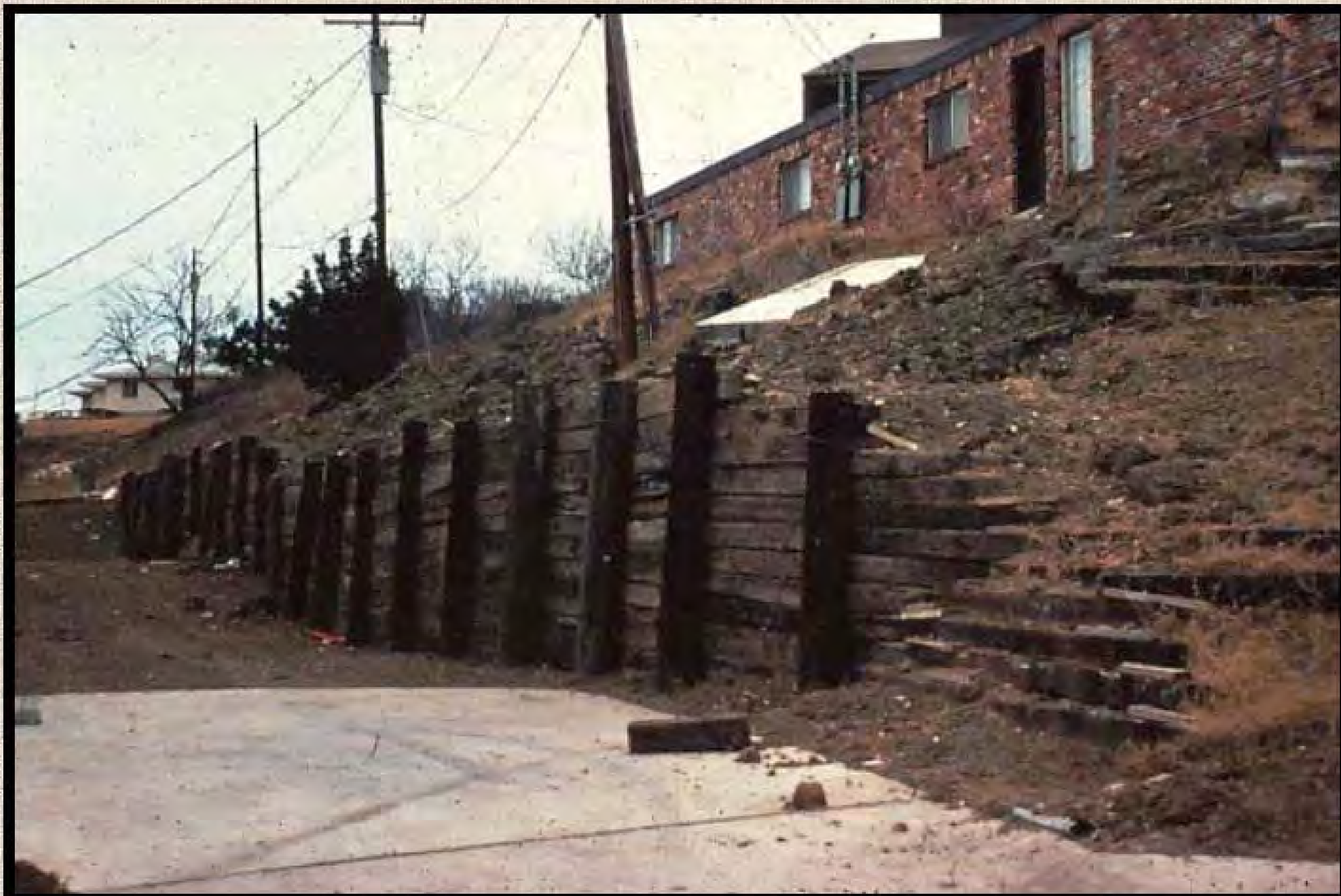

MOUND
DEPRESSION
AI1
AC2
AI2
AC2
AI2
AC2 & IIC
ACI
ACI
IIC


MOUND
DEPRESSION
PRESSURE ZONE
CARBONATE CONCRETIONS (due to evap.)
INTRUSION ZONE

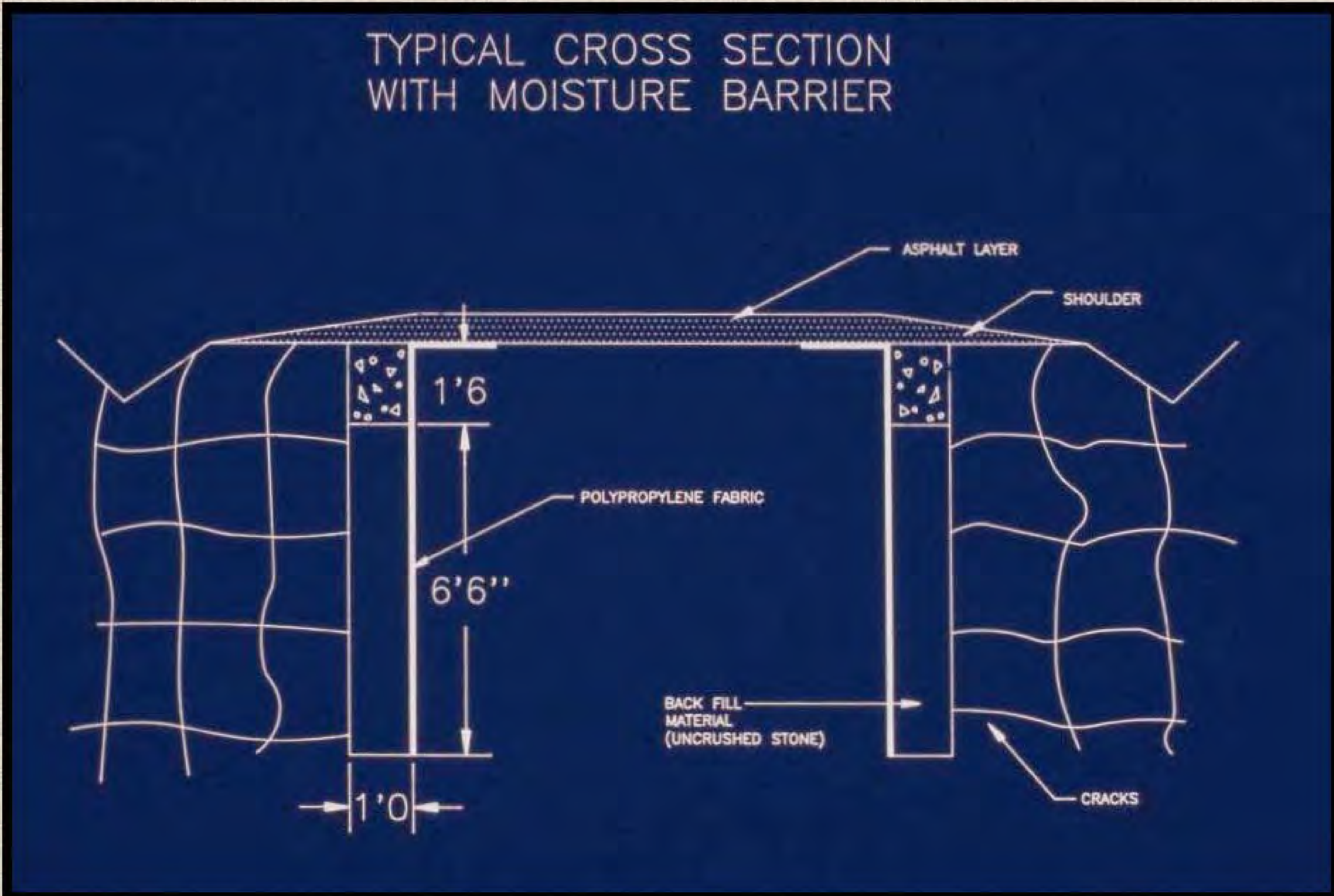

SOIL SURFACE
ELEVATION, ft.
DISTANCE, ft.
0
1
2
3
4
5
6
0
3
6
9
12
15
18
21
8.2
3.1
2.3
3.4
4.2
1.7
2.0
2.1
5.0
5.0
4.9
10.6
10.0
13.9
9.8
4.9
8.4
3.7
3.9
3.6
3.3
2.9
3.8
2.0
1.3
2.6
0.6
1.1
5.2
4.3
4.1
8.9
4.1
2.4
3.1
1.9

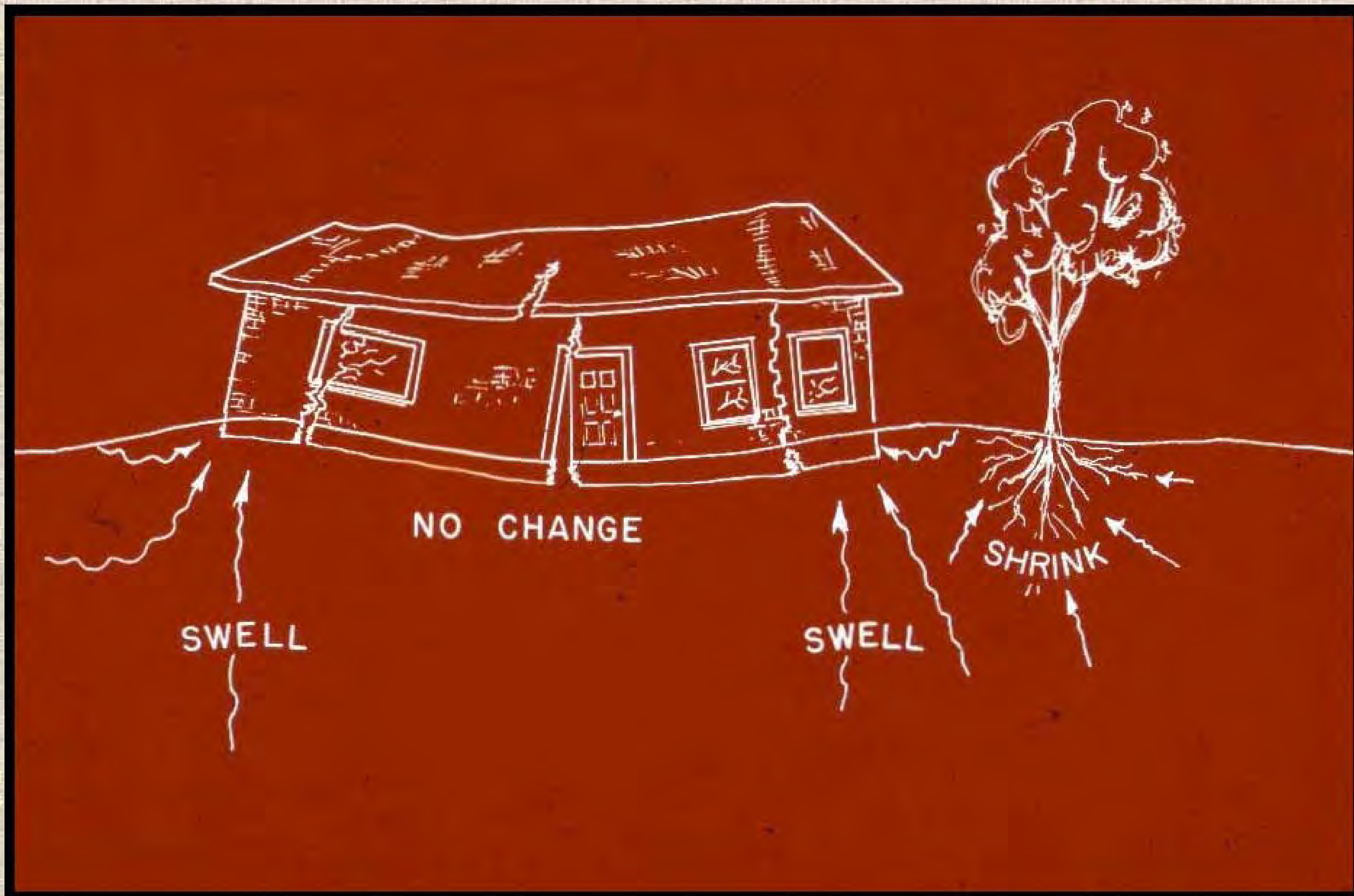

SNOOK
GILGAI
PROBABILITY OF OCCURRENCE
40
30
20
10
0
0 5 10 15 20 25 30 35 40
WAVELENGTH, ft.

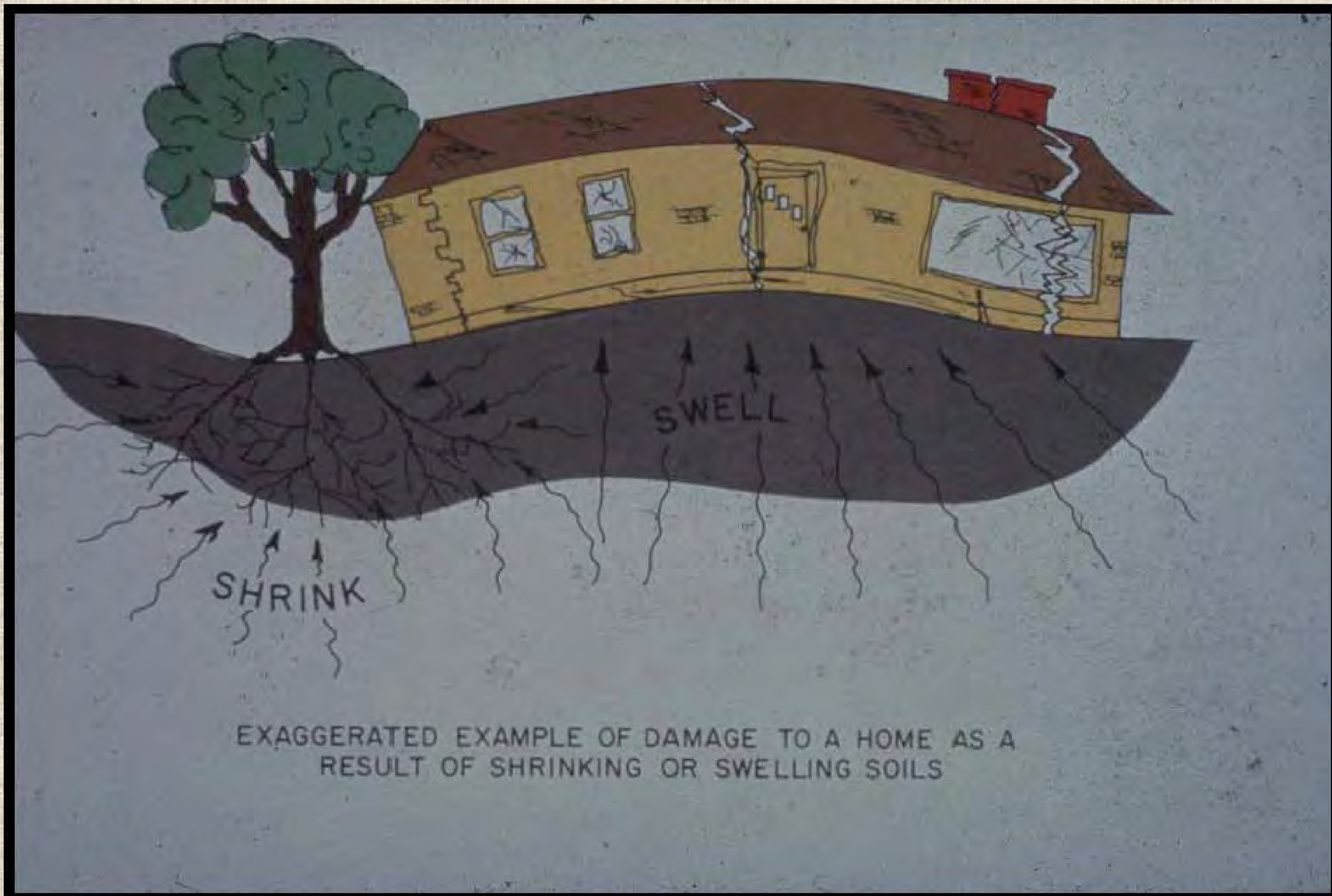

THRALL
GILGAI
PROBABILITY OF OCCURRENCE
0
10
20
30
40
0 5 10 15 20 25 30 35 40
WAVELENGTH, ft.



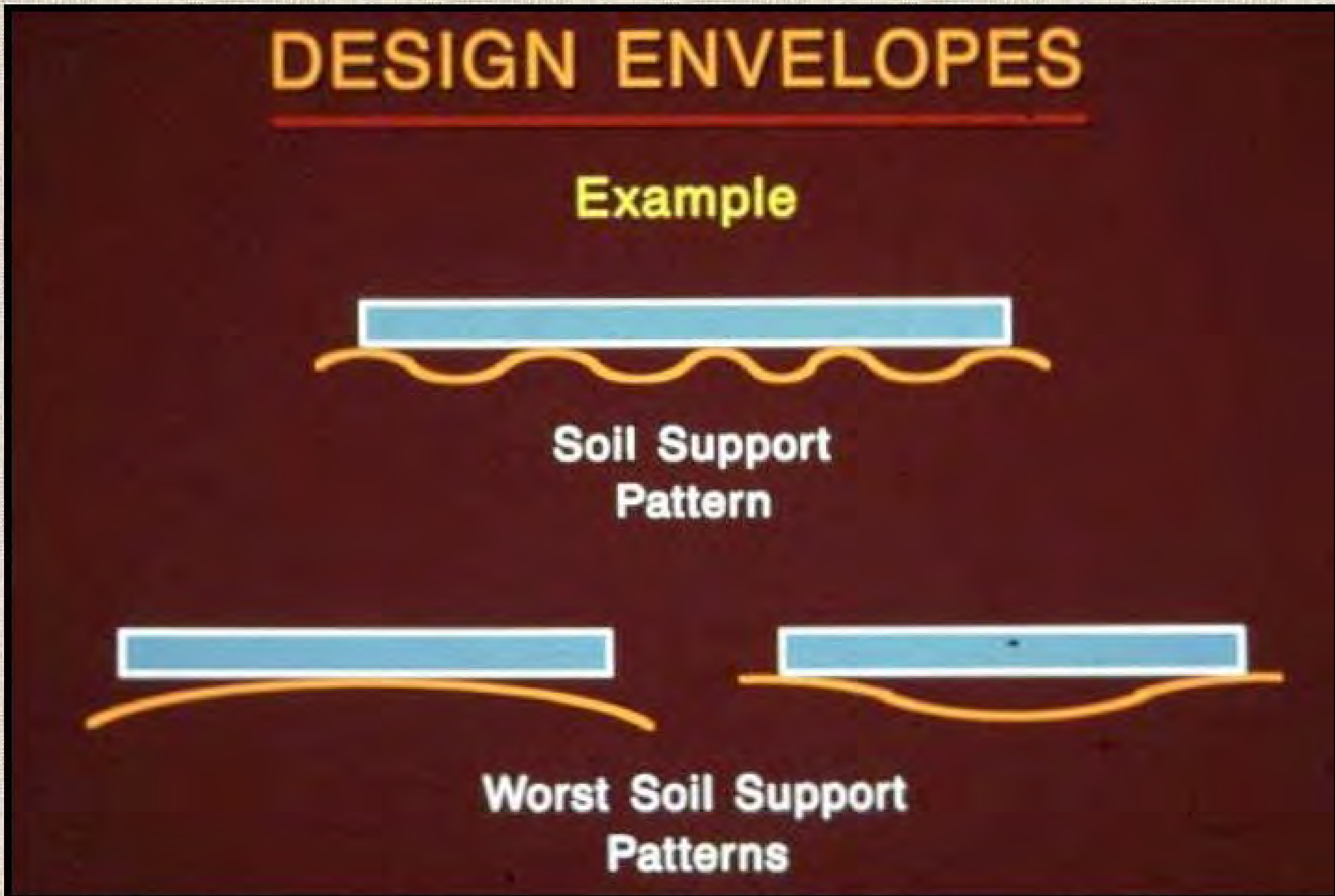

CEM. STAB. BACKFILL
GRAVEL BACKFILL
VERTICAL MOISTURE SEAL
PROPOSED
TYPICAL MAINLANE PAVEMENT SECTION

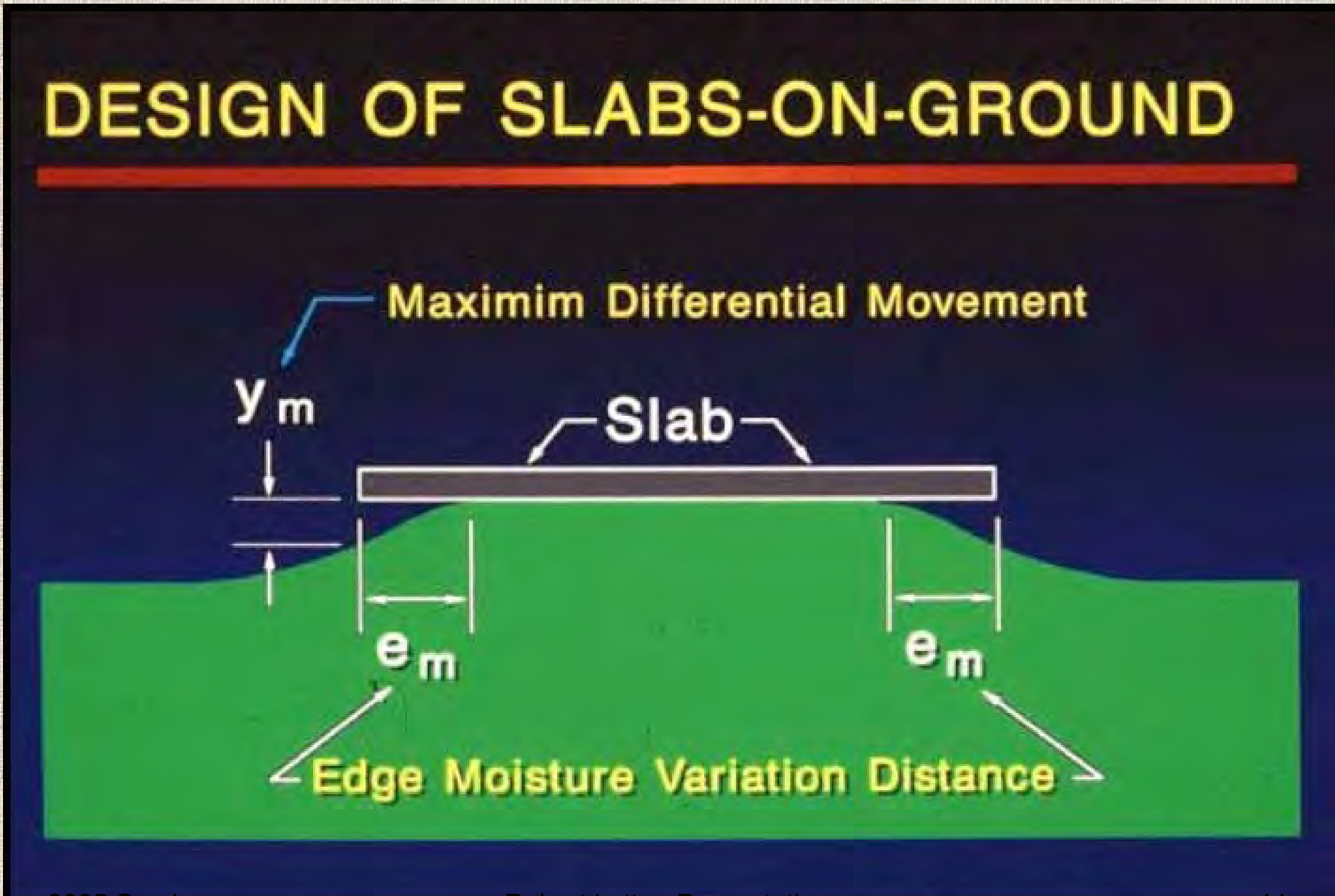

TYPICAL CROSS SECTION
WITH MOISTURE BARRIER
ASPHALT LAYER
SHOULDER
1'6
POLYPROPYLENE FABRIC
6'6"
BACK FILL
MATERIAL
(UNCRUSHED STONE)
1'0
CRACKS

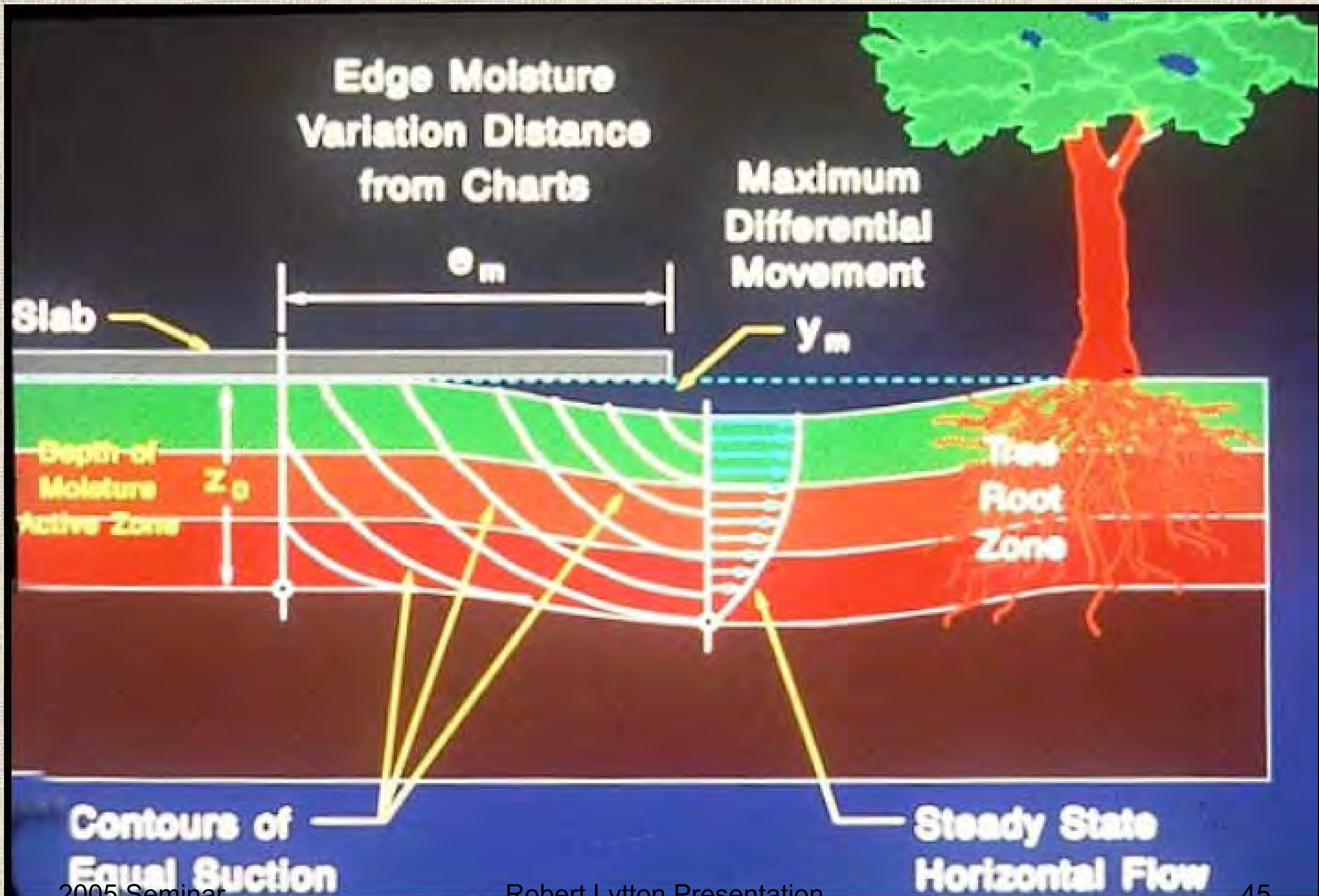

NO CHANGE
SWELL
SWELL
SHRINK

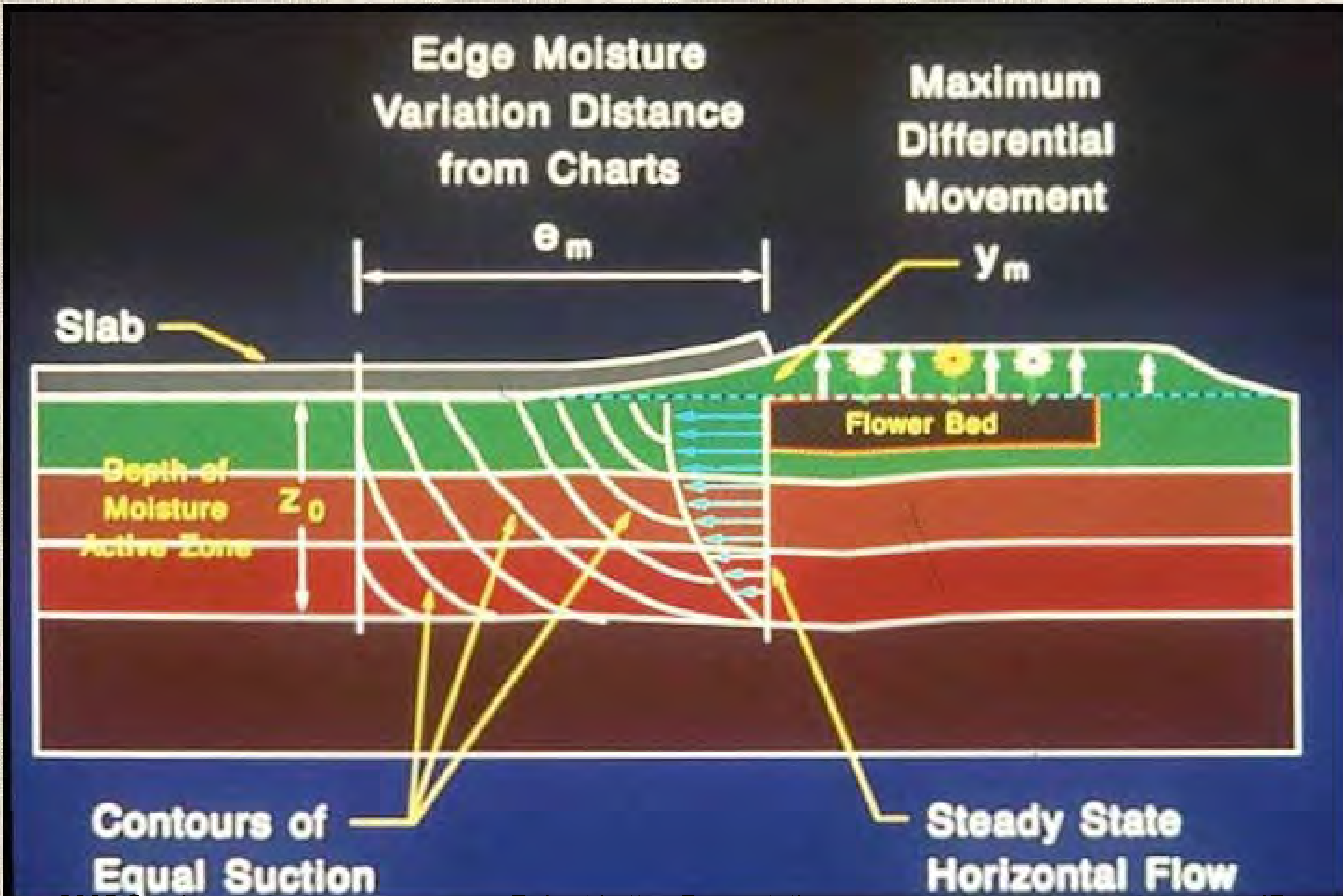

SWELL
SHRINK
EXAGGERATED EXAMPLE OF DAMAGE TO A HOME AS A
RESULT OF SHRINKING OR SWELLING SOILS


0.08
0.10
0.12
0.14
0.16
0.22
0.24
0.26
0.28
0.30
0.12
0.14
TYPICAL CONTOURS FOR RESIDENTIAL SLAB EXPERIENCING CENTER LIFT OR EDGE DRYING CONDITIONS IN ARLINGTON, TEXAS (Tucker and Poor,


DESIGN ENVELOPES
Example
Soil Support
Pattern
Worst Soil Support
Patterns

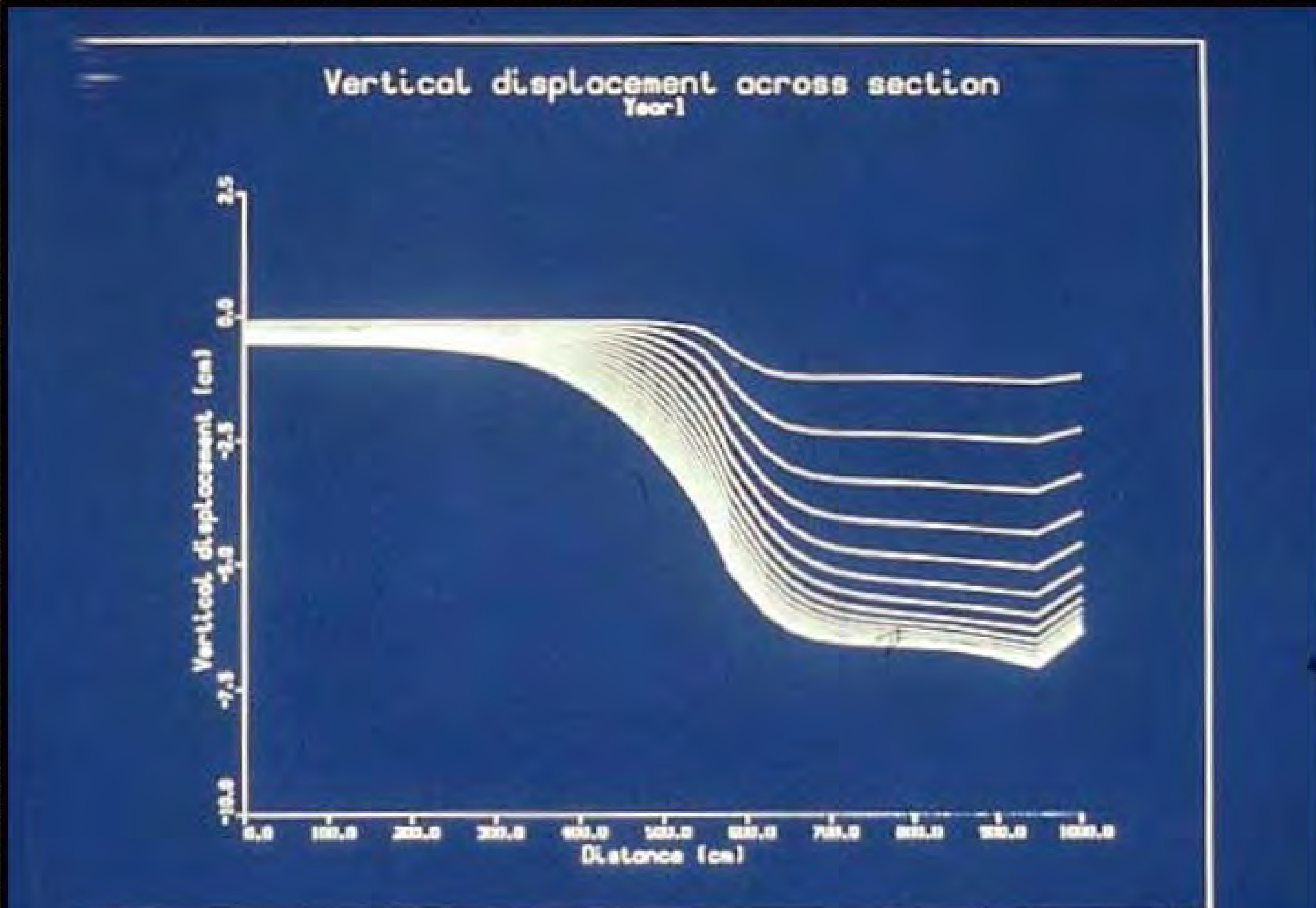

DESIGN OF SLABS-ON-GROUND
Maximim Differential Movement
y_m
Slab
e_m
e_m
Edge Moisture Variation Distance


Edge Moisture
Variation Distance
from Charts
Maximum
Differential
Movement
e_m
Slab
y_m
Depth of
Moisture
Active Zone
Z_0
Tree
Root
Zone
Contours of
Equal Suction
Steady State
Horizontal Flow

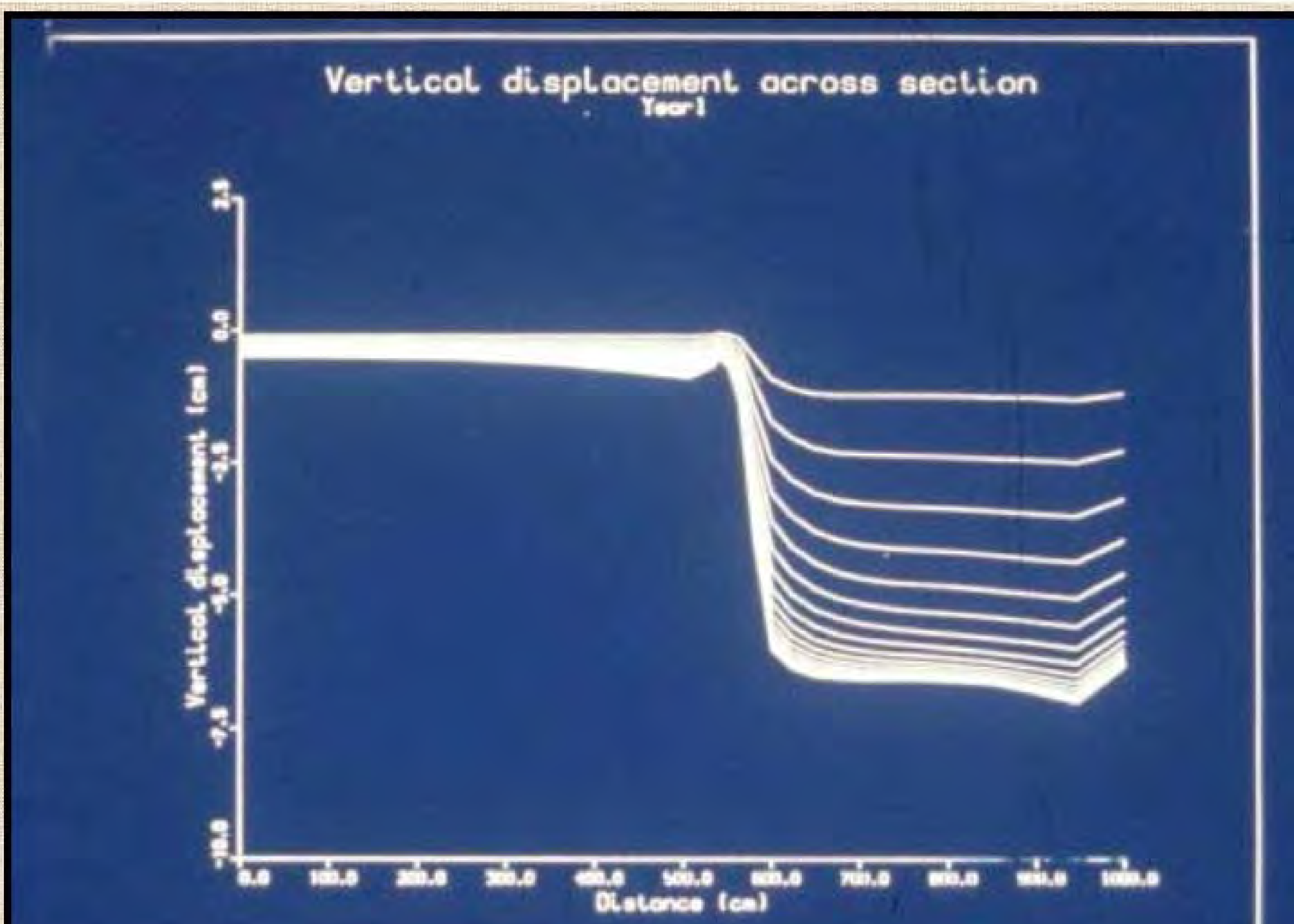

Edge Moisture
Variation Distance
from Charts
e_m
Maximum
Differential
Movement
y_m
Slab
Flower Bed
Depth of
Moisture
Active Zone
z_0
Contours of
Equal Suction
Steady State
Horizontal Flow

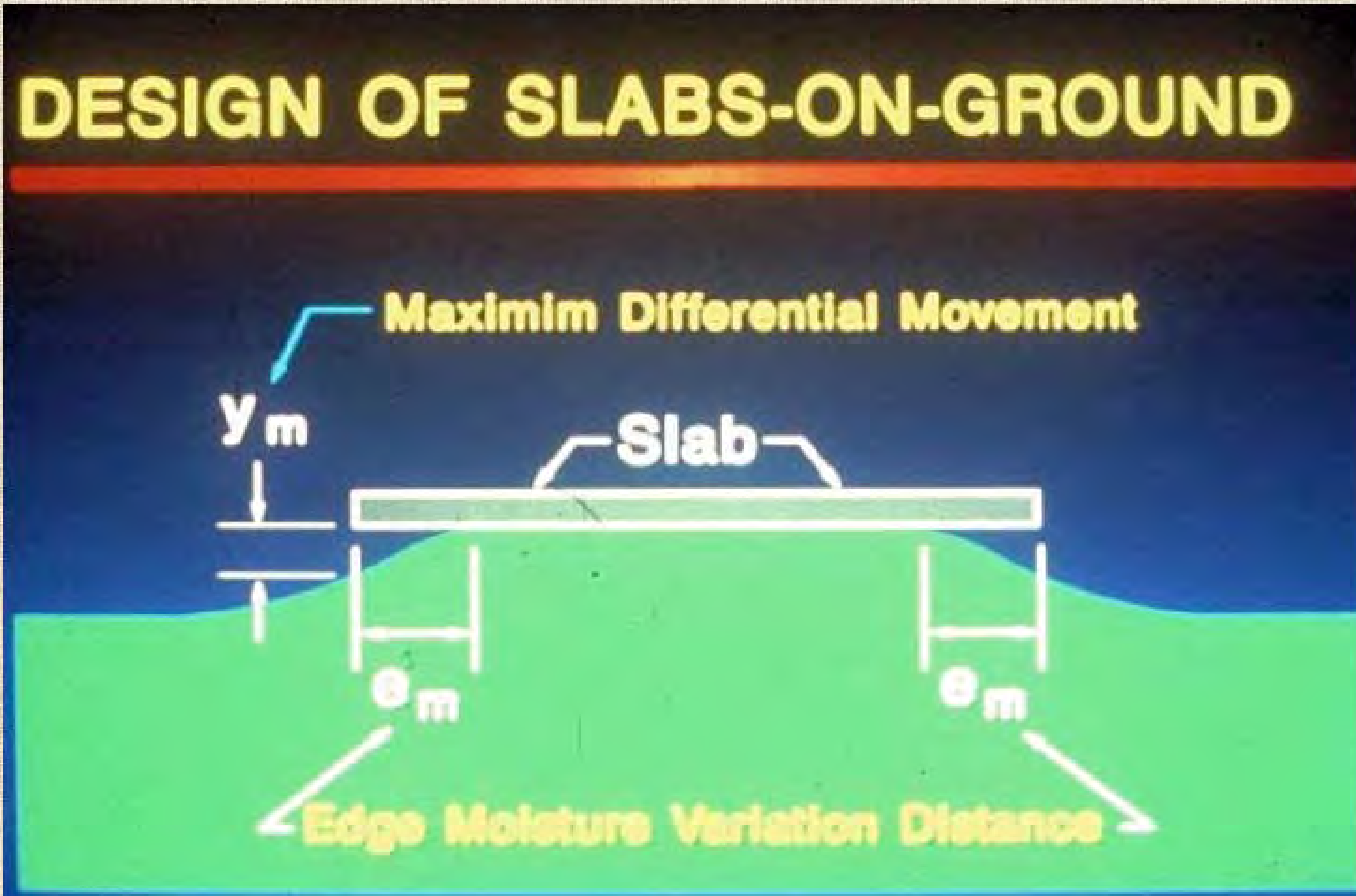
SITE CONDITIONS

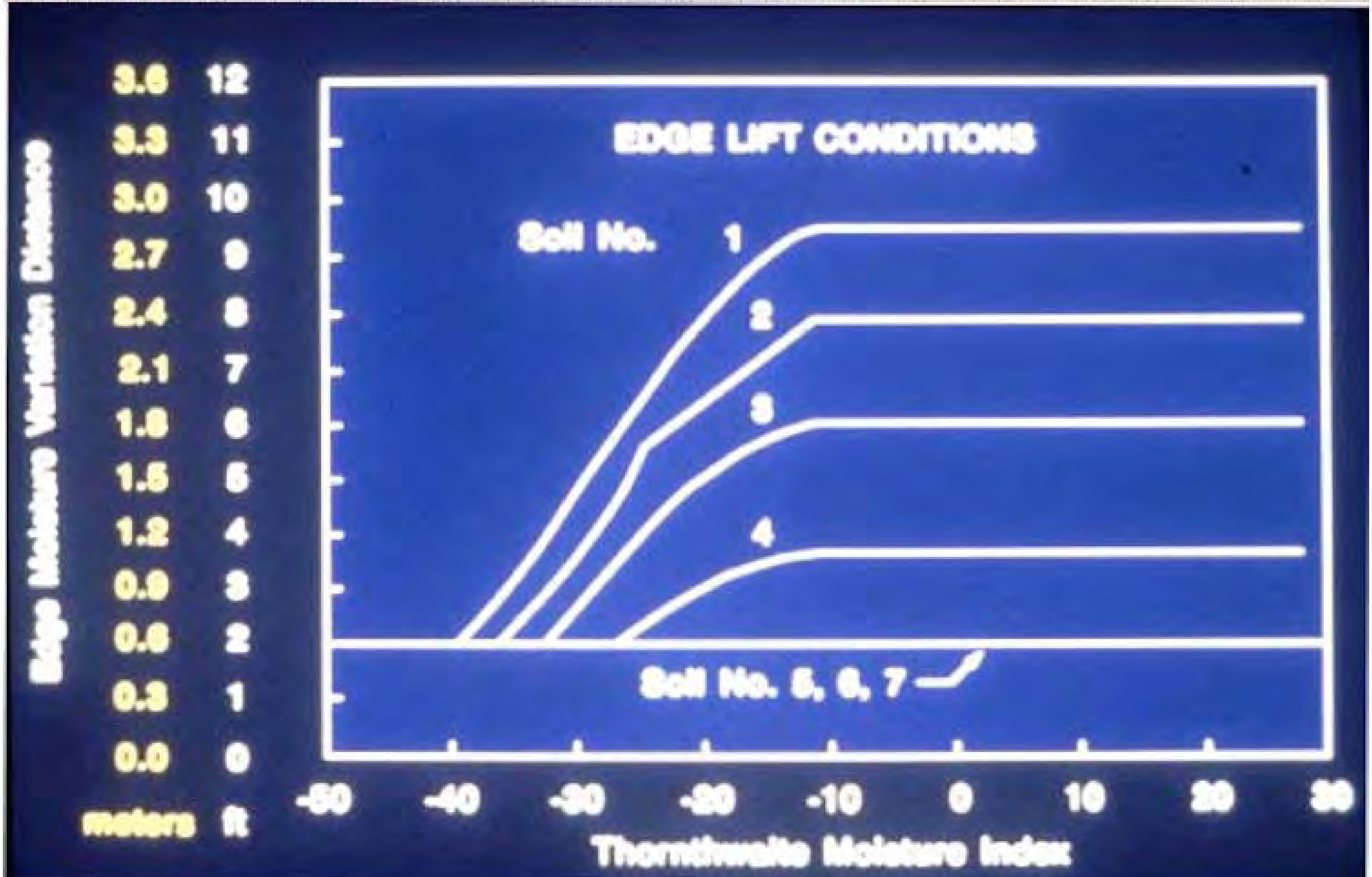
- Rainfall and evaporation
- Tree root zones
- Flower beds, ponds
- Vertical, horizontal barriers

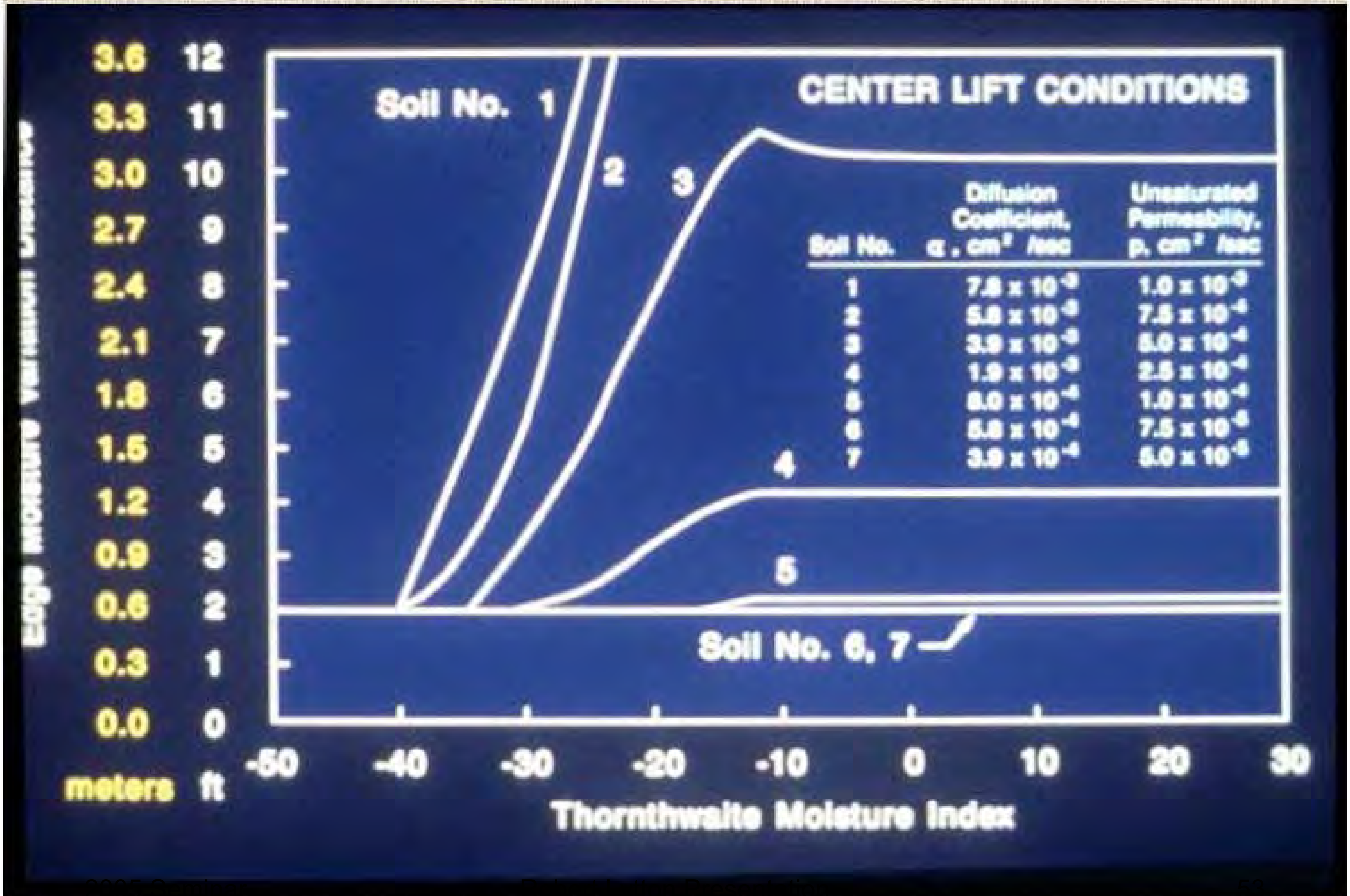


Vertical displacement across section
Year 1
Vertical displacement (cm)
2.5
0.0
-2.5
-5.0
-7.5
-10.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
800.0
900.0
1000.0
Distance (cm)


Vertical displacement across section
Year 1
Vertical displacement (cm)
Distance (cm)


DESIGN OF SLABS-ON-GROUND
Maximim Differential Movement
y_m
Slab
e_m
e_m
Edge Moisture Variation Distance


EDGE LIFT CONDITIONS
Soil No.
1
2
3
4
Soil No. 5, 6, 7
Edge Moisture Variation Distance
3.6 12
3.3 11
3.0 10
2.7 9
2.4 8
2.1 7
1.8 6
1.5 5
1.2 4
0.9 3
0.6 2
0.3 1
0.0 0
meters ft
-50 -40 -30 -20 -10 0 10 20 30
Thornthwaite Moisture Index


CENTER LIFT CONDITIONS
Soil No. 1
2
3
4
5
Soil No. 6, 7
Soil No. | Diffusion Coefficient, α, cm² /sec | Unsaturated Permeability, p, cm² /sec
1 | 7.8 x 10⁻³ | 1.0 x 10⁻³
2 | 5.8 x 10⁻³ | 7.5 x 10⁻⁴
3 | 3.9 x 10⁻³ | 5.0 x 10⁻⁴
4 | 1.9 x 10⁻³ | 2.5 x 10⁻⁴
5 | 8.0 x 10⁻⁴ | 1.0 x 10⁻⁴
6 | 5.8 x 10⁻⁴ | 7.5 x 10⁻⁵
7 | 3.9 x 10⁻⁴ | 5.0 x 10⁻⁵
Edge Moisture Variation Distance
3.6 12
3.3 11
3.0 10
2.7 9
2.4 8
2.1 7
1.8 6
1.5 5
1.2 4
0.9 3
0.6 2
0.3 1
0.0 0
meters ft
-50 -40 -30 -20 -10 0 10 20 30
Thornthwaite Moisture Index

EDGE MOISTURE DISTANCE

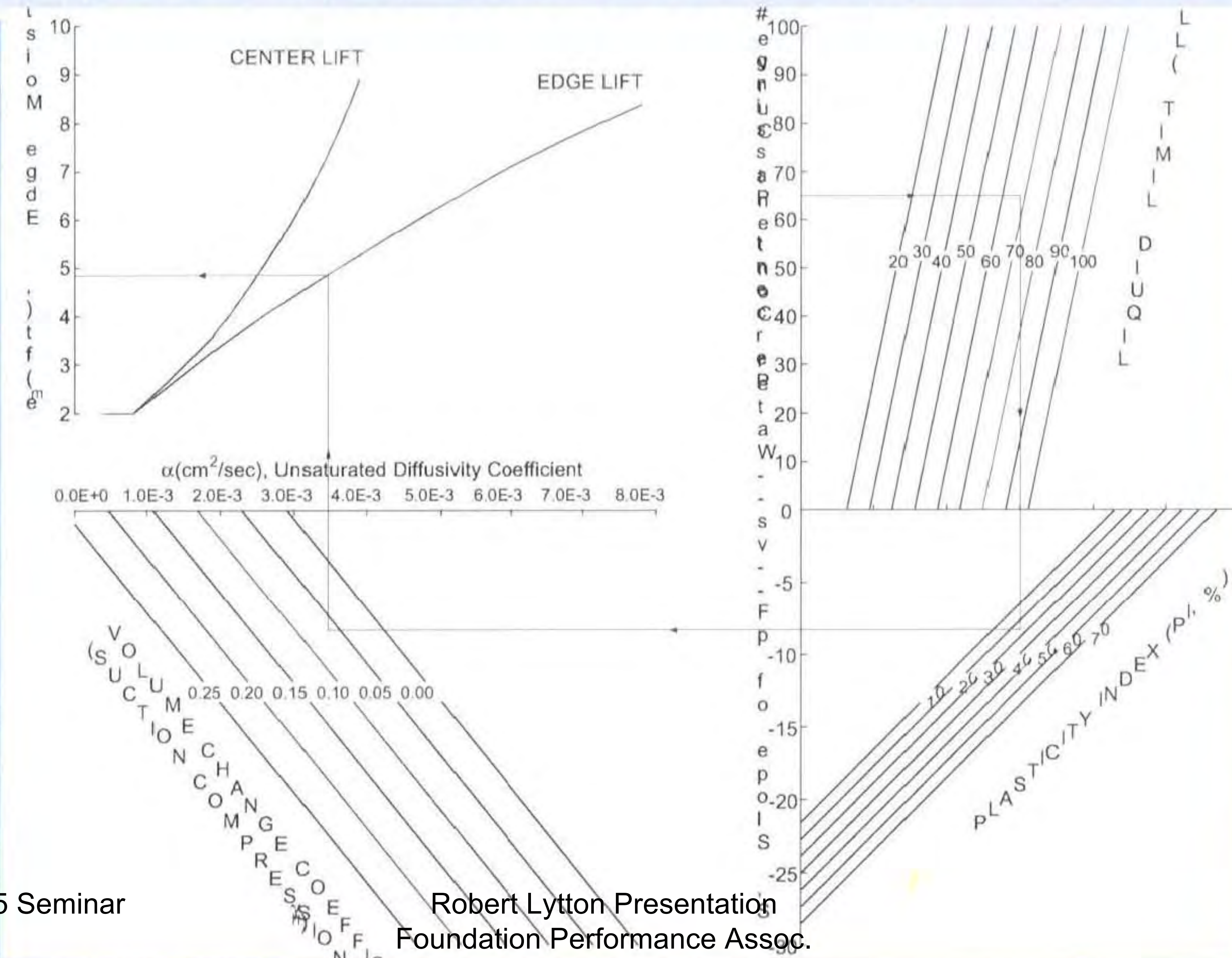
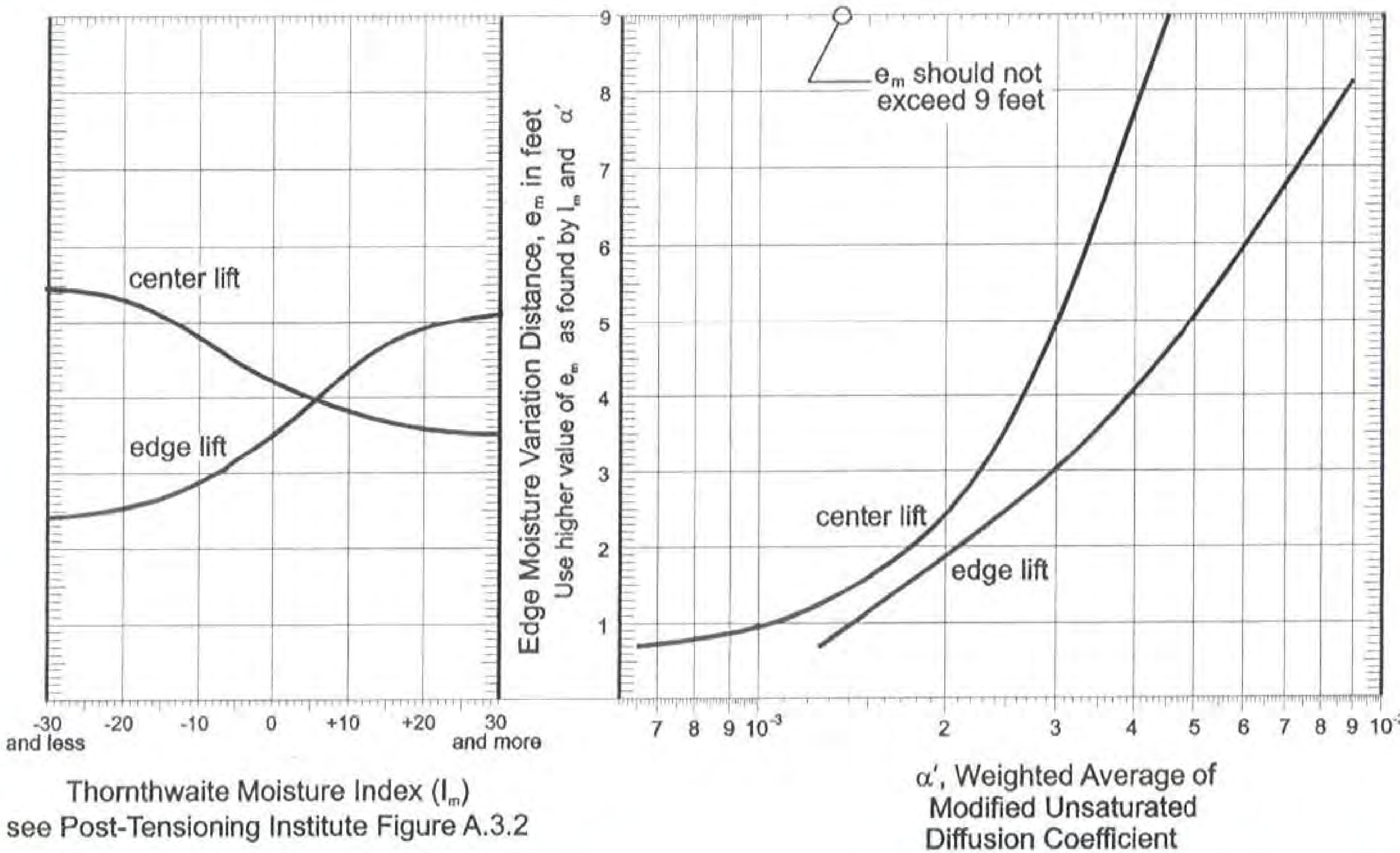


Figure 8 - e_m Selection Chart



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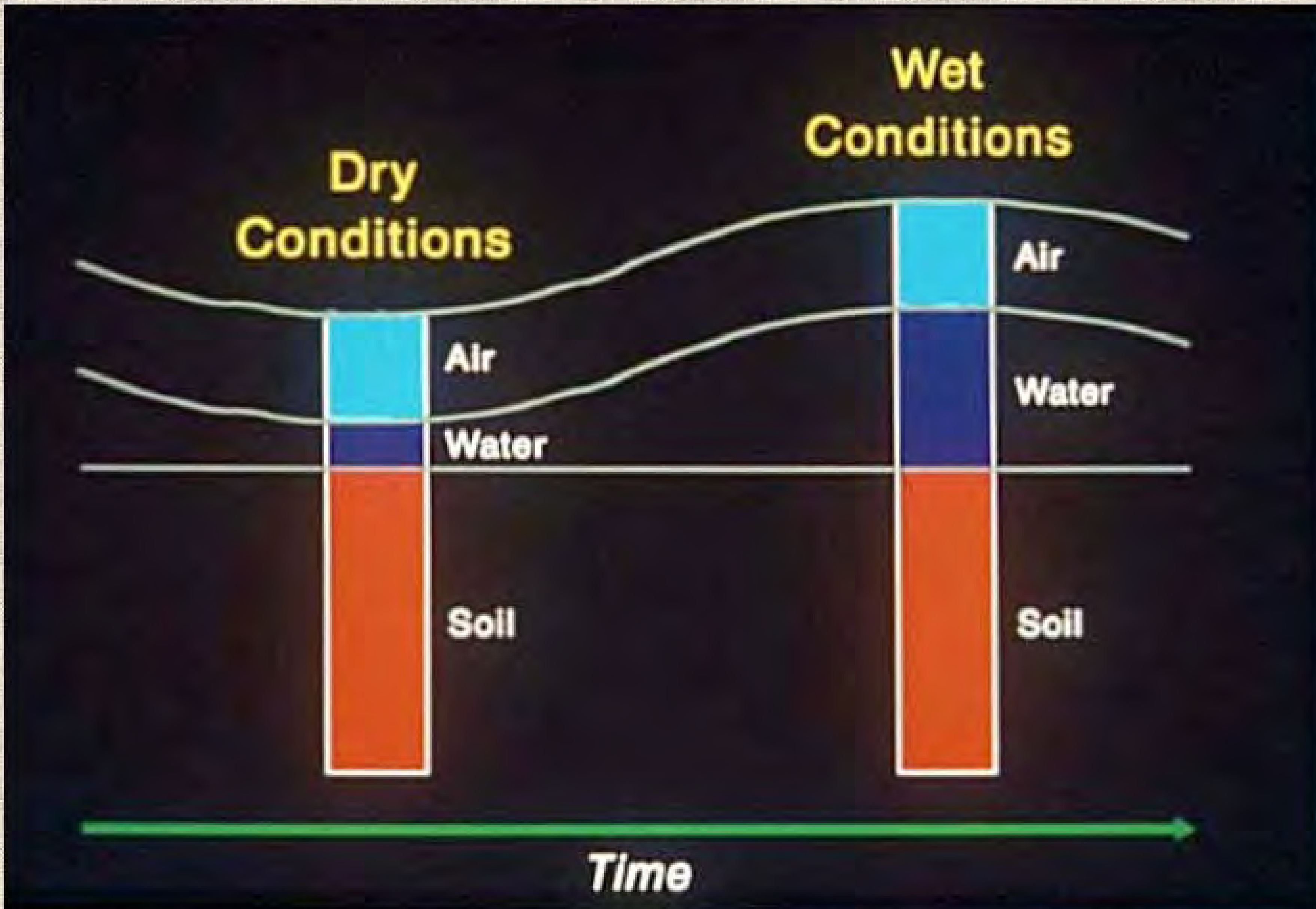
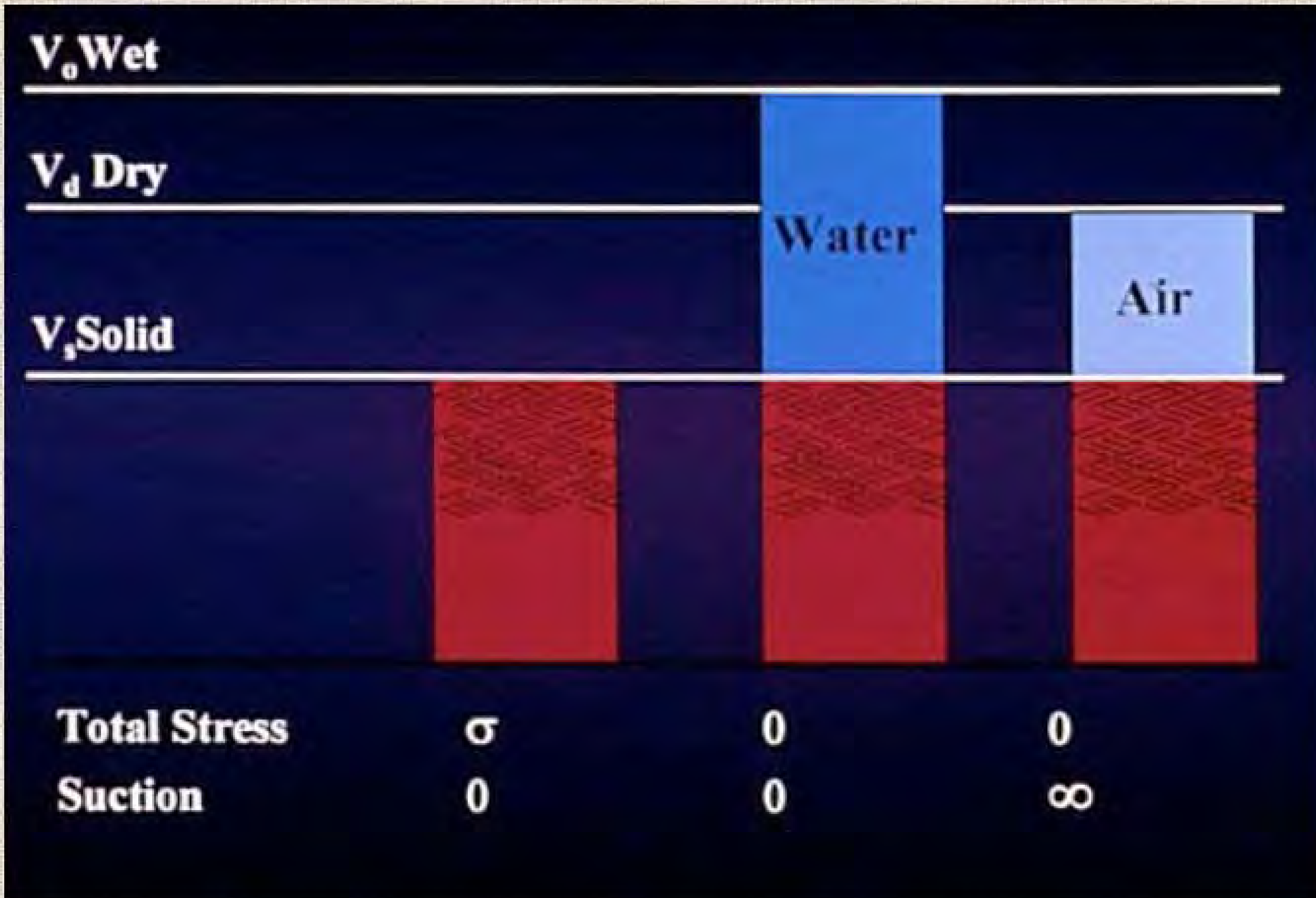
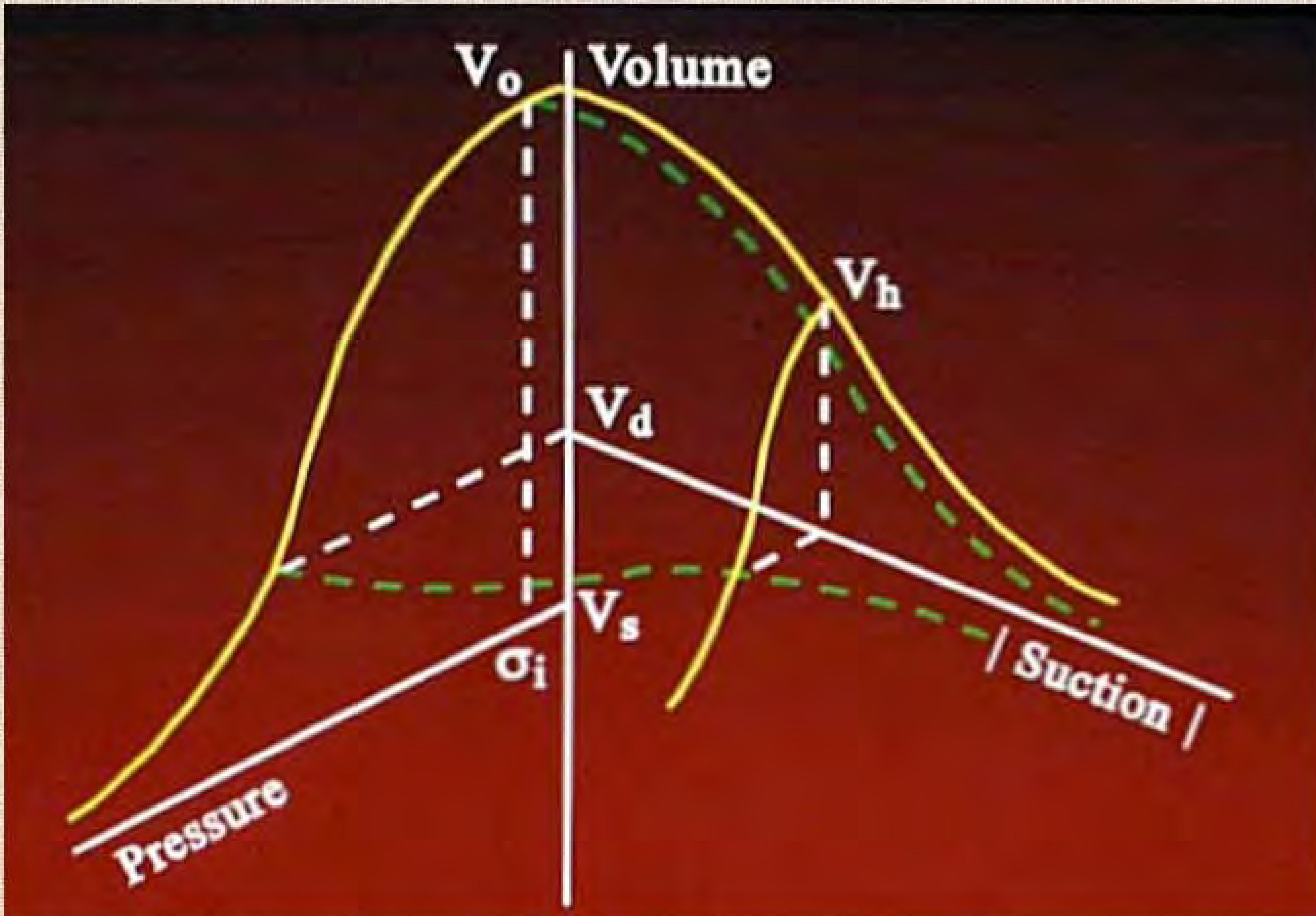
10:30 am	Break

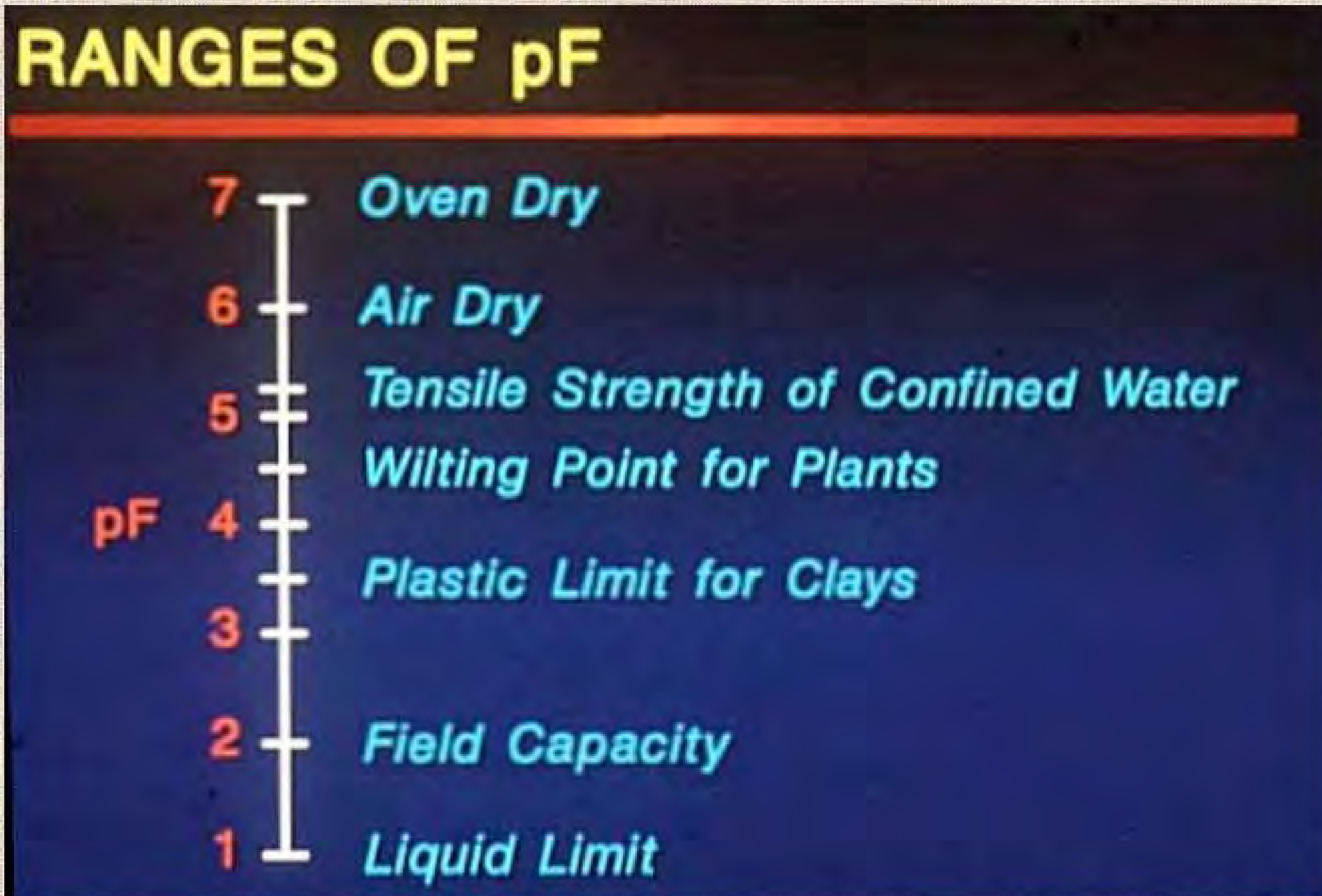

Dry
Conditions
Wet
Conditions
Air
Water
Soil
Air
Water
Soil
Time



V_o Wet
V_d Dry
V_s Solid
Water
Air
Total Stress
σ
0
0
Suction
0
0
∞



V_o
Volume
V_h
V_d
V_s
σ_i
| Suction |
Pressure

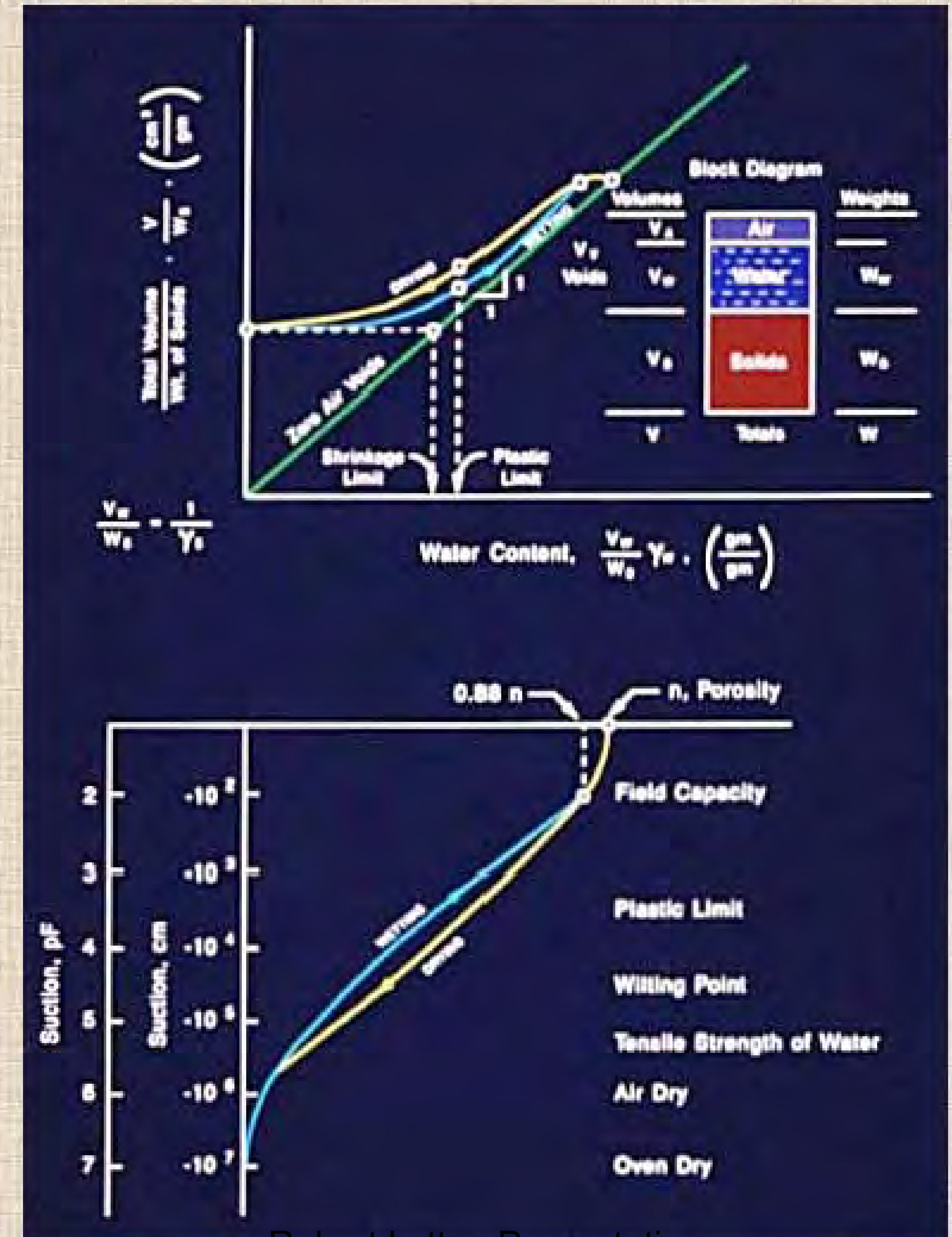

RANGES OF pF
pF
7
6
5
4
3
2
1
Oven Dry
Air Dry
Tensile Strength of Confined Water
Wilting Point for Plants
Plastic Limit for Clays
Field Capacity
Liquid Limit

p is the symbol for "logarithm"

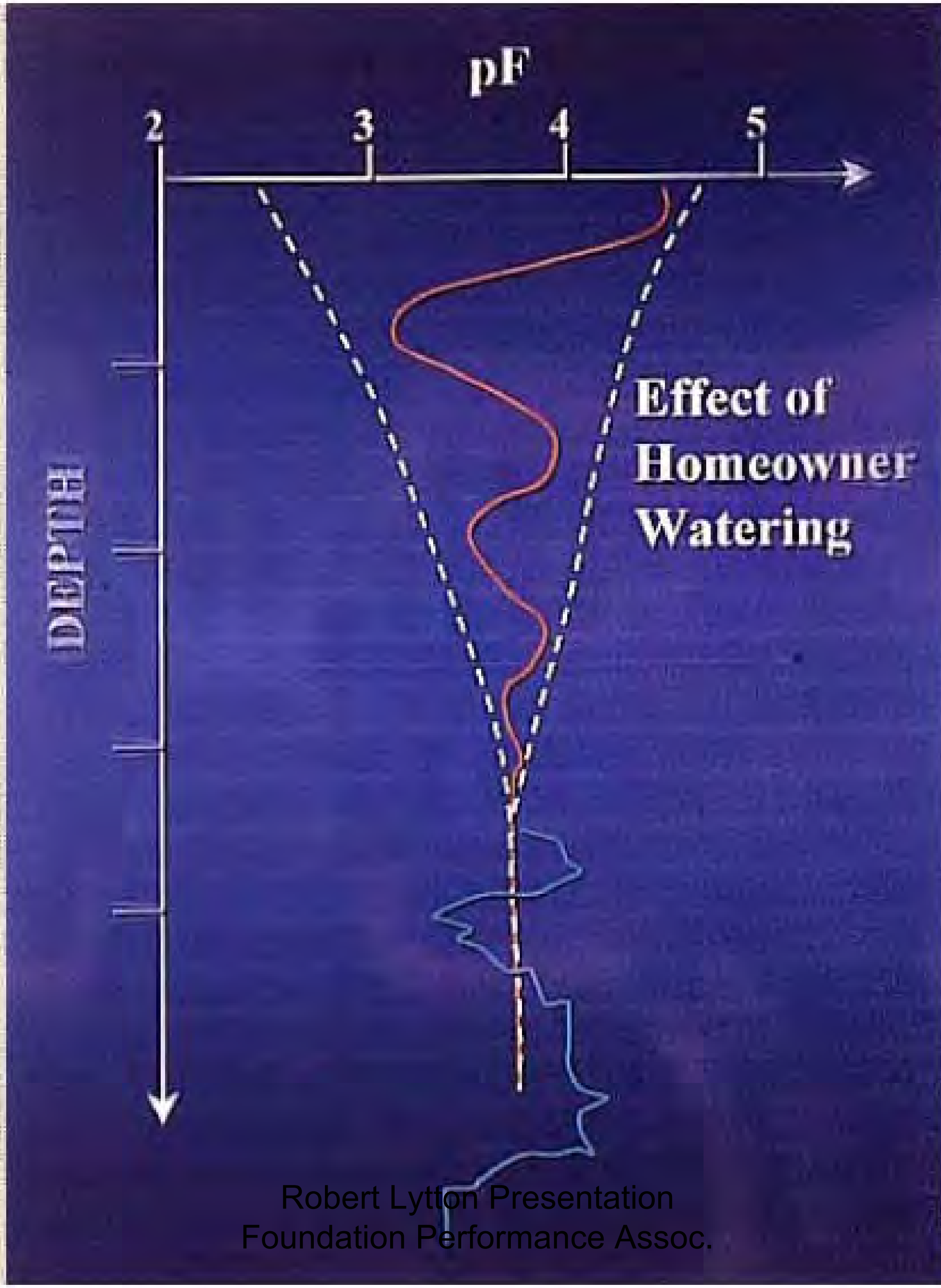
$$pF = \log_{10} |h|$$

F stands for Gibbs Free Energy, a thermodynamic quantity

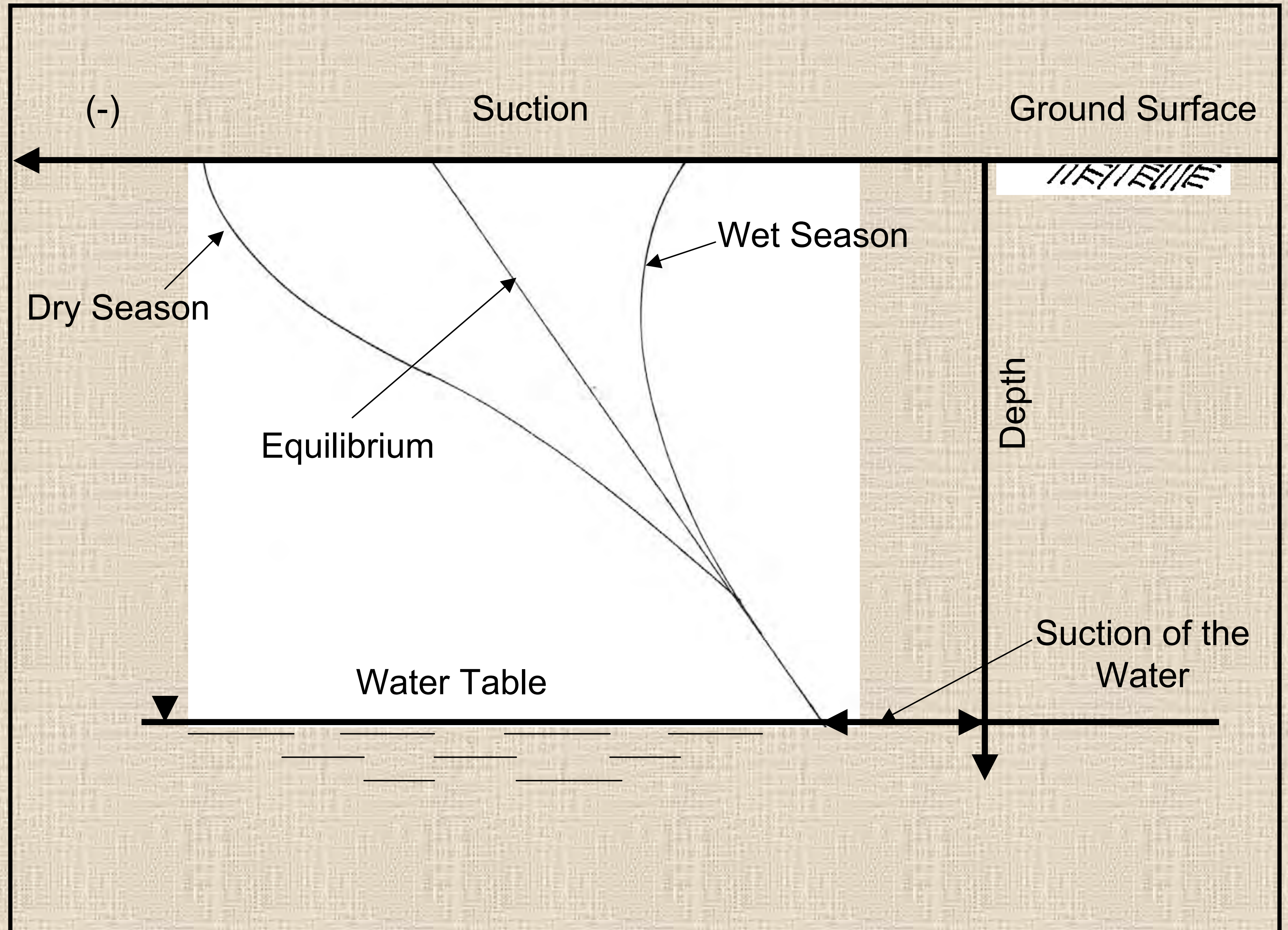
Suction in gm-cm/gm

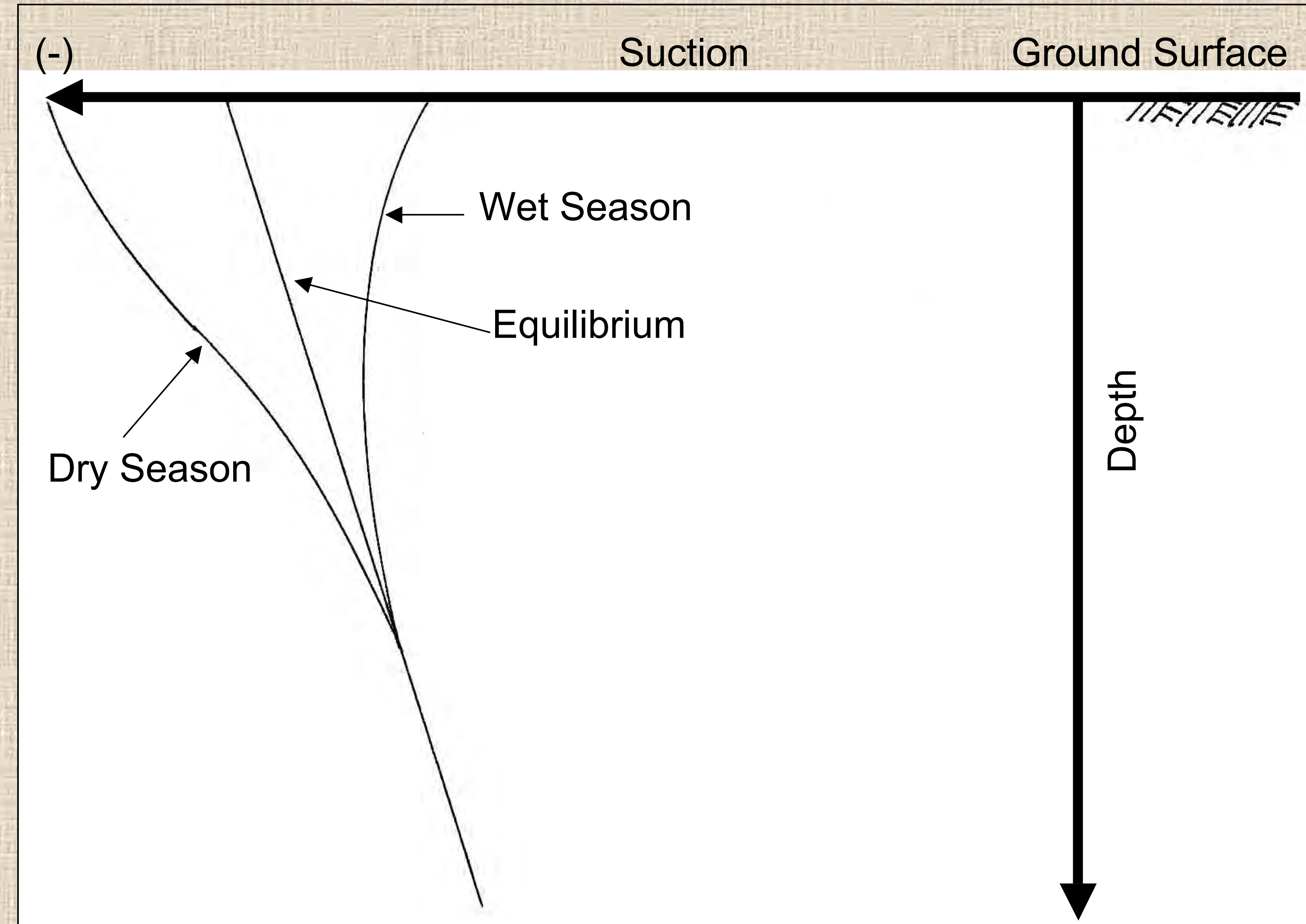

Block Diagram
Volumes
Weights
Air
Water
Solids
Voids
Totals
Zero Air Voids
Shrinkage Limit
Plastic Limit
Water Content,
0.88 n
n, Porosity
Field Capacity
Plastic Limit
Wilting Point
Tensile Strength of Water
Air Dry
Oven Dry
Suction, pF
Suction, cm
2
3
4
5
6
7
-10 2
-10 3
-10 4
-10 5
-10 6
-10 7

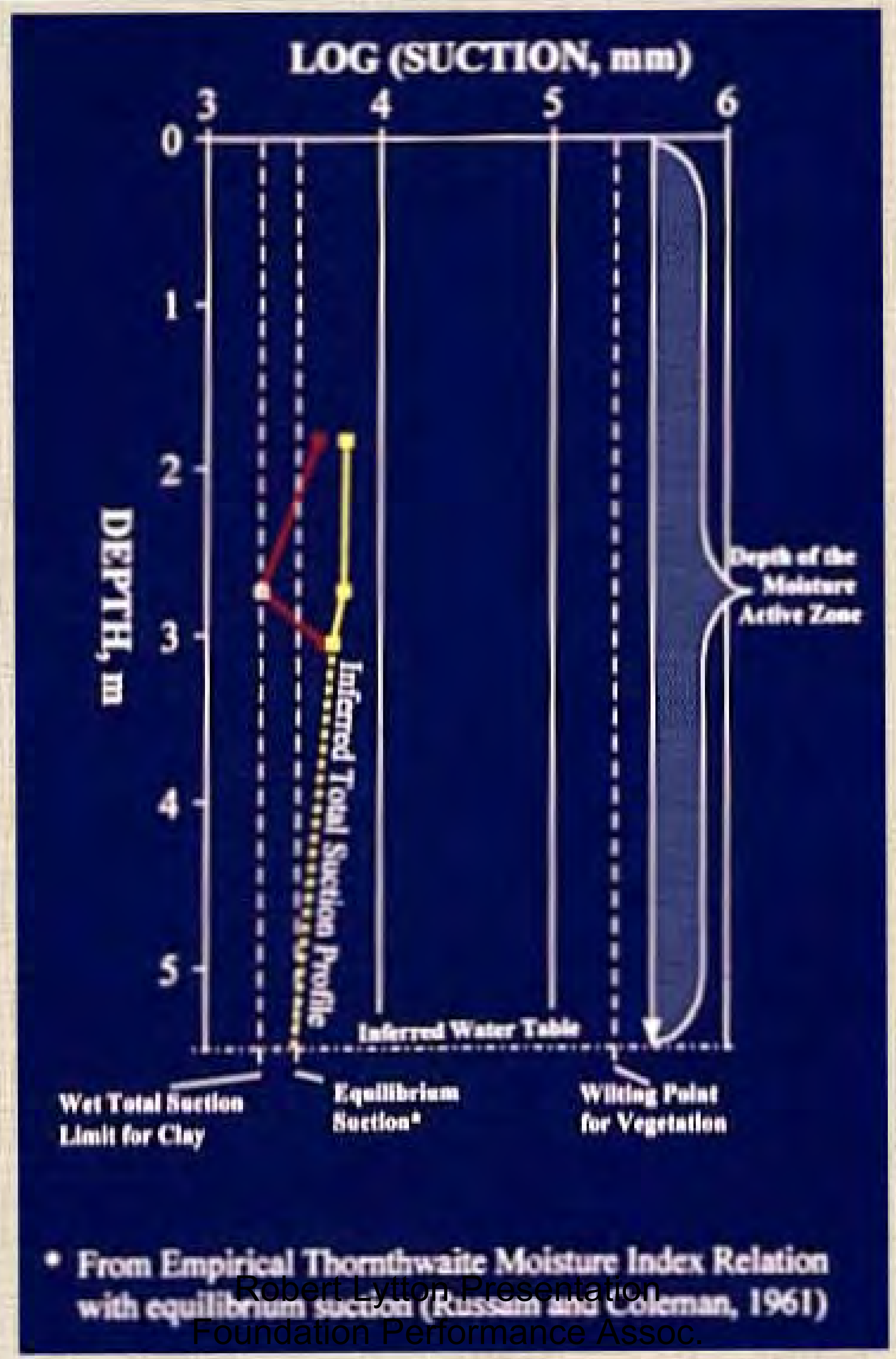

MOISTURE ACTIVE ZONE
MOISTURE INACTIVE ZONE

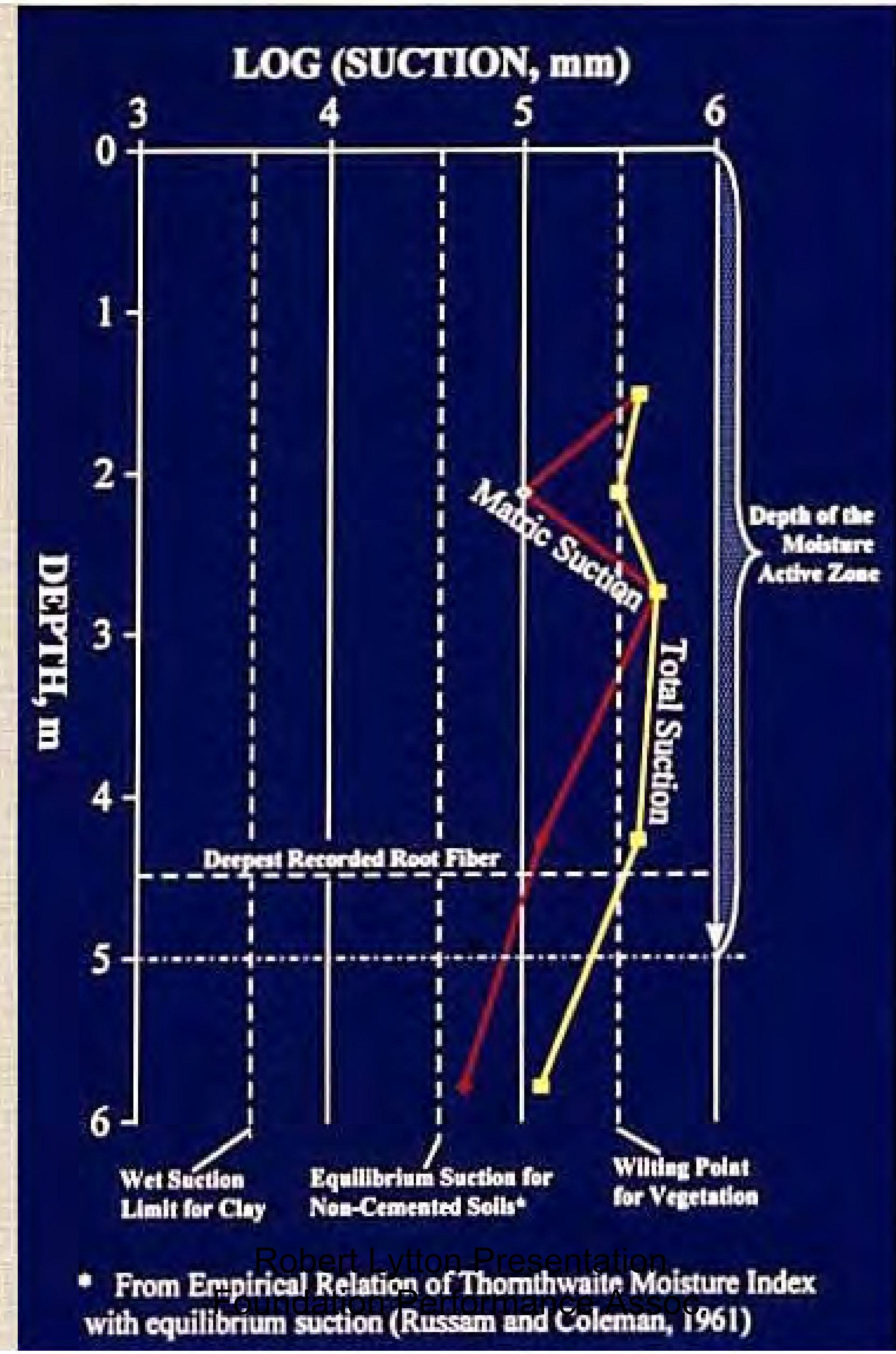


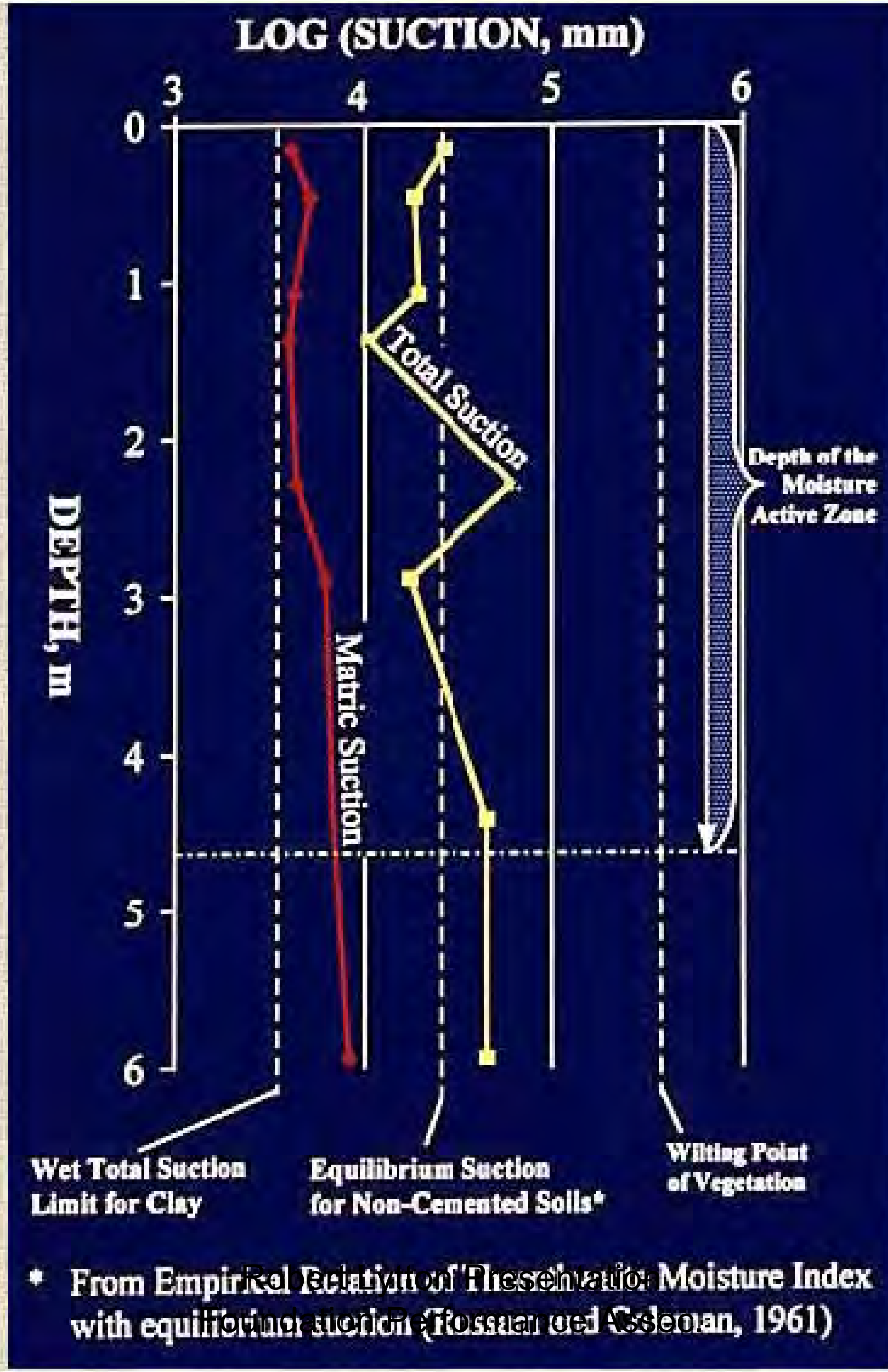
pF
2
3
4
5
DEPTH
Effect of Homeowner Watering

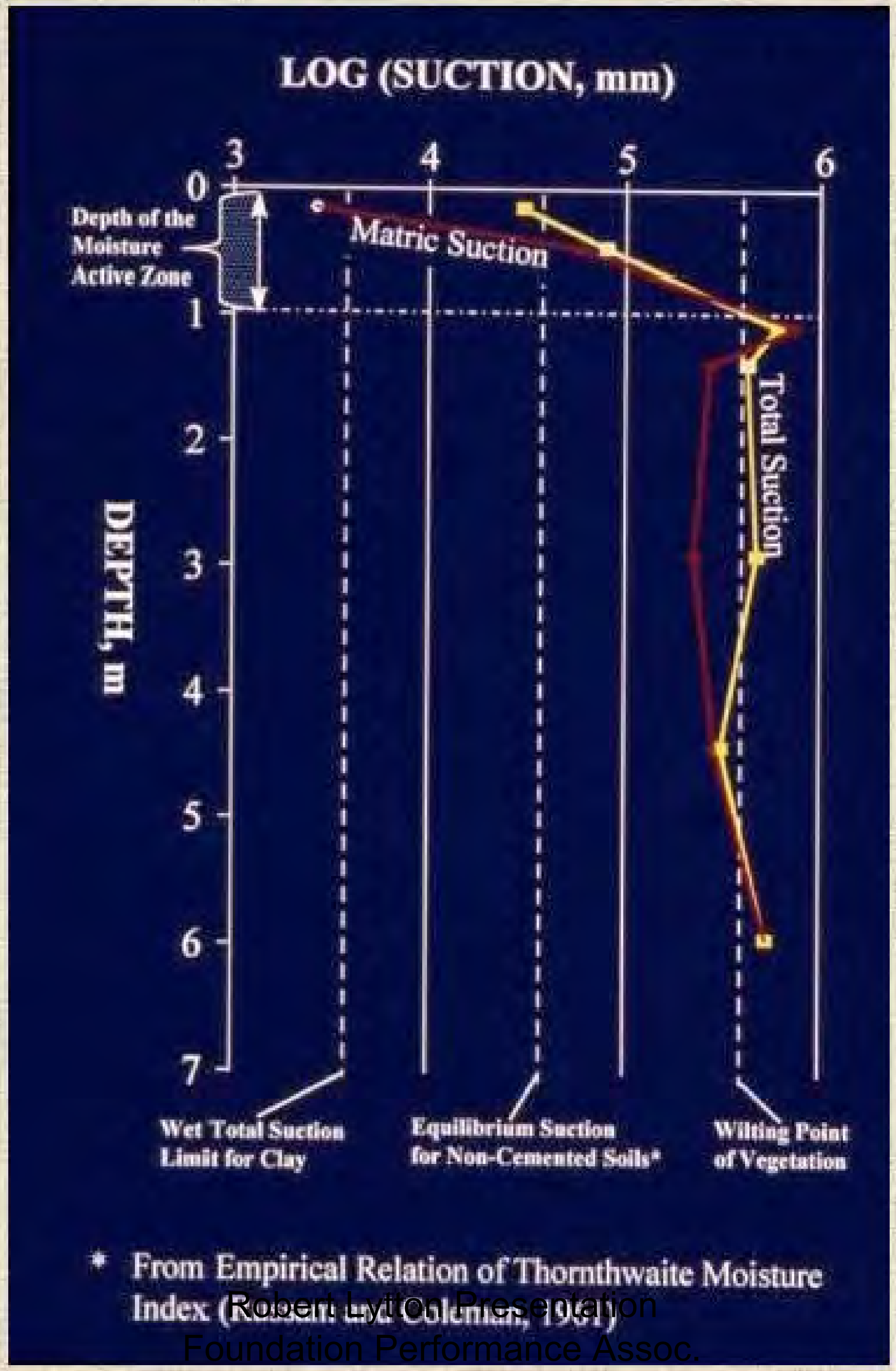

(-)
Suction
Ground Surface
Wet Season
Dry Season
Equilibrium
Depth
Suction of the Water
Water Table

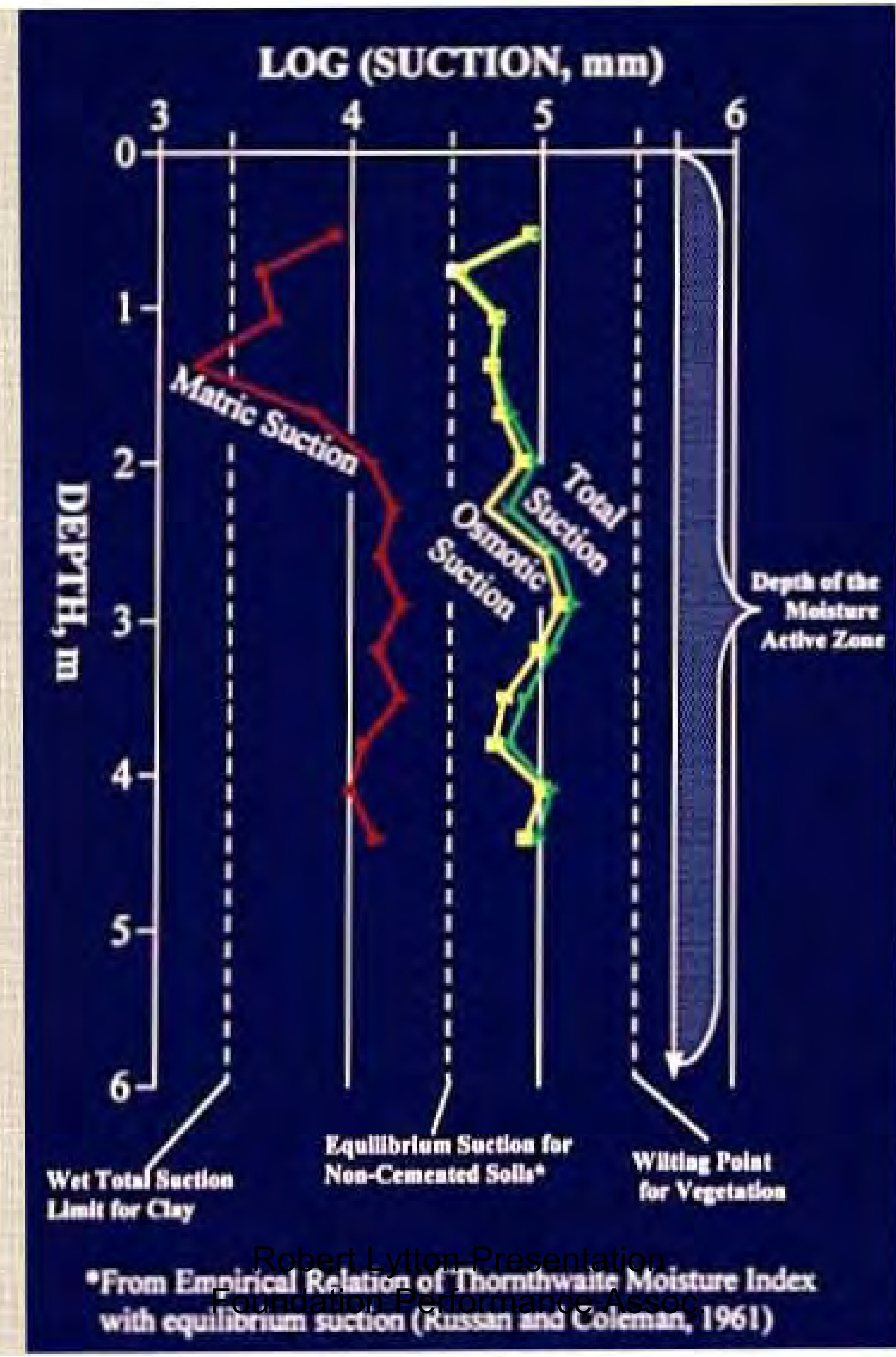

(-)
Suction
Ground Surface
Wet Season
Equilibrium
Depth
Dry Season


LOG (SUCTION, mm)
3
4
5
6
0
1
2
3
4
5
DEPTH, m
Inferred Total Suction Profile
Depth of the Moisture Active Zone
Inferred Water Table
Wet Total Suction Limit for Clay
Equilibrium Suction*
Wilting Point for Vegetation
* From Empirical Thornthwaite Moisture Index Relation with equilibrium suction (Russam and Coleman, 1961)


LOG (SUCTION, mm)
3
4
5
6
0
1
2
3
4
5
6
DEPTH, m
Matric Suction
Total Suction
Depth of the Moisture Active Zone
Deepest Recorded Root Fiber
Wet Suction Limit for Clay
Equilibrium Suction for Non-Cemented Soils*
Wilting Point for Vegetation
* From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)

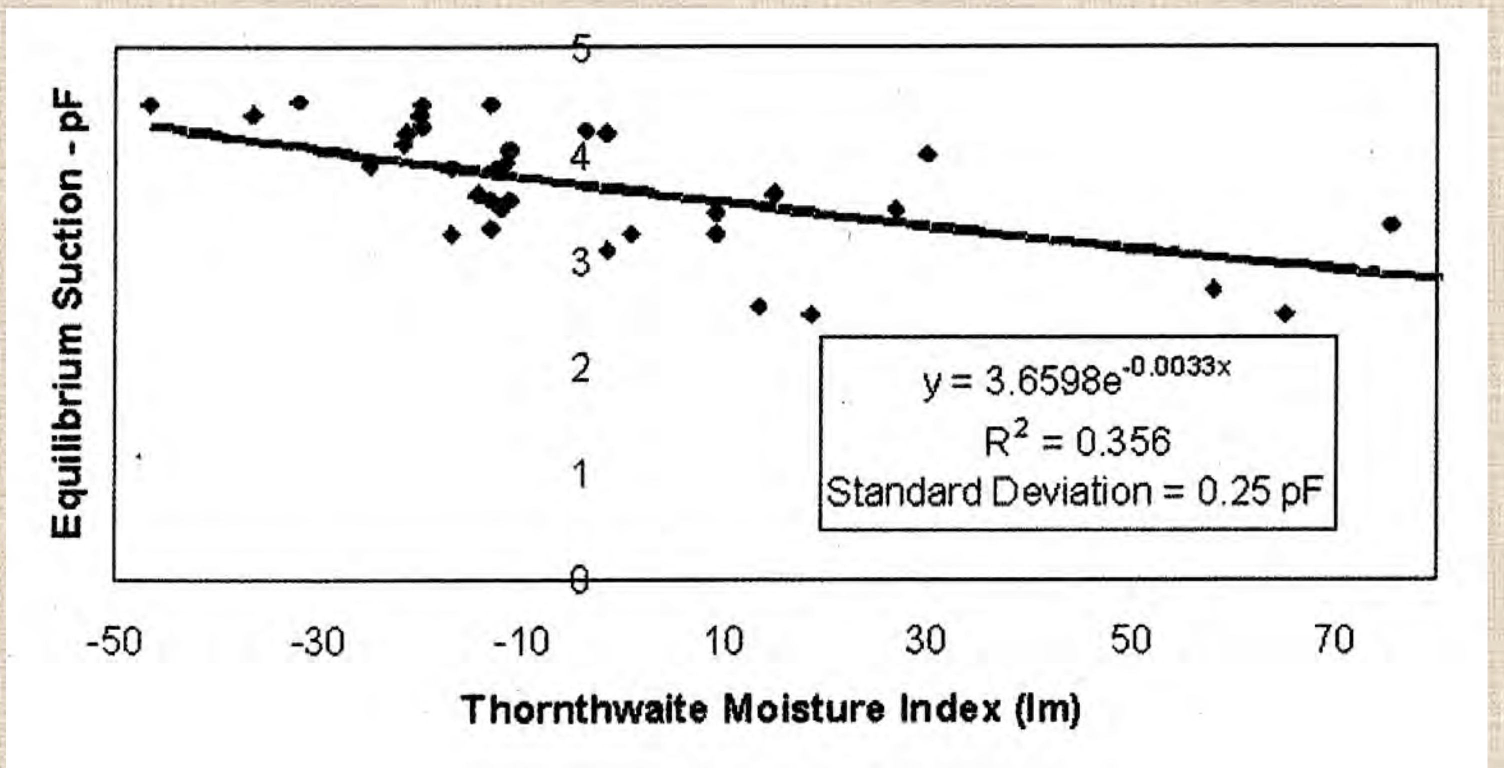

LOG (SUCTION, mm)
3
4
5
6
0
1
2
3
4
5
6
DEPTH, m
Total Suction
Matric Suction
Depth of the Moisture Active Zone
Wet Total Suction Limit for Clay
Equilibrium Suction for Non-Cemented Soils*
Wilting Point of Vegetation
* From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)


LOG (SUCTION, mm)
3
4
5
6
0
1
2
3
4
5
6
7
DEPTH, m
Depth of the Moisture Active Zone
Matric Suction
Total Suction
Wet Total Suction Limit for Clay
Equilibrium Suction for Non-Cemented Soils*
Wilting Point of Vegetation
* From Empirical Relation of Thornthwaite Moisture Index (Russam and Coleman, 1961)


LOG (SUCTION, mm)
3
4
5
6
0
1
2
3
4
5
6
DEPTH, m
Matric Suction
Total Suction
Osmotic Suction
Depth of the Moisture Active Zone
Wet Total Suction Limit for Clay
Equilibrium Suction for Non-Cemented Soils*
Wilting Point for Vegetation
*From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russan and Coleman, 1961)

PTI 3rd Edition Manual

Equilibrium Soil Suction vs. TMI

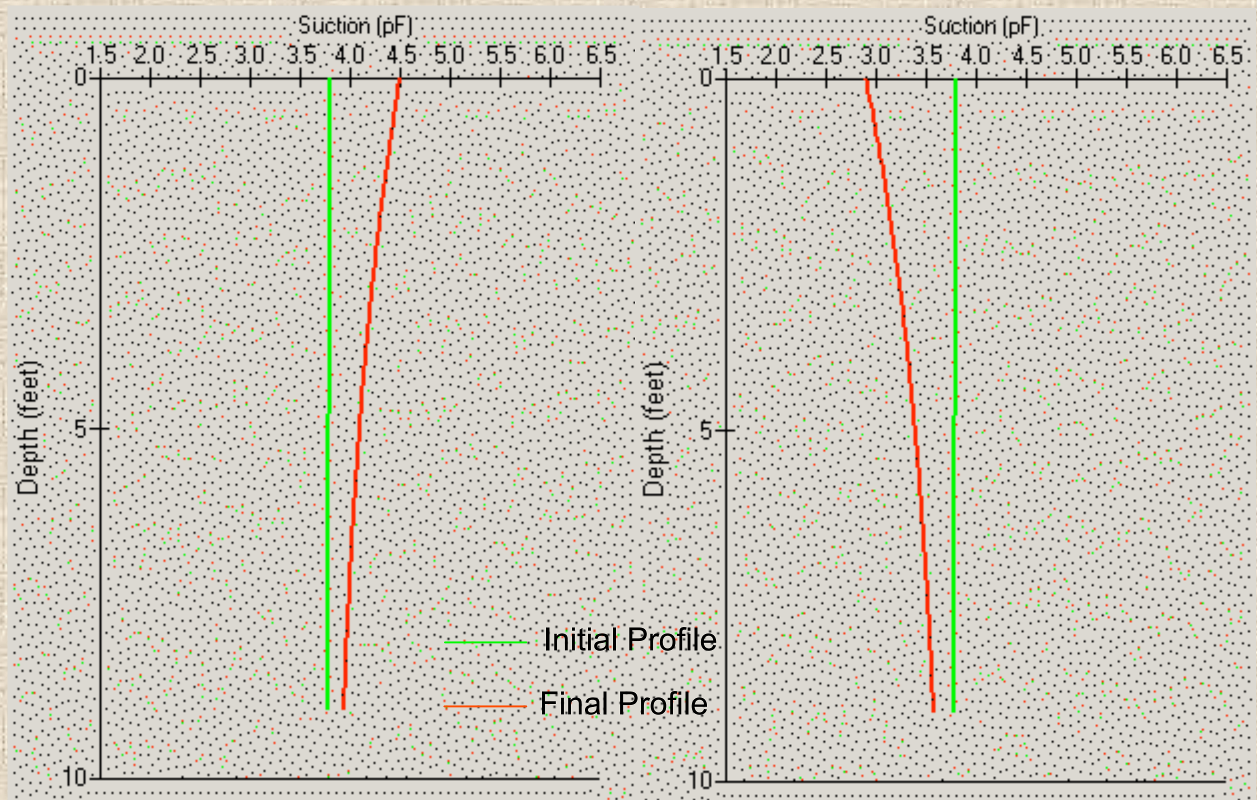


Note: Modified curve and equation of curve provided in 3rd Edition Manual.

PTI 3rd Edition Manual

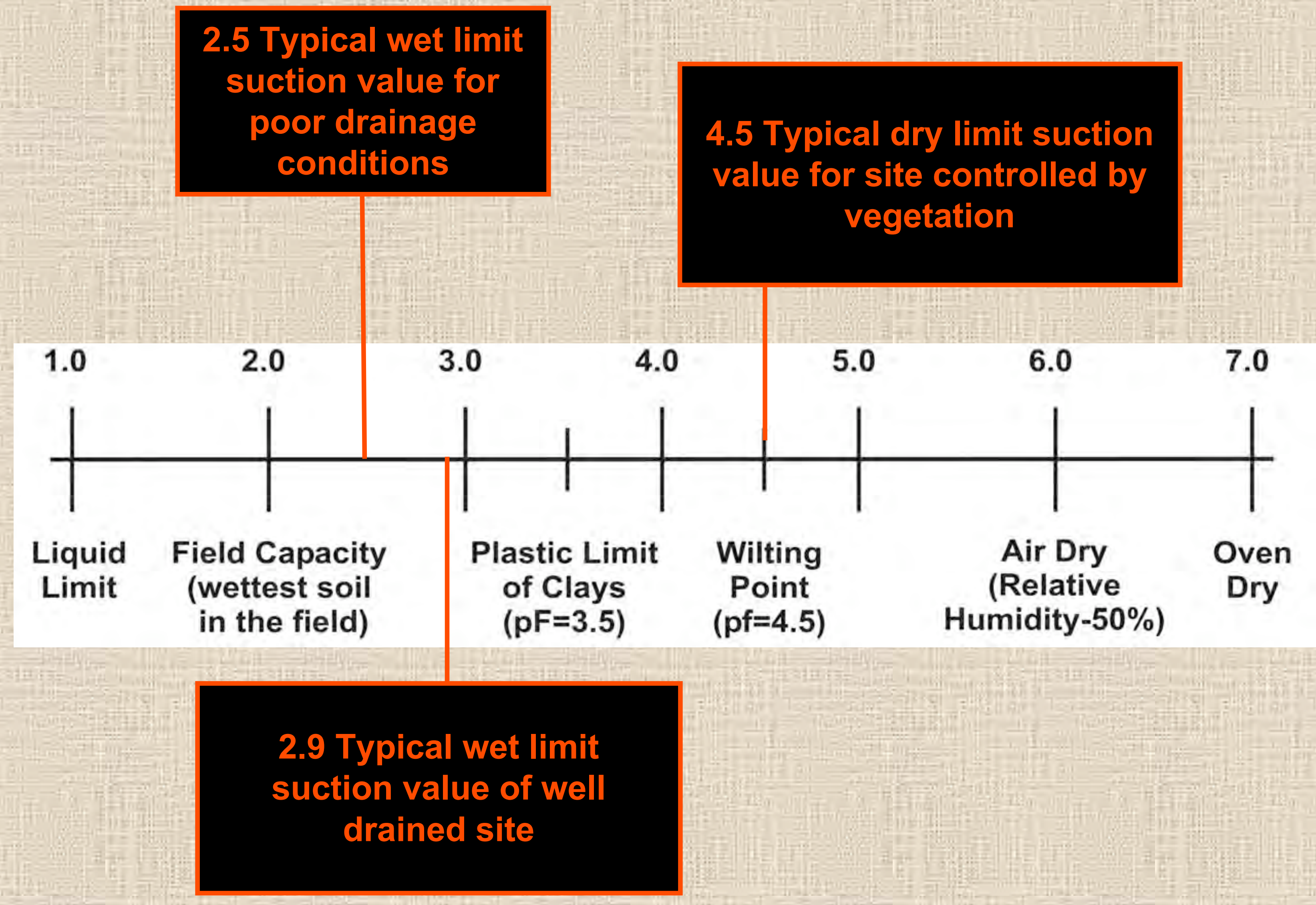
Soil Suction – SCF Assumptions

- Post Equilibirum - Initial suction set at equilibrium suction and changes to a wet or dry final suction profile.
- Final suction profile is assumed to be “trumpet” shape.



Shrinking **Swelling**

Typical Levels of Suction



LABORATORY TESTS

- Atterberg limits
- Hydrometer
- Water content
- Dry density
- Sieve analysis

VOLUME CHANGE RULE

$$\Delta V/V = -\gamma_h \log_{10}(h_f/h_i) - \gamma_\sigma \log_{10}(\sigma_f/\sigma_i) - \gamma_\pi \log_{10}(\pi_f/\pi_i)$$

$\Delta V/V$ = volume strain

h_i, h_f = initial and final matrix suction

σ_i, σ_f = initial and final mean principal stress

π_i, π_f = initial and final osmotic suction

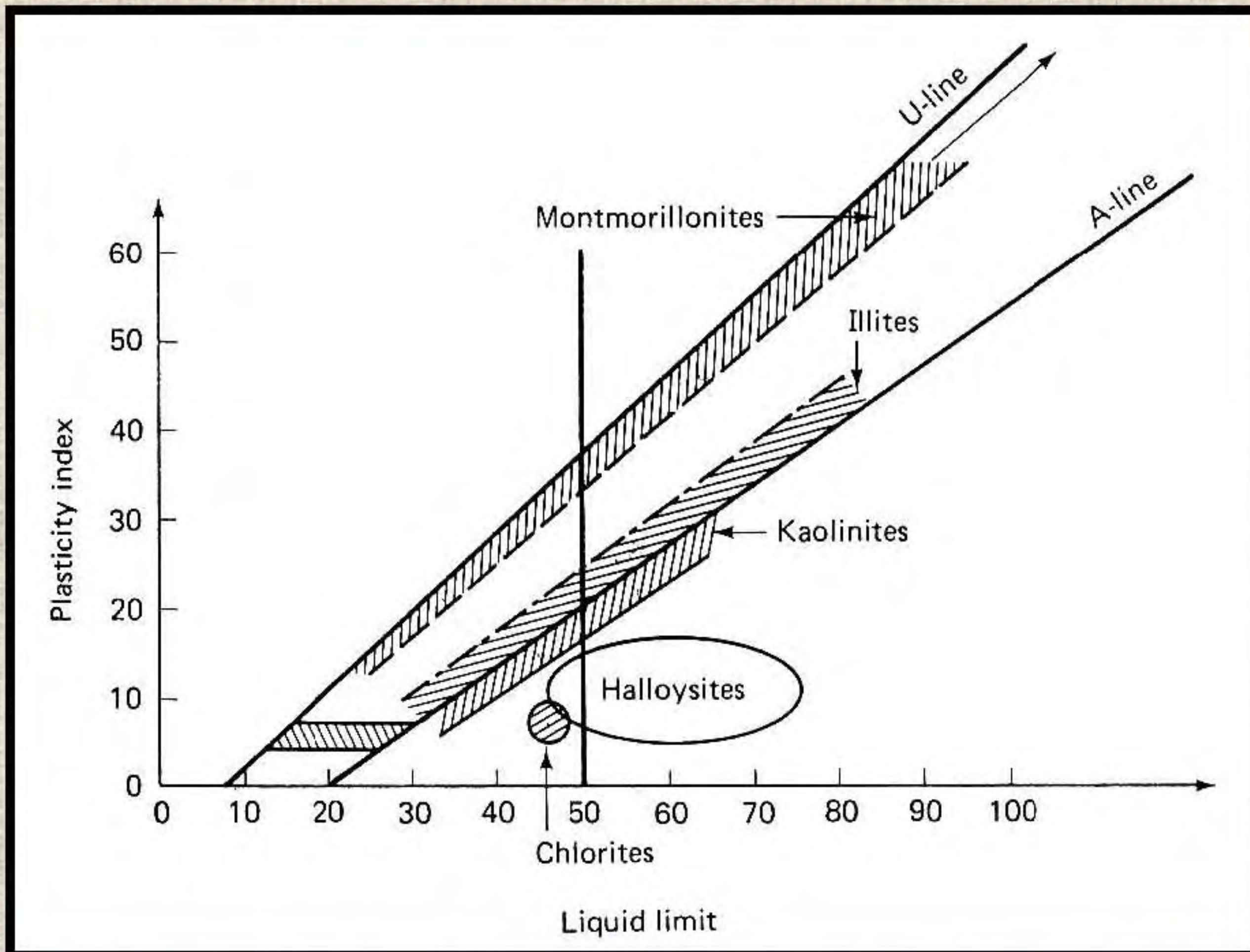
$\gamma_h, \gamma_\sigma, \gamma_\pi$ = compression indexes

SUCTION COMPRESSION INDEX

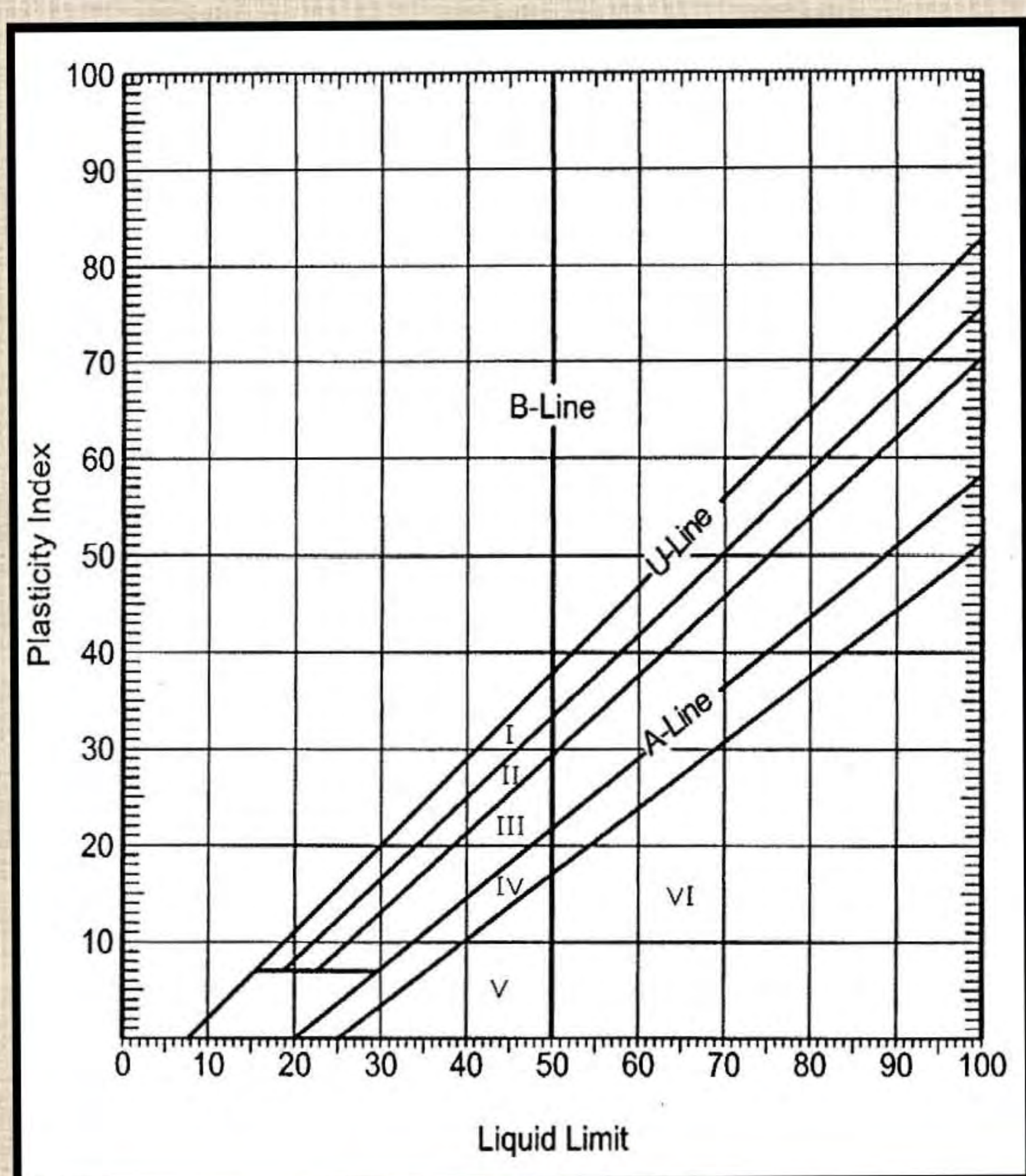
$$SCI = \frac{\dfrac{\Delta V}{V}}{\log_{10}\left(\dfrac{\text{Final Suction}}{\text{Initial Suction}}\right)}$$

SUCTION COMPRESSION INDEX

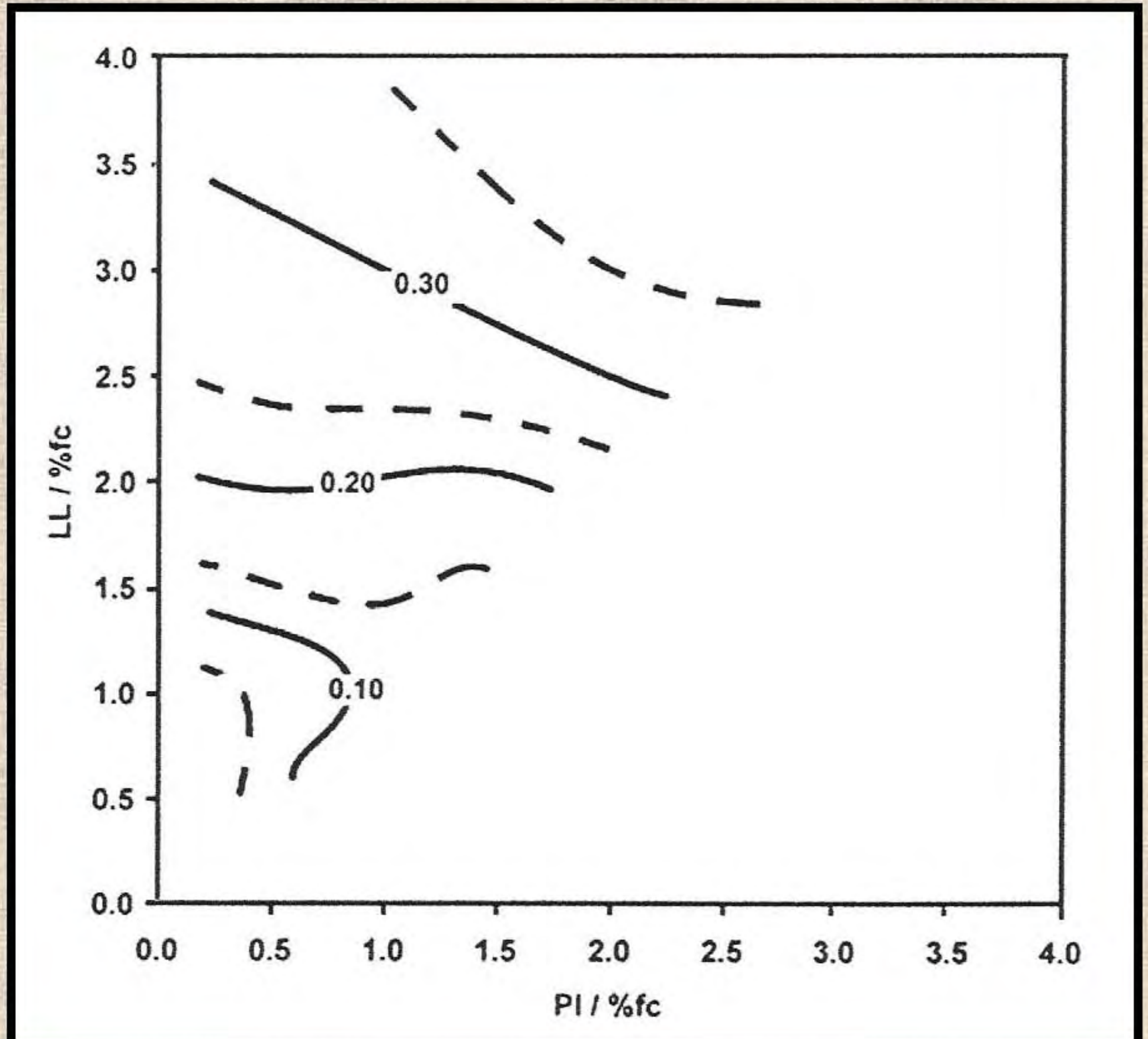
$$SCI = \frac{(\% - 2\mu)}{(\% - \#200)} \times \left(\begin{array}{c} \text{Guide Number} \\ \text{From Chart} \end{array} \right)$$



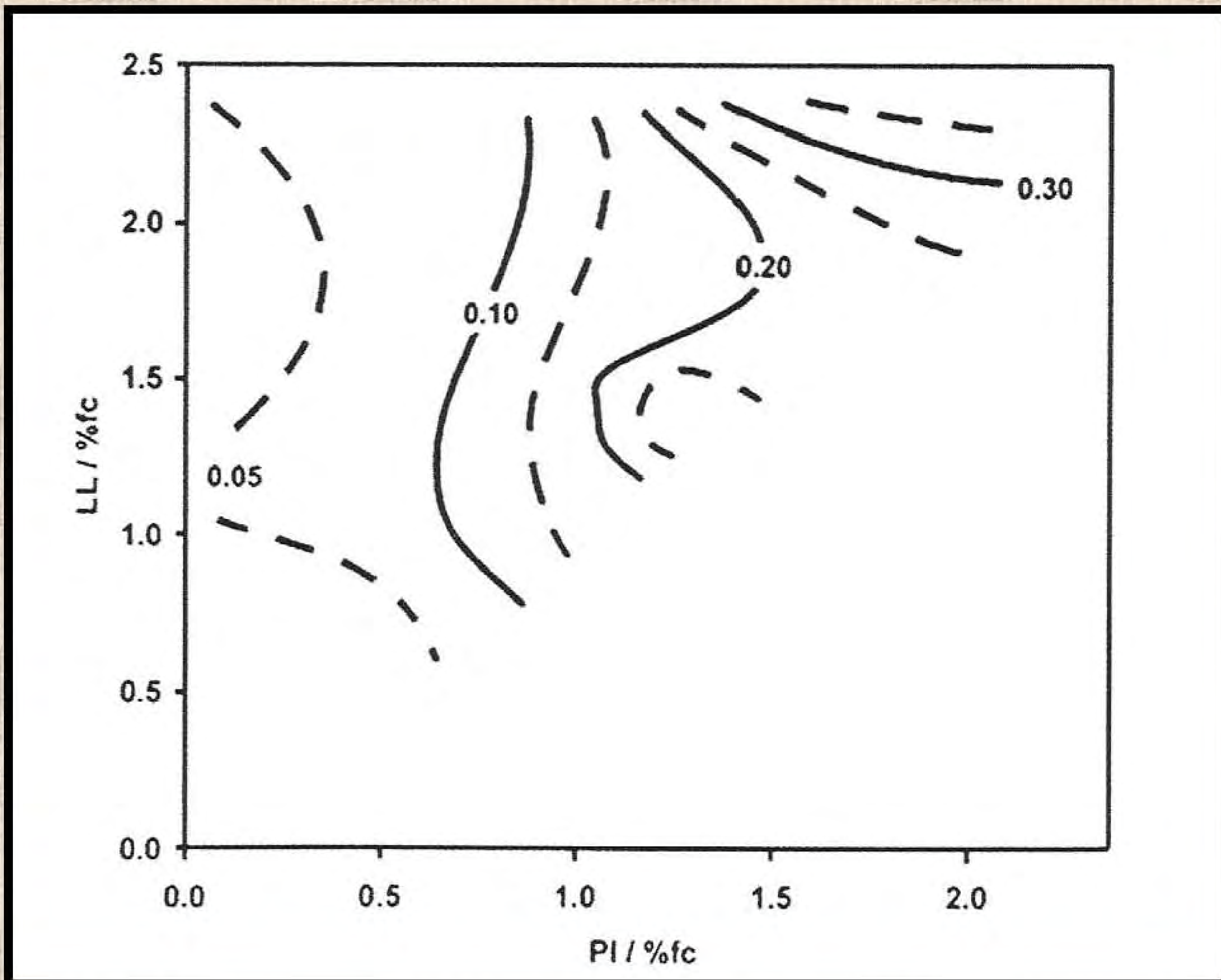
U-line
A-line
Montmorillonites
Illites
Kaolinites
Halloysites
Chlorites
Plasticity index
Liquid limit
0
10
20
30
40
50
60
70
80
90
100



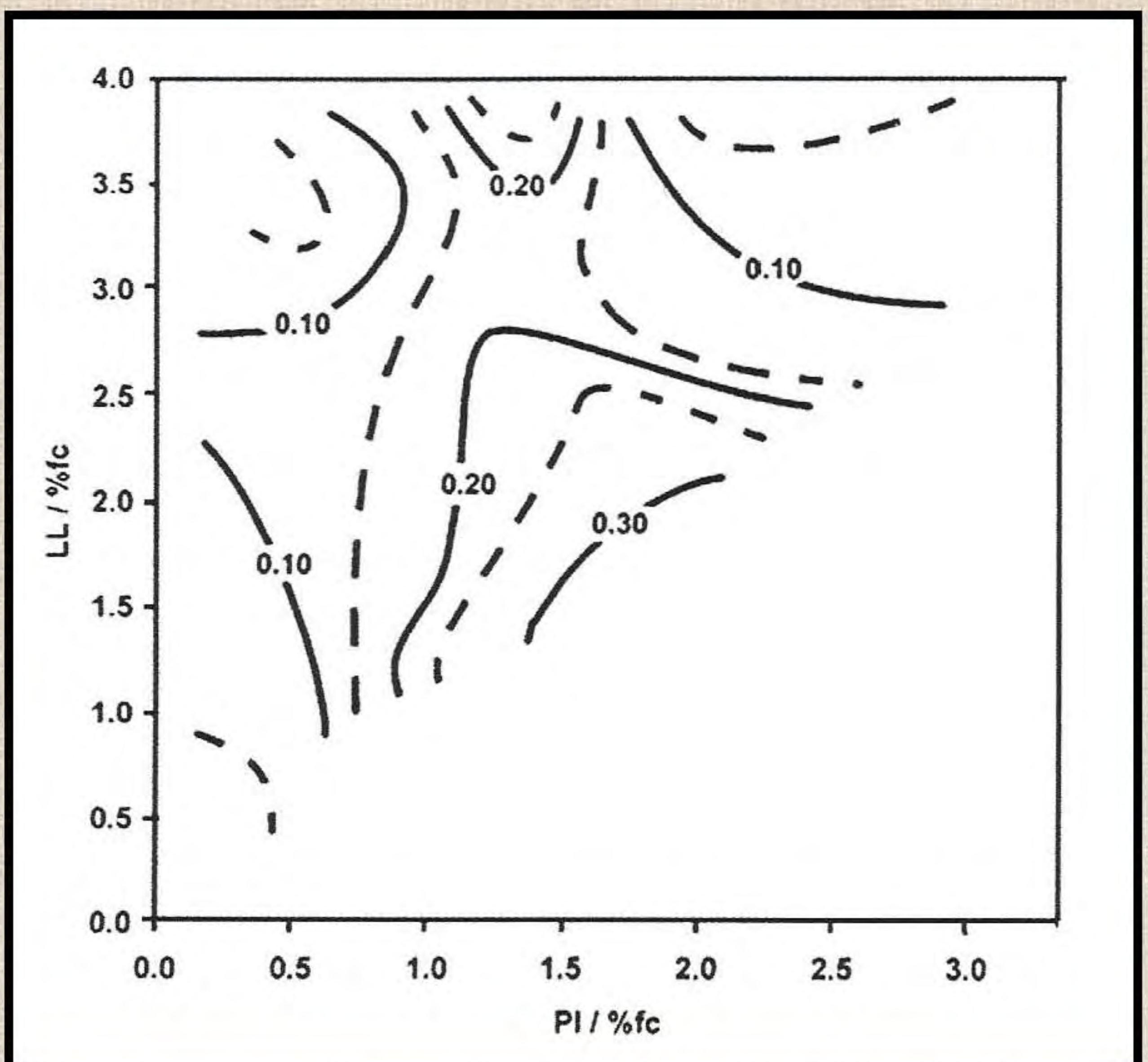
Plasticity Index
100
90
80
70
60
50
40
30
20
10
0
10
20
30
40
50
60
70
80
90
100
Liquid Limit
B-Line
U-Line
A-Line
I
II
III
IV
V
VI

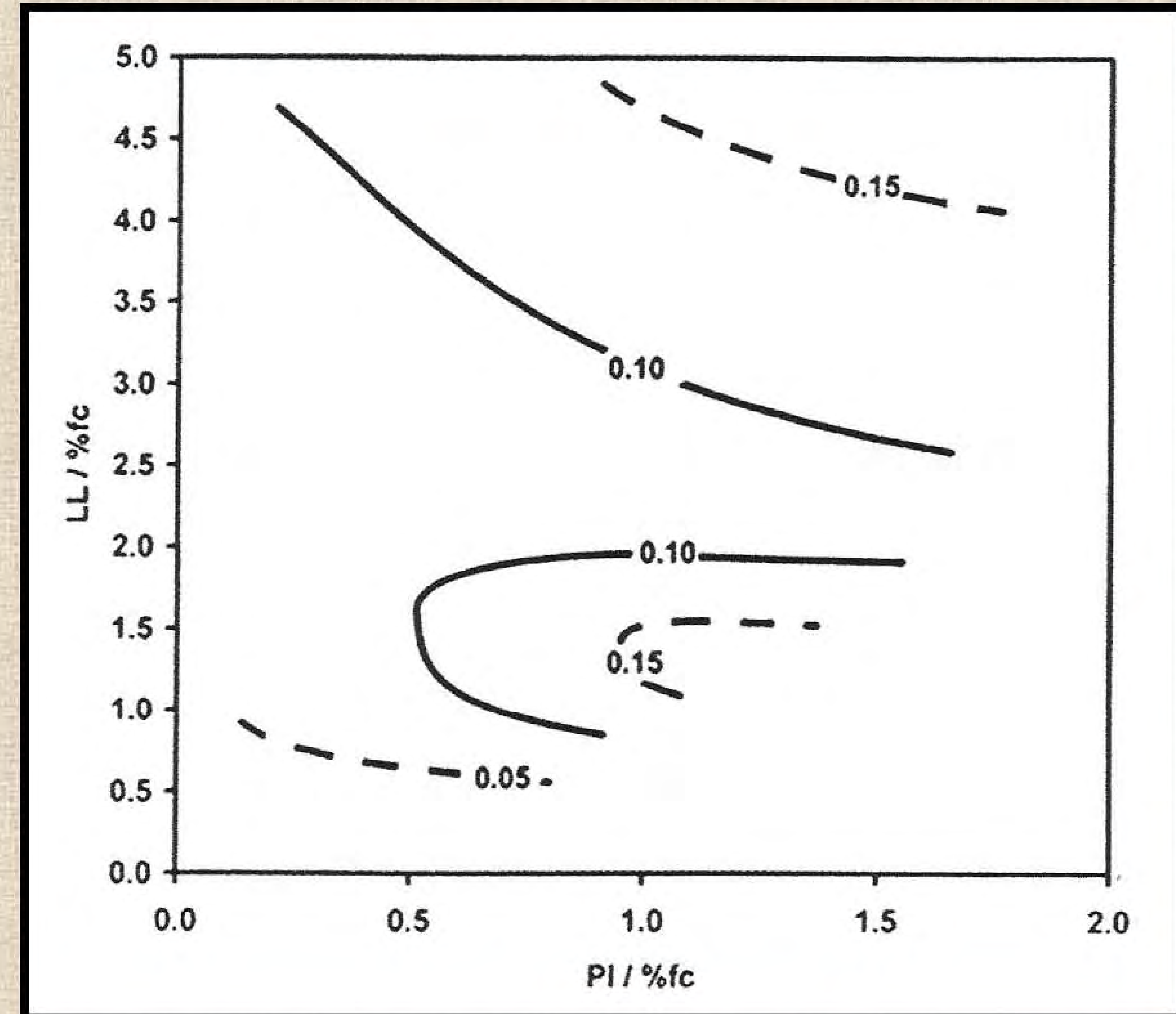


4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
LL / %fc
0.30
0.20
0.10
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
PI / %fc

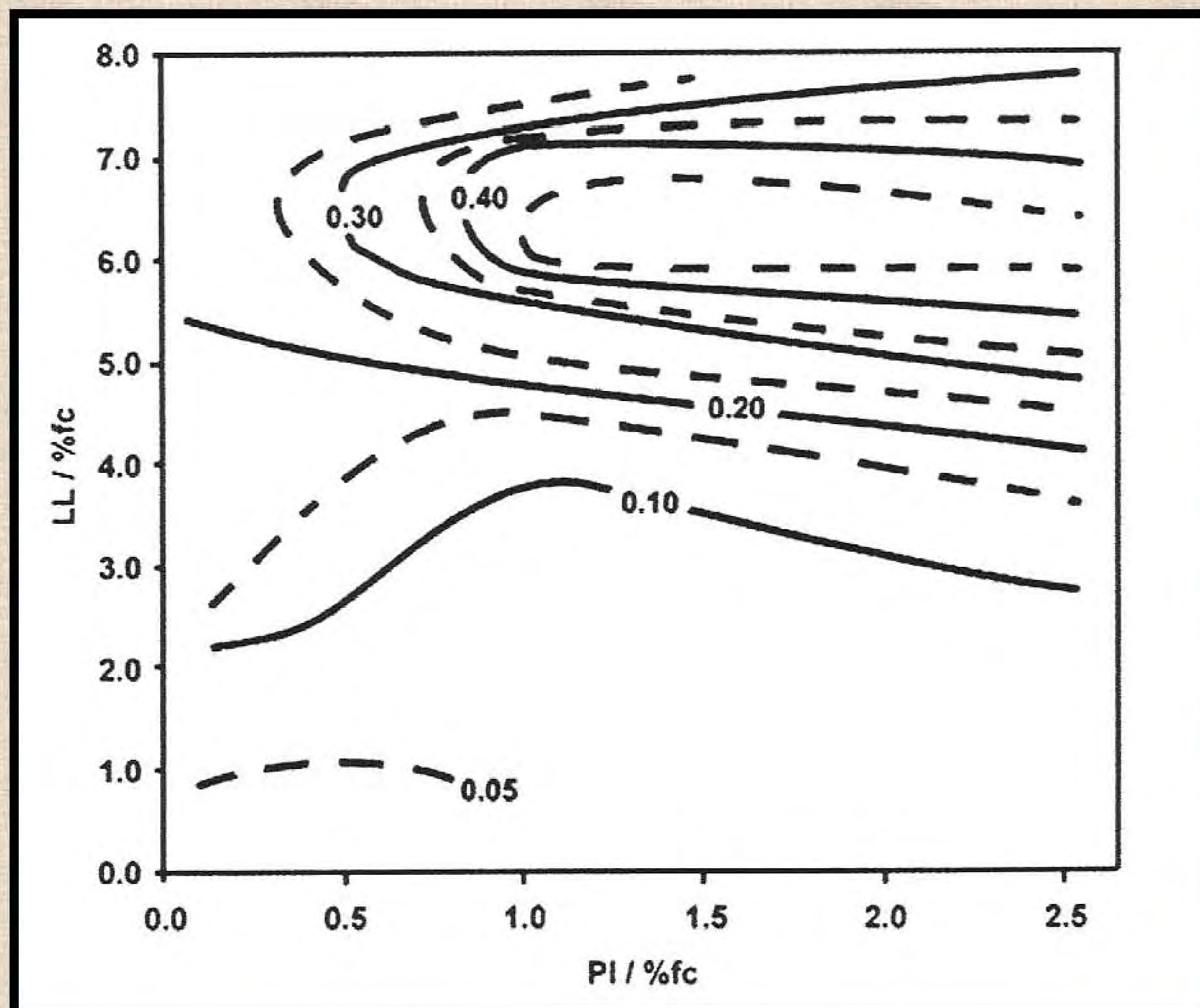


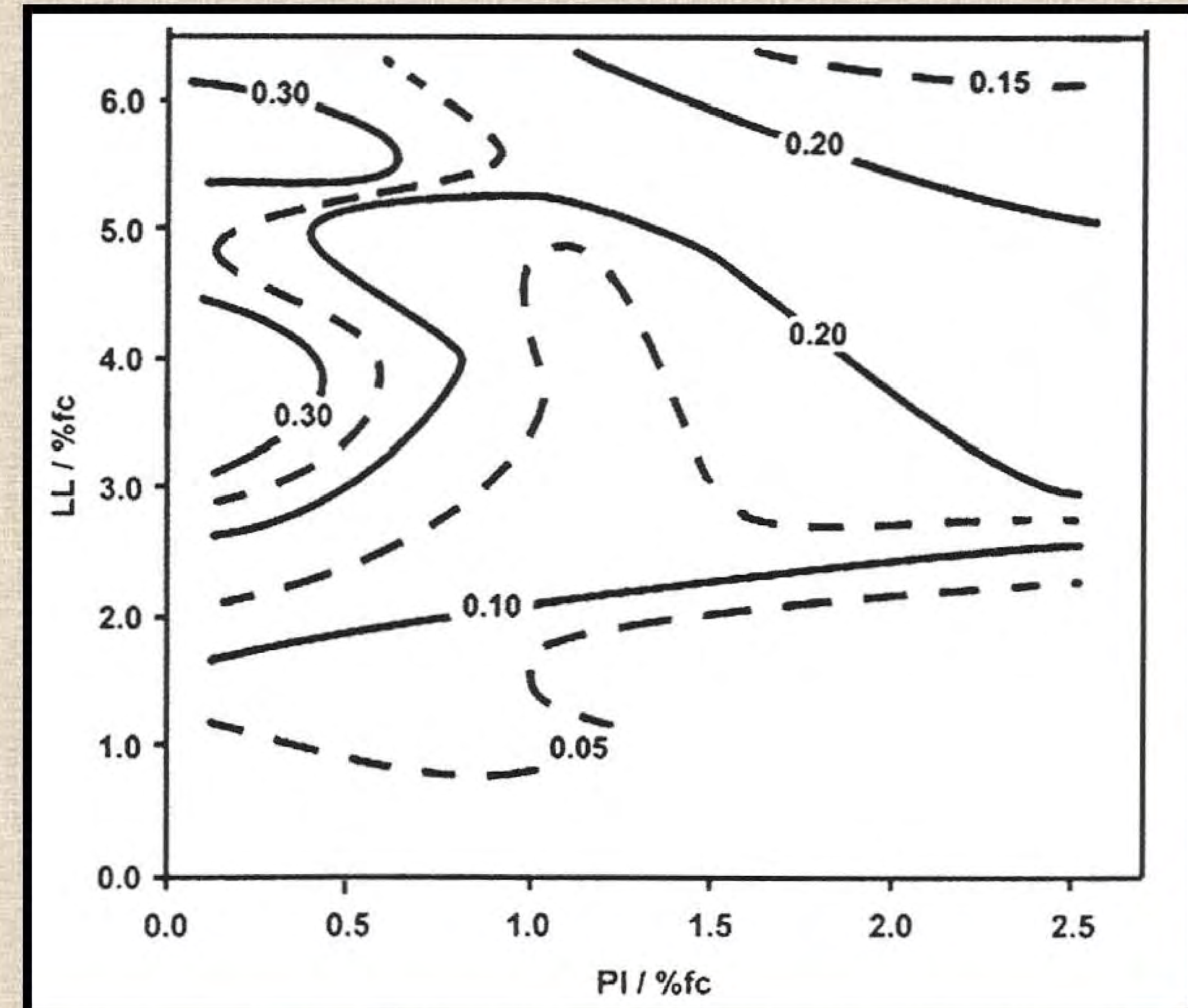
2.5
2.0
1.5
1.0
0.5
0.0
LL / %fc
0.0
0.5
1.0
1.5
2.0
PI / %fc
0.05
0.10
0.20
0.30

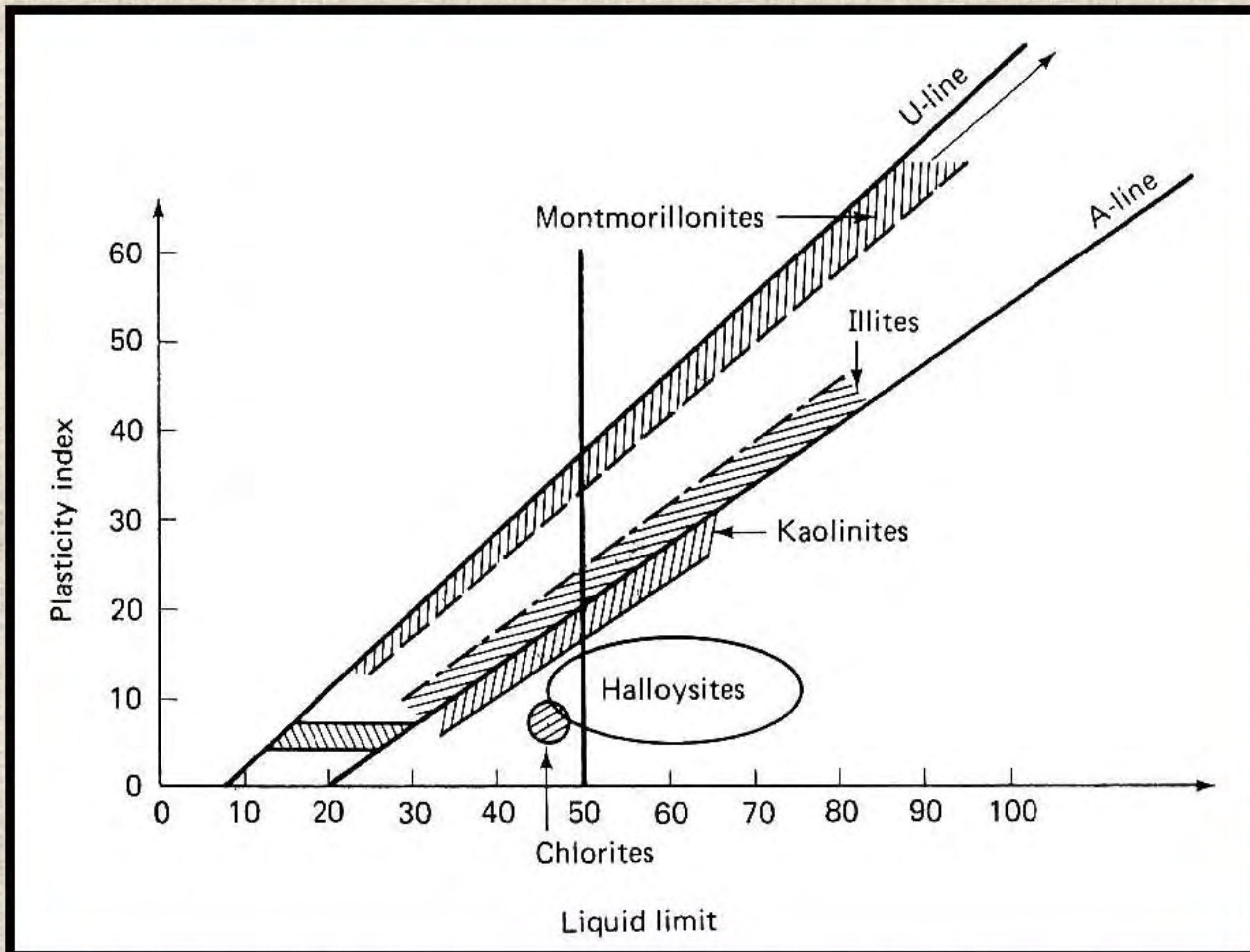

4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
LL / %fc
0.0
0.5
1.0
1.5
2.0
2.5
3.0
PI / %fc
0.20
0.10
0.10
0.20
0.30
0.10



5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
LL / %fc
0.15
0.10
0.10
0.15
0.05
0.0
0.5
1.0
1.5
2.0
PI / %fc


8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
LL / %fc
0.30
0.40
0.20
0.10
0.05
0.0
0.5
1.0
1.5
2.0
2.5
PI / %fc


0.30
0.30
0.15
0.20
0.20
0.10
0.05
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0.0
0.5
1.0
1.5
2.0
2.5
LL / %fc
PI / %fc


U-line
A-line
Montmorillonites
Illites
Kaolinites
Halloysites
Chlorites
Plasticity index
Liquid limit
0
10
20
30
40
50
60
70
80
90
100

VOLUME CHANGE RULE

$$\Delta V/V = -\gamma_h \log_{10}(h_f/h_i) - \gamma_\sigma \log_{10}(\sigma_f/\sigma_i) - \gamma_\pi \log_{10}(\pi_f/\pi_i)$$

$\Delta V/V$ = volume strain

h_i, h_f = initial and final matrix suction

σ_i, σ_f = initial and final mean principal stress

π_i, π_f = initial and final osmotic suction

$\gamma_h, \gamma_\sigma, \gamma_\pi$ = compression indexes

MEAN PRINCIPAL STRESS COMPRESSION INDEX

$$\gamma_\sigma = \frac{C_c}{1 + e_0}$$

Test No.	Pressure kPa	Volume Strain	$\tilde{a}_{\acute{o}}$	e_o	c_r
1	46.0	.0225	0.033	0.38	0.046
2	38.3	.0123	0.020	0.48	0.030
3	37.4	.0145	0.025	0.45	0.036
4	153.2	.0510	0.042	0.43	0.060
5	35.4	.0195	0.034	0.46	0.050
6	75.7	.0390	0.043	0.45	0.062
7	114.9	.0400	0.037	0.49	0.055
8	153.2	.0530	0.044	0.45	0.064
9	91.0	.0390	0.040	0.40	0.056
10	134	.0300	0.026	0.46	0.038
30	325	.0890	0.058	0.33	0.079
42	375	.1360	0.086	0.46	0.126
47	517	.0900	0.052	0.39	0.072
53	345	.1280	0.082	0.40	0.115
93	575	.0970	0.055	0.37	0.075
132	418	.1300	0.080	0.43	0.114
133	479	.0830	0.049	0.59	0.073
170	192	.1200	0.092	0.58	0.145

MEAN PRINCIPAL STRESS

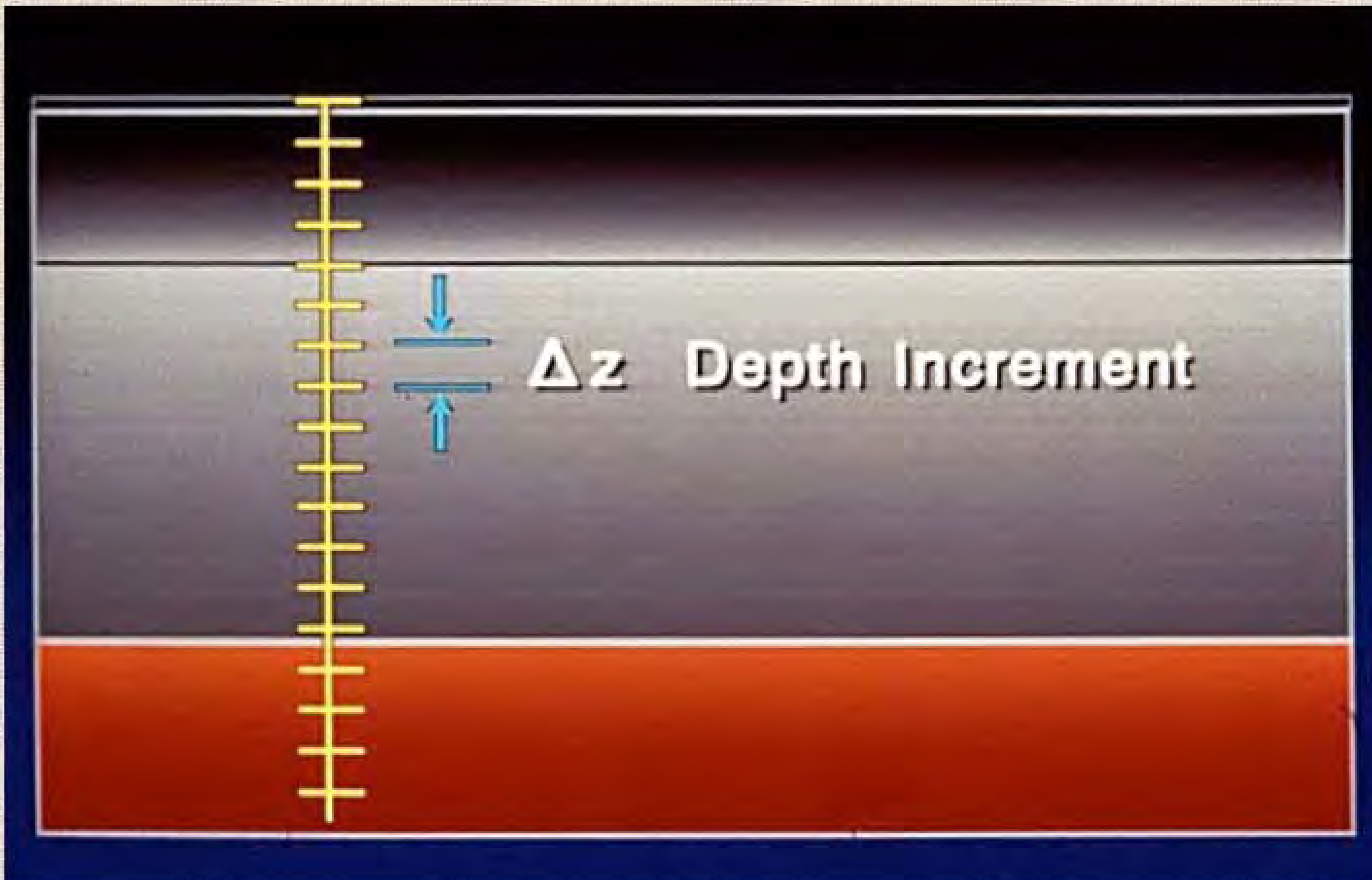
$$\sigma = \left(\frac{1 + 2K_0}{3} \right) \sigma_z$$

Mean Principal Stress

Vertical Pressure

Values of K_o

	Cracks	Suction
$K_o = 0$	Open	Dry
$K_o = 1/3$	Opening	Drying
$K_o = 1/2$	Closed	Steady
$K_o = 2/3$	Closed	Wetting
$K_o = 1$	Closed	Wetting
$K_o = 2\text{-}3$	Closed	Wetting


Δz Depth Increment

VERTICAL STRAIN

Vertical Strain

Volume Strain

$$\frac{\Delta H}{H} = f\left(\frac{\Delta V}{V}\right)$$

f = crack fabric factor

f = 0.5 soil drying

f = 0.8 soil wetting

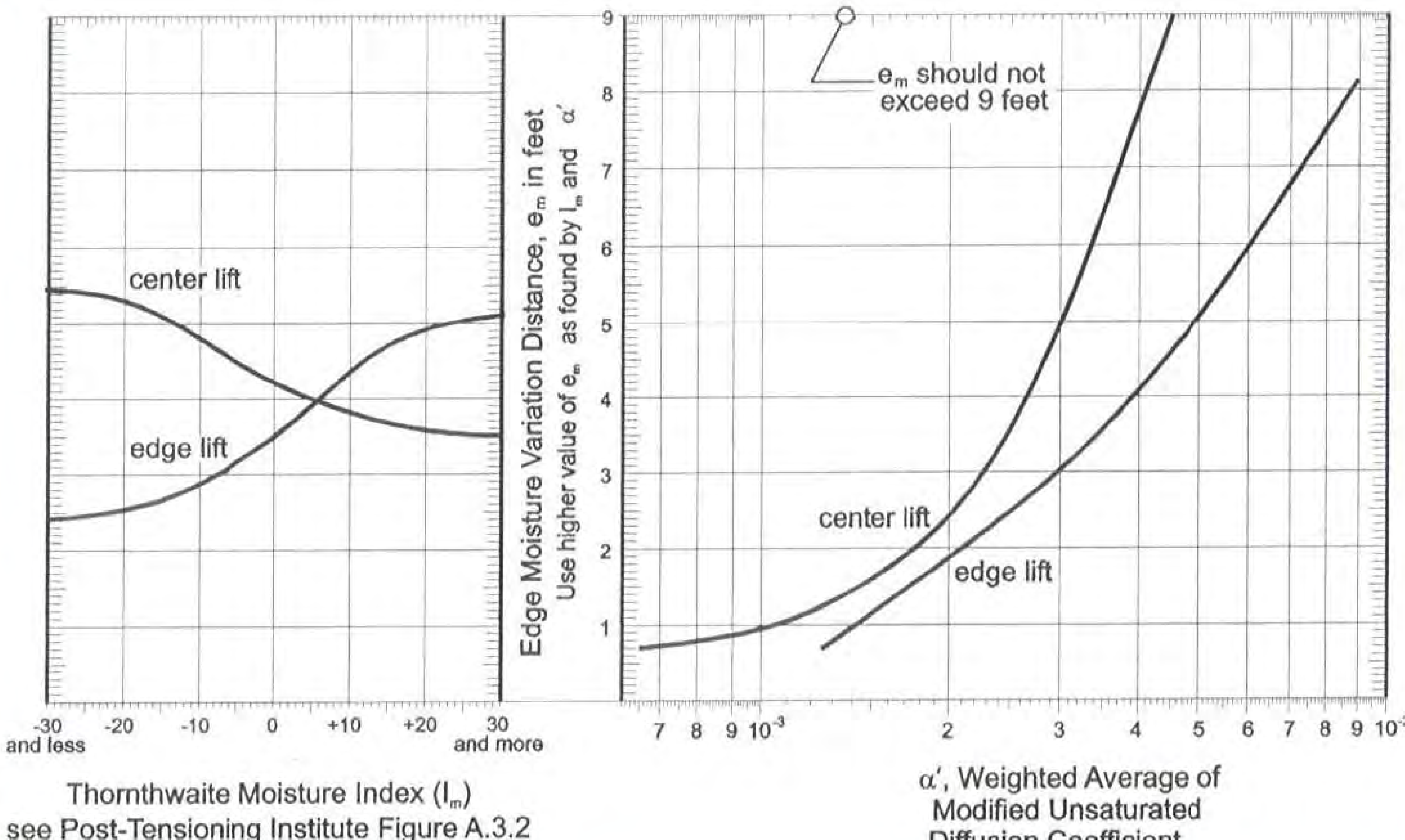
VERTICAL MOVEMENT

$$\Delta = \sum_{i=1}^{n} f_i \left(\frac{\Delta V}{V}\right)_i \Delta z_i$$

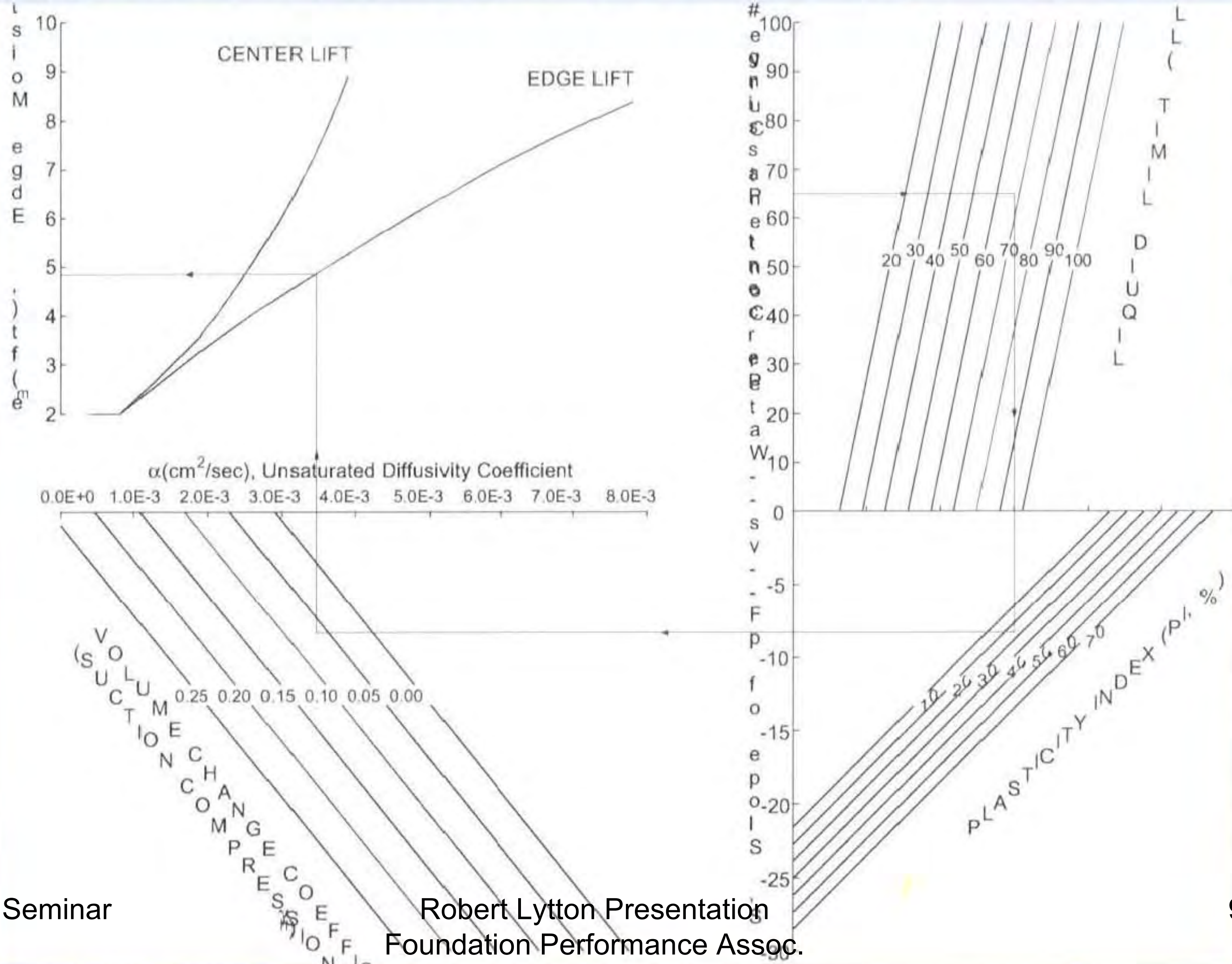
Vertical Movement

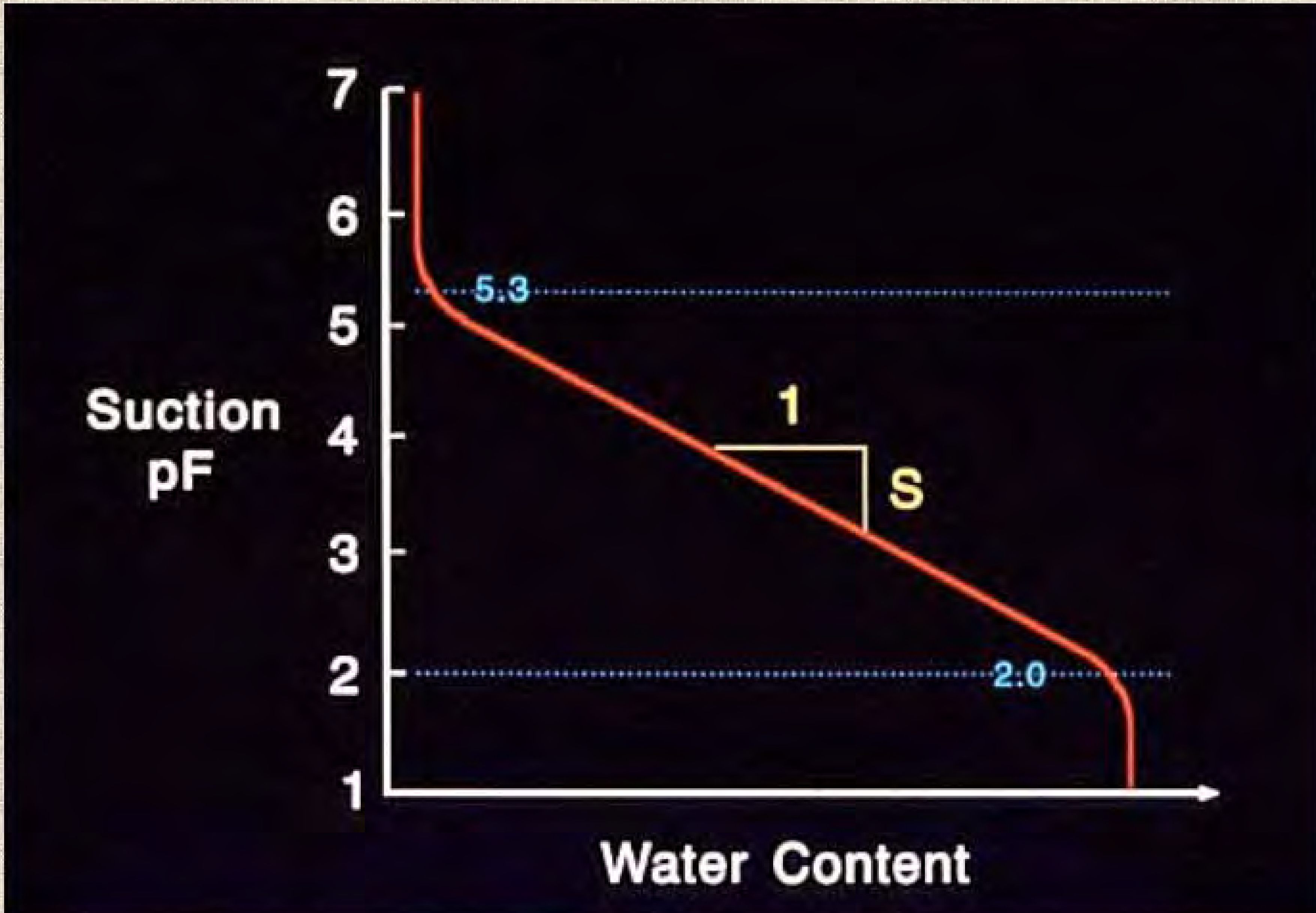
Depth Increment

Figure 8 - e_m Selection Chart



EDGE MOISTURE DISTANCE




7
6
5.3
5
Suction
pF
4
1
S
3
2
2.0
1
Water Content

DIFFUSION COEFFICIENT

$$\alpha = 0.0029 - 0.000162\,(S) - 0.0122\,(SCI)$$

α = Diffusion Coefficient

SCI = Suction Compression Index

RANGES OF SUCTION

CONVERSION OF UNITS

SIMPLIFIED SAMPLE CALCULATIONS OF

- VERTICAL FLOW
- HORIZONTAL FLOW
- HEAVE
- SHRINKAGE

R. L. LYTTON, Ph.D., P.E.

Notes originally prepared for:

The Expansive Soils Seminar

Conducted on June 21, 1985

At the University of Houston

At the Invitation of Dr. Michael O’Neil

RANGES OF SUCTION

The following scale is presented as a guide in determining reasonable levels of suction for estimating differential and total heave and shrinkage.

7 — Oven Dry

6 — Air Dry (Relative Humidity = 50%)

5

— Wilting Point (pF = 4.5)

4

— Plastic Limit of Clays (pF = 3.5)

3

— Wet Limit of Clay in the Field (pF = 2.5)

2 — Field Capacity (wettest soil in the field)

1 — Liquid Limit

CONVERSION OF UNITS

Geotechnical laboratories may report suction measurements in a variety of units, especially with metric conversion going on in the United States. The following is to make it easy to convert from any one system of units into the pF – scale that is used in the VOLFLO program.

CONVERSIONS TO pF

pF = log10 (kPa) + 1.009

pF = log10 (Tsf) + 2.990

pF = log10 (psi) + 1.847

pF = log10 (psf) + 0.311

CONVERSIONS TO *cm* OF SUCTION

$$cm \quad = \quad \frac{lb}{in^2}\ x\ 70.37 \quad \left(\frac{cm}{psi}\right)$$

$$cm \quad = \quad \frac{T}{ft^2}\ x\ 977.36 \ \left(\frac{cm}{Tsf}\right)$$

$$cm \quad = \quad kPa\ x\ 10.21 \quad \left(\frac{cm}{kPa}\right)$$

$$cm \quad = \quad psf\ \ x\ \ 0.4887 \ \left(\frac{cm}{psf}\right)$$

Typical Suction Levels

Air Dry	- 6.0 pF
Drying in Grass and Tree Root Zones	- 4.5 pF
Plastic Limit in Fat Clays	- 3.5 pF
Natural Water Content in Clays	- 3.2 – 3.7 pF
Clay Wet Limit	- 2.5 pF
Liquid Limit	- 1.0 pF

pF = log 10 (suction in cm)

Flow of Water in Clays

Horizontal Flow

$$v = k\frac{\Delta h}{\Delta x} \qquad \Delta h = \Delta x \left(\frac{v}{k}\right)$$

+ velocity = out of the soil

- velocity = into the soil

Δh = change in suction, in cm

Δx = change of horizontal location, in cm

Vertical Flow

$$v = -k\left(\frac{\Delta h}{\Delta z} + 1\right) \qquad \Delta h = \Delta z\left(\frac{v}{k} + 1\right)$$

Same sign convention on velocity flow direction

Δh = change in suction, in cm

Δz = upward change in elevation, in cm

Note: when $v = 0$, $\Delta h/\Delta z = -1$.

Permeability

$$k = 2 \times 10^{-6}\ \frac{cm}{sec}\left[\frac{-200cm}{-h(cm)}\right]$$

h = value of suction, cm

Volume Change in Clays

Percent Volume Change

1. Swelling — matrix suction swelling term; overburden + surcharge correction; osmotic suction swelling term

$$\left(\frac{\Delta v}{v}\right) = -\tilde{a}_h \log_{10}\left(\frac{h_f}{h_i}\right) - \tilde{a}_h \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \tilde{a}_o \log_{10}\left(\frac{\partial_f}{\partial_i}\right)$$

h_f = final matrix suction, cm.

h_i = initial matrix suction, cm.

σ_i = overburden correction constant

$$40^{cm} \text{ x } \tilde{a}_t \left({}^{gm}\!/_{cm^3}\right)$$

σ_f = mean pressure in $\left(g/cm^2\right)$ at depth z

$$\text{below 40 cm} \quad \left[= \tilde{a}_t \left(\frac{1+2\,K_o}{3}\right)\right]$$

∂_i, ∂_f = initial and final osmotic suction, cm.

$\left(\frac{\Delta v}{v}\right)$ = volume change percent (in decimal form)

$\tilde{a}_h$ = volume change coefficient

Note: Overburden and surcharge correction term is NOT applied above 40 cm or Below where it exceeds the swelling term or when it is the same sign.

2. Shrinking

$$\left(\frac{\text{Äv}}{v}\right) = -\tilde{a}_h \log_{10}\left(\frac{h_f}{h_i}\right) + \tilde{a}_h \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \tilde{a}_o \log_{10}\left(\frac{\partial_f}{\partial_i}\right)$$

shrinking term — overburden and surcharge correction term — osmotic suction shrinkage term

3. Lateral Earth Pressure Coefficient

K_o = 0.0 when there are many cracks in the soil

K_o = 1/3 when the soil is drying out

K_o = 2/3 when the soil is wetting up

K_o = 1.0 when the cracks are closed tightly

4. Vertical Volume Change at Depth, z

$$\left(\frac{\text{ÄH}}{H}\right) = f\left(\frac{\Delta v}{v}\right)$$

f = 0.5 when soil is drying out

f = 0.8 when soil is wetting up

5. Total Heave or Shrinkage

$$y_m = \sum_{i=1}^{n} \left(\frac{\Delta H}{H}\right) (\Delta z)$$

vertical volume change — vertical increment, cm

VolumeChangeCoefficient, $\tilde{a}_h$

Need to know : 1. PI%, LL%
2. % Fine Clay

1. PI(%) = Liquid Limit (LL) - Plastic Limit (PL)

2. $$\% \text{ Fine Clay} = \frac{\% \text{ Passing } (2ì) \text{ size}}{\% \text{ Passing } (\#200) \text{ size}} \times 100$$

3. $$\text{Activity Ratio, AC} = \frac{\text{PI } (\%)}{\% \text{ Fine Clay}}$$

Mineral Classification

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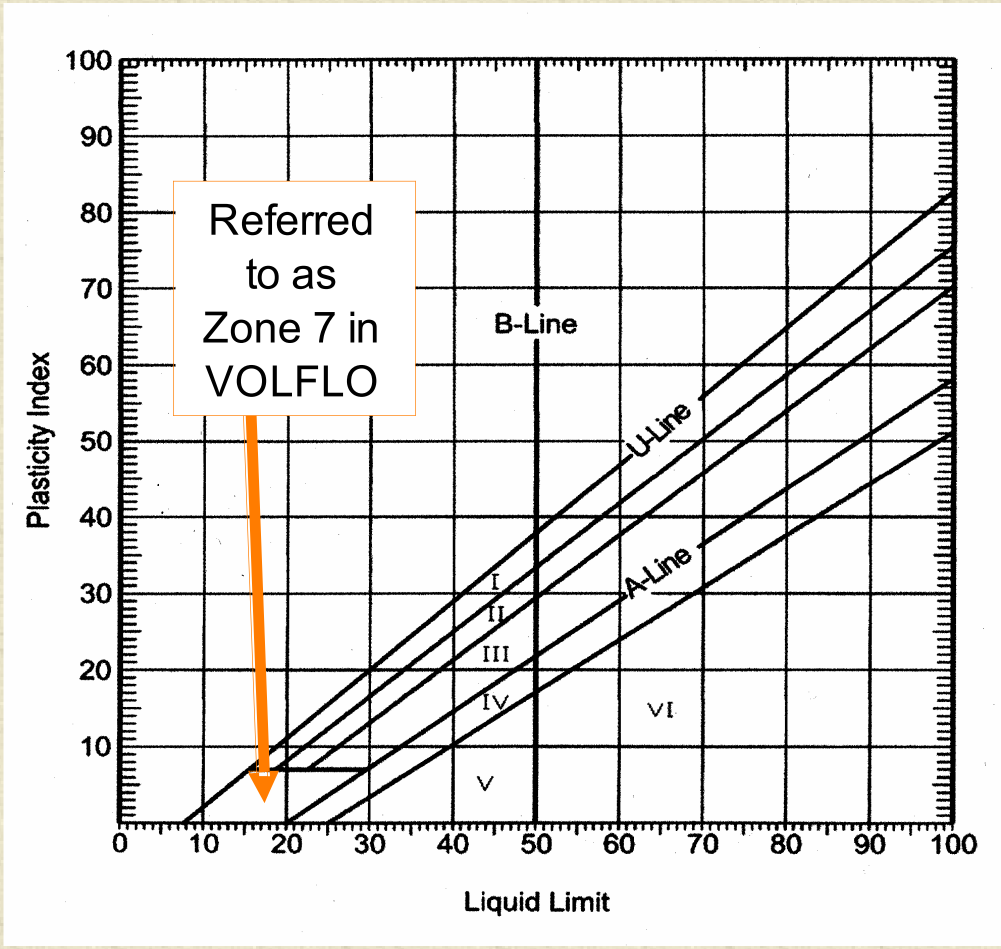

Note: No soil should plot above U -Line

Suction Compression Index – γ_o

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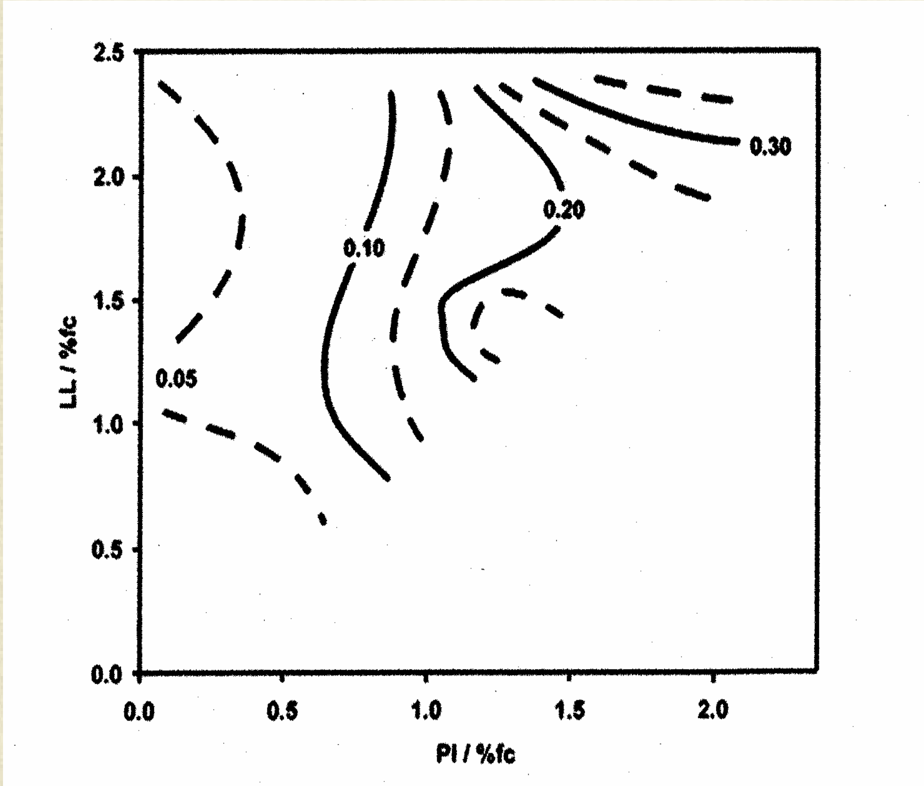
Zone II Chart



Note: There is no chart for "Zone 7". PTI recommends γ_o = 0. 01 for this zone.

4. Liquid Limit Activity $= \dfrac{LL(\%)}{\%\ \text{Fine Clay}}$

5. Volume Change Guide Number (From Chart), $\tilde{a}_o$

6. Volume Change Coefficient, $\tilde{a}_h$

$= \%$ Fine Clay (decimal) x $\tilde{a}_o$

Example Problem - Equilibrium Suction Profile

No vertical flow: v = o

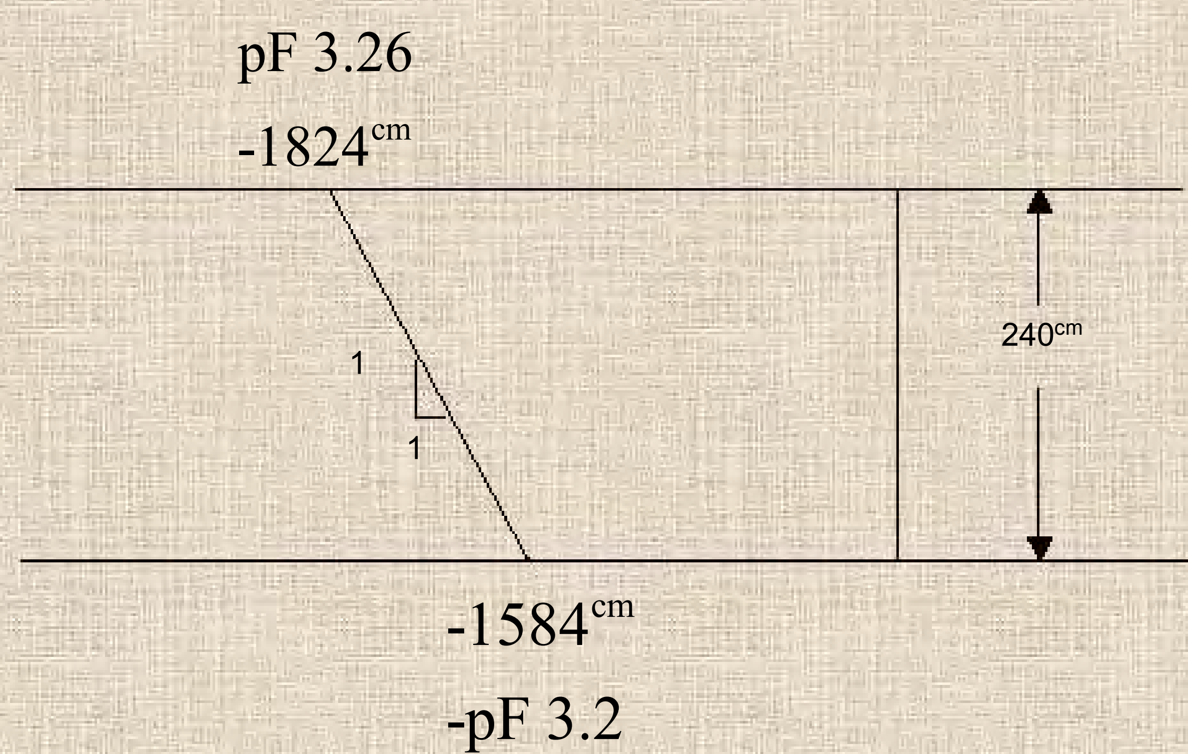
$$\frac{\Delta h}{\Delta z} = -1$$

Suction at 8 ft: pF 3.2 = - 1584 cm

Suction at Surface - 1584 cm + $\left(\dfrac{\Delta h}{\Delta z}\right)$ x 8' x 30 $\dfrac{cm}{ft}$

$$= -1584^{cm} + \left(-1\frac{cm}{cm}\right) \times 240^{cm}$$

$$= \underline{-1824}^{cm}$$


pF 3.26
-1824cm
1
1
240cm
-1584cm
-pF 3.2

Example Problem - Volume Change

$$\left.\begin{array}{l} LL = 76\% \\ PL = 22\% \\ PI = 54 \end{array}\right\} = \text{Soil Zone II}$$

% Fine Clay = 60%

$$\left.\begin{array}{r} AC = 54/60 = 0.90 \\ LLAC = 76/60 = 1.27 \end{array}\right\} \quad \text{Zone II}$$

$\tilde{a}_o = 0.15$ From Chart No. II

$\tilde{a}_h = 0.15 \text{ x } 0.60 = 0.09$

Assume $\tilde{a}_o =$ $= 0.09$

Dry pF = 4.20 (-15984$_{cm}$)

Wet pF = 2.60 (-398$_{cm}$)

$$\left(\frac{\Delta v}{v}\right) = -\tilde{a}_h \log_{10}\left(\frac{h_f}{h_i}\right) - \tilde{a}_{\acute{o}} \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)$$

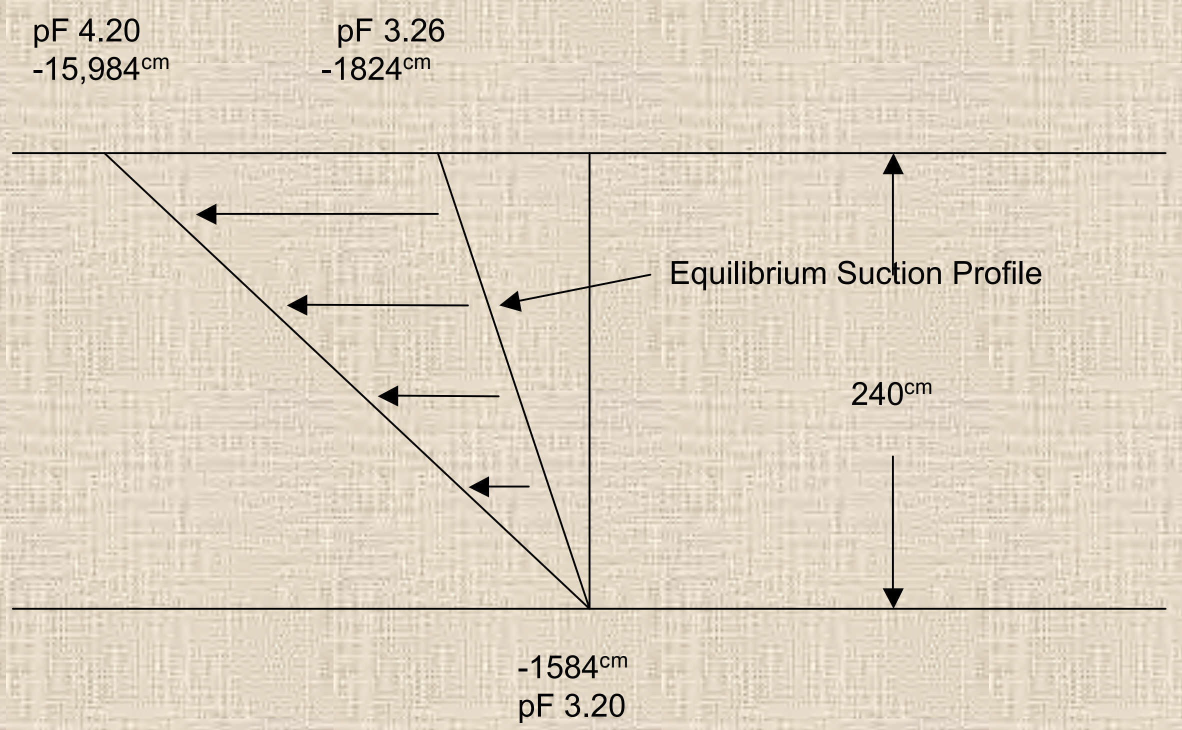
$$\frac{\Delta H}{H} = f\left(\frac{\Delta v}{v}\right)$$

f = 0.5 (Drying) $\acute{o}_i = 40^{cm} \text{ x } \tilde{a}_t$

f = 0.8 (wetting) $\acute{o}_f = 2 \text{ x } \tilde{a}_t \text{ x } \left(\frac{1 + 2Ko}{3}\right)$

Ko = 1 (Assume)

$$\left(\frac{\sigma_f}{\sigma_i}\right) = \left(\frac{z^{cm}}{40}\right)$$

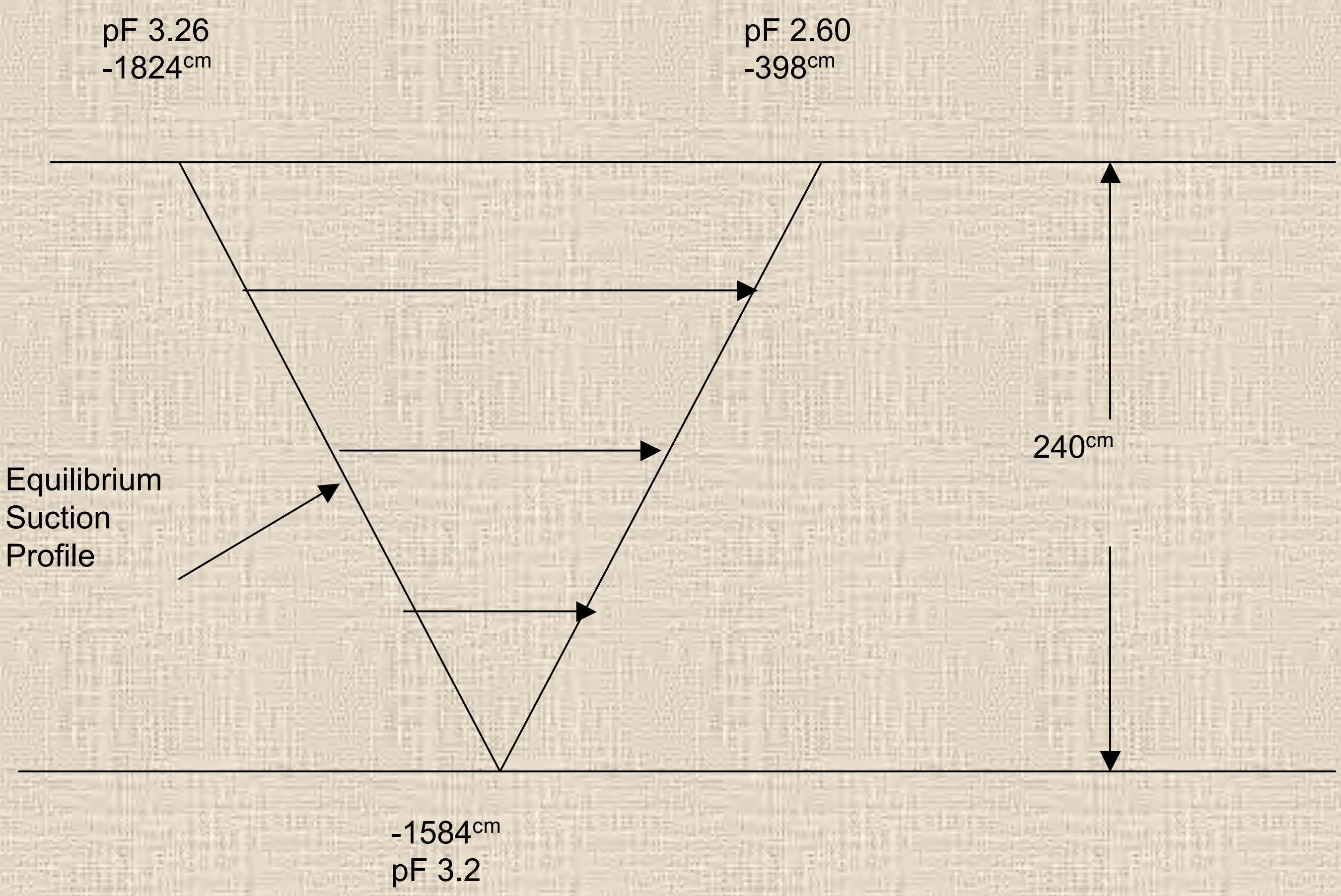

pF 4.20
-15,984cm
pF 3.26
-1824cm
Equilibrium Suction Profile
240cm
-1584cm
pF 3.20

Drying

$\tilde{a}_h = \tilde{a}_{ó} = 0.090$ $\quad f = 0.5$

	Depth, cm	h_i, cm	h_f, cm	$-\gamma_h \log_{10}\left(\frac{hf}{hi}\right)$	$+ãó \log_{10}\left(\frac{óf}{ó_i}\right)$	$\left(\frac{Äv}{v}\right)$	$\left(\frac{ÄH}{H}\right)$
0’	0	-1824	-15984	-0.0848		-0.0848	-0.0424
	20	-1804	-14784	-0.0822		-0.0822	-0.0411
	40	-1784	-13584	-0.0793		-0.0793	-0.0397
2’	60	-1764	-12384	-0.0762	0.0158	-0.0604	-0.0302
	80	-1744	-11184	-0.0726	0.0271	-0.0455	-0.0228
	100	-1724	-9984	-0.0686	0.0358	-0.0328	-0.0164
4’	120	-1704	-8784	-0.0641	0.0429	-0.0212	-0.0106
	140	-1684	-7584	-0.0588	0.0490	-0.0098	-0.0049
	160	-1664	-6384	-0.0526	0.0542		
6’	180	-1644	-5184	-0.0449	0.0588		
	200	-1624	-3984	-0.0351	0.0629		
	220	-1604	-2784	-0.0216	0.0666		
8’	240	-1584	-1584	-0.0000	0.0700		
					Assume		
					Ko = 1		

Depth, cm	$\left(\frac{\text{ÄH}}{H}\right)$	H, cm	ÄH, cm	y_m	
0	-0.0424	10	-0.424	3.738 cm	1.47in
20	-0.0411	20	-0.822		
40	-0.0397	20	-0.794		
60	-0.0302	20	-0.604		
80	-0.0228	20	-0.456		
100	-0.0164	20	-0.328		
120	-0.0106	20	-0.212		
140	-0.0049	20	-0.098		
160					
180					
200					
220					
240					


pF 3.26
-1824cm
pF 2.60
-398cm
240cm
Equilibrium
Suction
Profile
-1584cm
pF 3.2

Wetting

$\tilde{a}_h = \tilde{a}_{ó} = 0.090$ $f = 0.8$

	Depth, cm	h_i, cm	h_f, cm	$-\gamma_h \log_{10}\left(\frac{hf}{hi}\right)$	$-ãó \log_{10}\left(\frac{óf}{ó_i}\right)$	$\left(\frac{Äv}{v}\right)$	$\left(\frac{ÄH}{H}\right)$
0’	0	-1824	-398	+0.0595		+0.0595	0.0476
	20	-1804	-497	+0.0504		+0.0504	0.0403
	40	-1784	-596	+0.0429		+0.0429	0.0343
2’	60	-1764	-694	+0.0365	-0.0158	+0.0207	0.0166
	80	-1744	-793	+0.0308	-0.0271	+0.0037	0.0030
	100	-1724	-892	+0.0258	-0.0358		
4’	120	-1704	-991	+0.0212	-0.0429		
	140	-1684	-1090	+0.0170	-0.0490		
	160	-1664	-1189	+0.0131	-0.0542		
6’	180	-1644	-1287	+0.0096	-0.0588		
	200	-1624	-1386	+0.0062	-0.0629		
	220	-1604	-1485	+0.0030	-0.0666		
8’	240	-1584	-1584	0.0000	-0.0700		
					Assume		
					Ko = 1		

Wetting (Continued)

Depth, cm	$\left(\frac{ÄH}{H}\right)$	H, cm	ÄH, cm	y_m, cm
0	+0.0476	10	0.476	2.360cm = 0.93in
20	+0.0403	20	0.806	
40	+0.0343	20	0.686	
60	+0.0166	20	0.332	
80	+0.0030	20	0.060	
100				
120				
140				
160				
180				
200				
220				
240				

Wetting With Open Cracks

$$\text{ã}_h = \text{ãó} = 0.090$$

$$\frac{\text{wetting}}{f = 0.8}$$

	Depth, cm	$-\text{ã}_h \log_{10}\left(\frac{h_f}{h_i}\right)$	$-\text{ãó} \log_{10}\left(\frac{\text{ó}_f}{\text{ó}_i}\right)$	$\left(\frac{\text{Äv}}{v}\right)$	$\left(\frac{\text{ÄH}}{H}\right)$	H, cm	ÄH	y_m, cm
0’	0	+0.0595		+0.0595	0.0476	10	0.476	4.002cm = 1.58in
	20	+0.0504		+0.0504	0.0403	20	0.806	
	40	+0.0429		+0.0429	0.0343	20	0.686	
2’	60	+0.0365		+0.0365	0.0292	20	0.584	
	80	+0.0308		+0.0308	0.0246	20	0.492	
	100	+0.0258		+0.0258	0.0206	20	0.412	
4’	120	+0.0212		+0.0212	0.0170	20	0.340	
	140	+0.0170	-0.0060	+0.0110	0.0088	20	0.176	
	160	+0.0131	-0.0112	+0.0019	0.0015	20	0.030	
6’	180	+0.0096	-0.0158					
	200	+0.0062	-0.0200					
	220	+0.0030	-0.0237					
8’	240	+0.0000	-0.0271					
			Assume					
			$K_o = 0$					

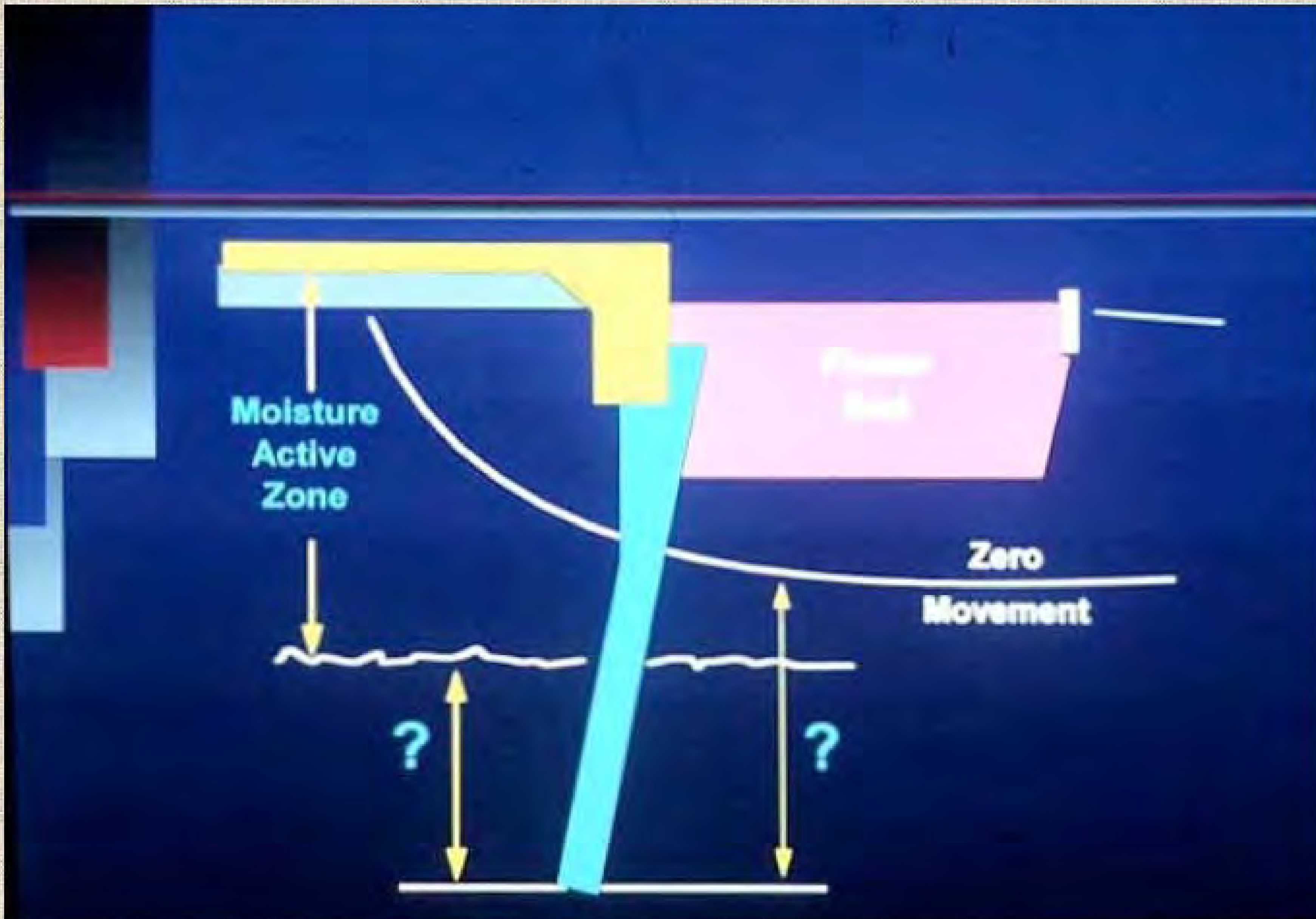
$$ó_f = \frac{Z}{3} \text{ x } ã_t$$

$$ó_i = 40 \text{ x } ã_t$$

$$\frac{ó_f}{ó_i} = \frac{Z}{120}$$

PROGRAM AGENDA

1:00 pm	Geotechnical and Structural Design of Post-Tensioned Slabs-on-Ground using PTI 2004 Manual and Computer Programs VOLFLO 1.5 and PTISLAB 3.0 – Meyer, Read
2:30 pm	Break
➢**2:40 pm**	**Design Concepts of Various Foundation Systems – Lytton**
	Drilled Footings
	Floating Slabs
	Moisture Barrier
	Root Barrier
	Slopes
	Pavements
	Sulfates
4:00 pm	Forensic Evaluation of Foundations – Dr. David Eastwood, P.E.
4:30 pm	Legal Issues – Mr. David Dorr, Esquire
5:00 pm	Panel Discussion
	Questions and Answers
6:00 pm	Adjourn


Moisture
Active
Zone
Zero
Movement
?
?

Questions:

How deep to carry piers?

a. Near trees

b. Near flower beds

Can vertical moisture or root barriers help?

a. Near trees

b. Near flower beds

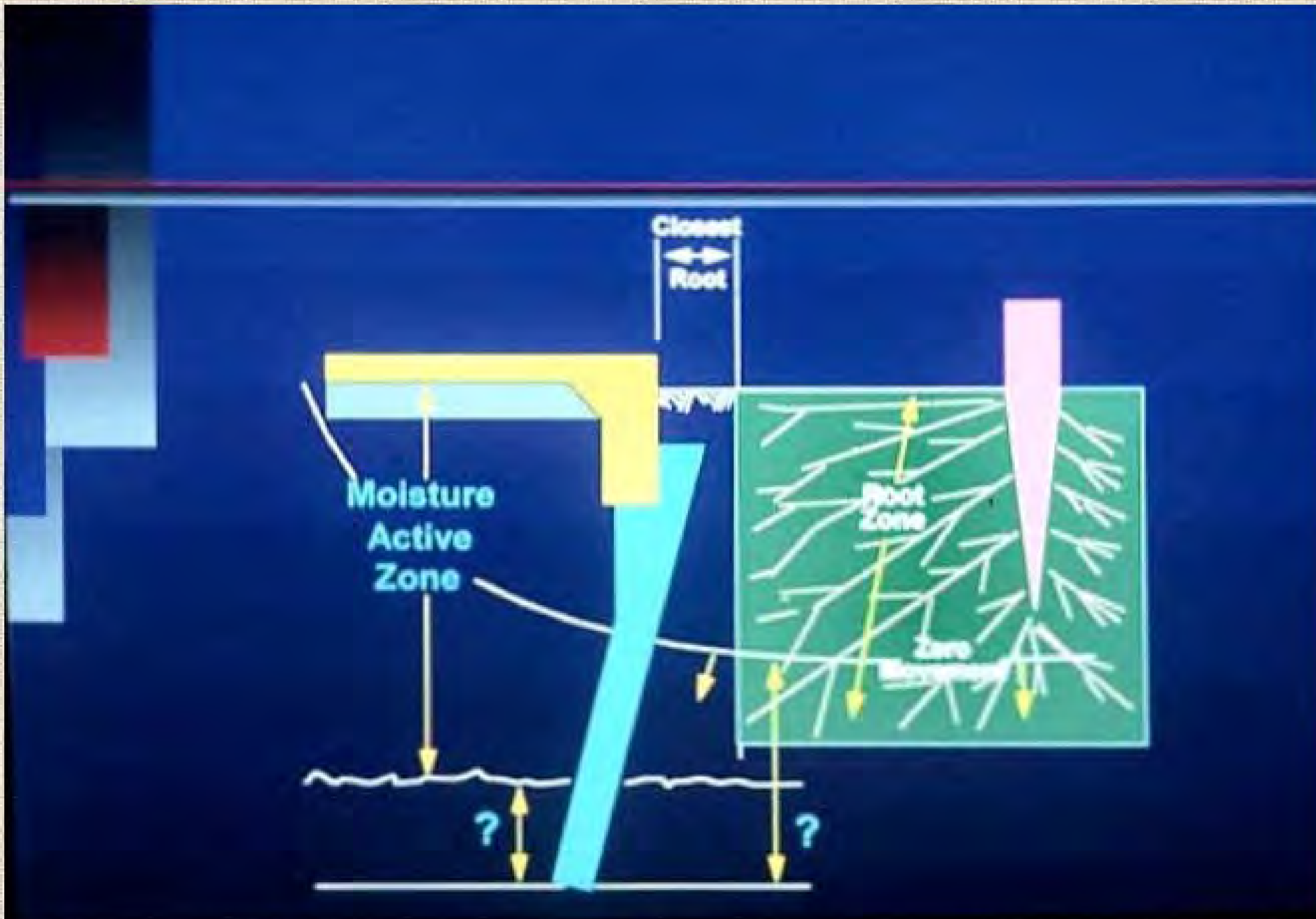
Clay Profile	Percent of Dallas County
Houston Black Clay	18.2%
Trinity Clay	10.9%
Total	29.1%

Typical Clay Properties

Property	Range	Example
Liquid Limit	55-90	86
Plasticity	30-60	59
Plastic Limit	25-30	27
Percent Passing #200	80-99	97
Percent Passing 0.002mm	-	65

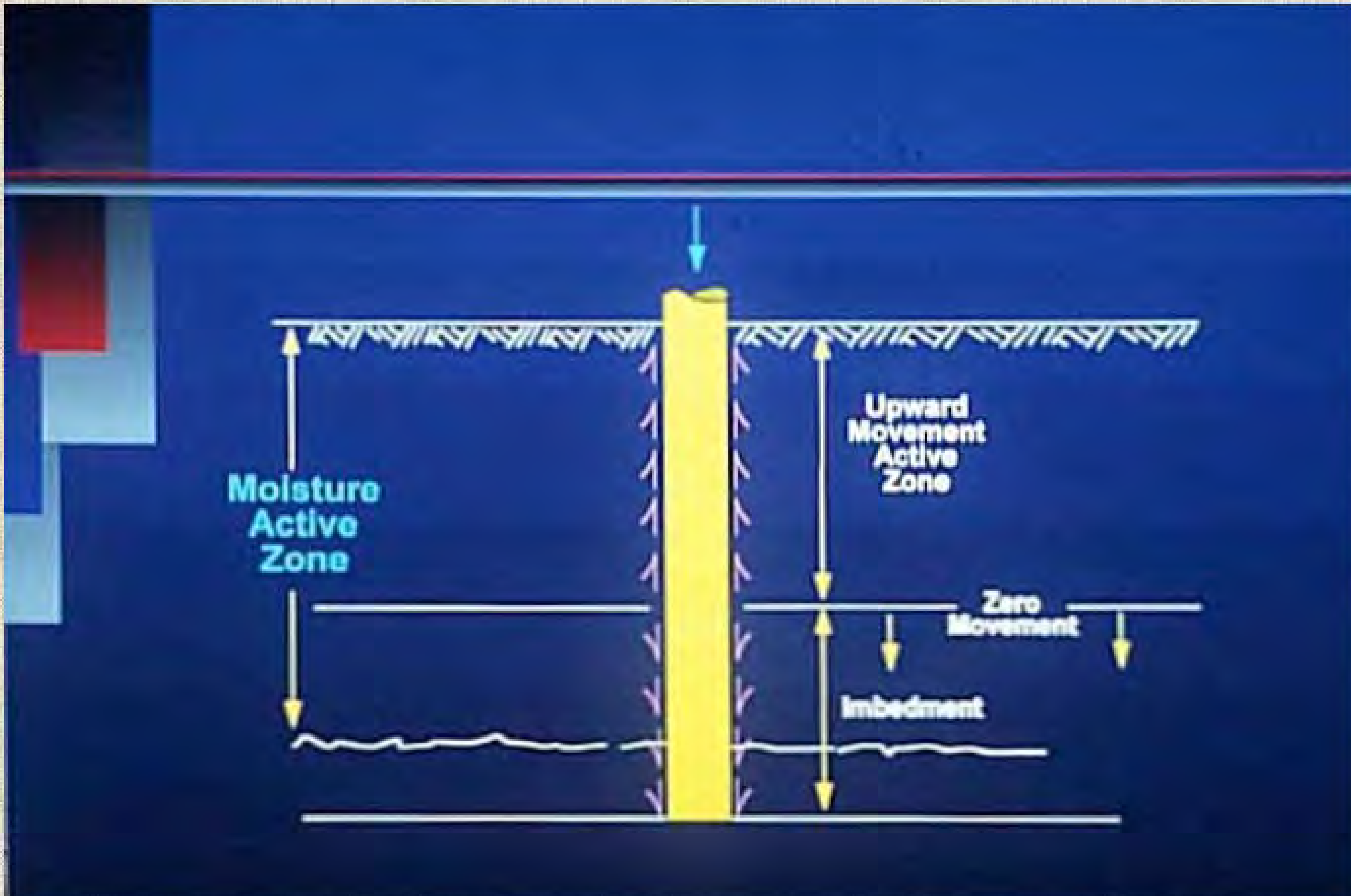
DERIVED EXAMPLE CLAY PROPERTIES

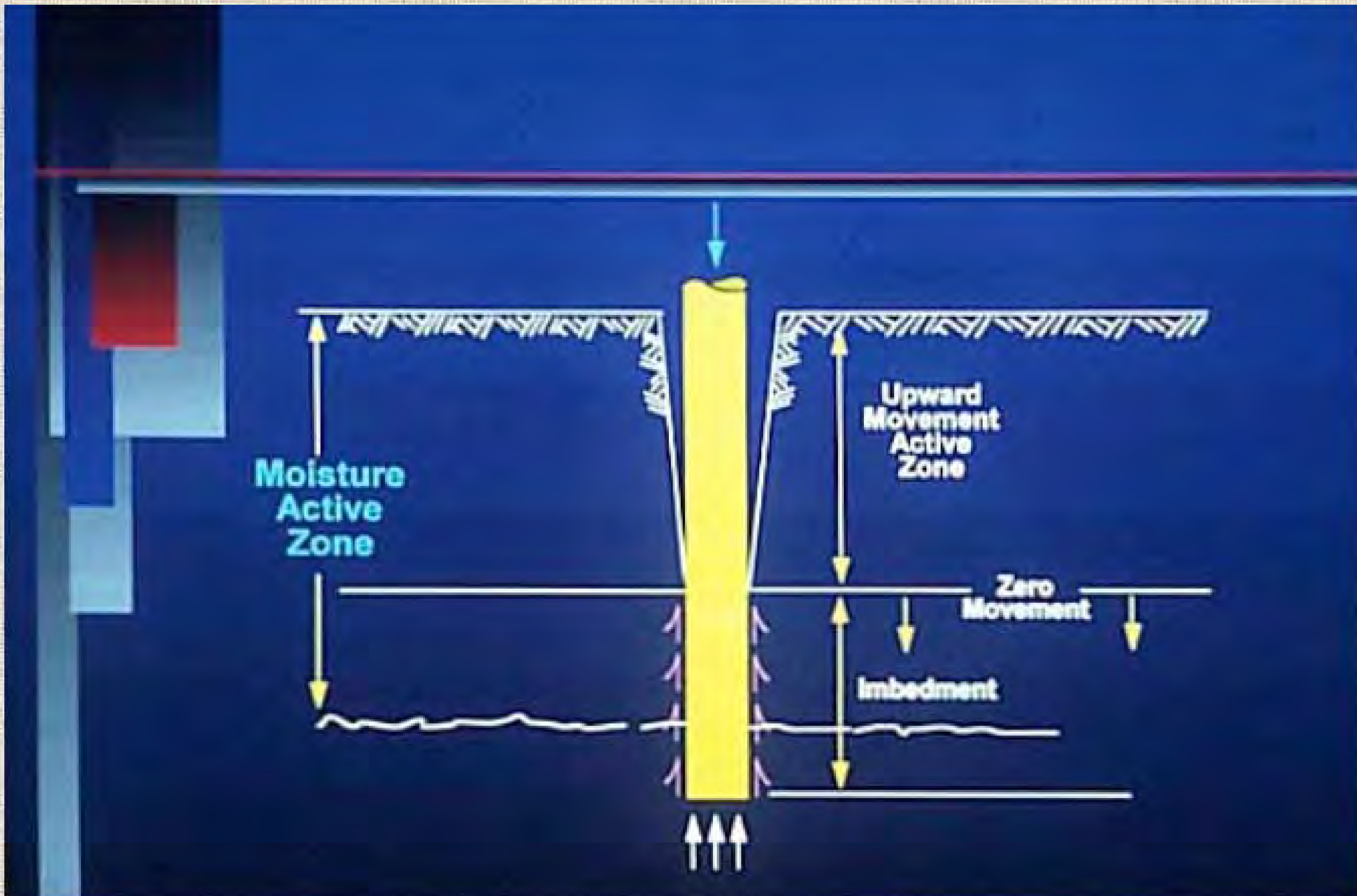
PROPERTY	VALUE
PERCENT FINE CLAY (% fc)	67%
ACTIVITY INDEX (PI/% fc)	0.88
LIQUID LIMIT INDEX (LL/% fc)	1.28
SOIL ZONE (CHART NO.)	II
100% SUCTION COMPRESSION INDEX (γ_O)	0.16
SUCTION COMPRESSION INDEX (γ_O x % fc/100)	0.107


Closest
Root
Moisture
Active
Zone
Root
Zone
Zero
?
?

DERIVED EXAMPLE CLAY PROPERTIES

PROPERTY	VALUE
SLOPE OF pF-vs-WATER CONTENT	-7.19
UNSATURATED DIFFUSIVITY	$3.07 \times 10^{-3} \frac{cm^2}{sec}$
THORNTHWAITE MOISTURE INDEX	-11
EDGE MOISTURE VARIATION DISTANCE	
DRYING	7.4 FEET
WETTING	4.8 FEET


Moisture
Active
Zone
Upward
Movement
Active
Zone
Zero
Movement
Imbedment


Moisture
Active
Zone
Upward
Movement
Active
Zone
Zero
Movement
Imbedment

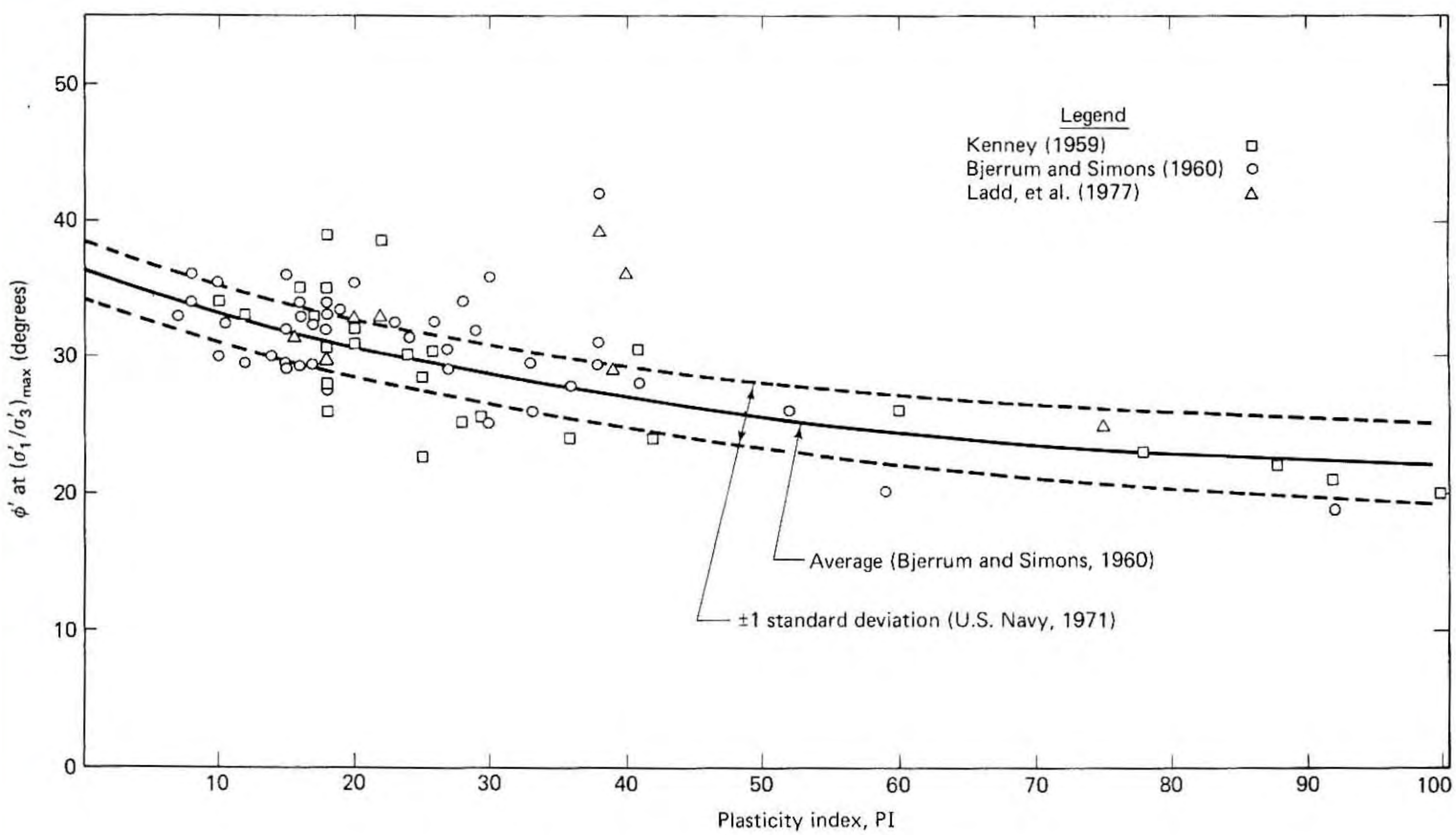
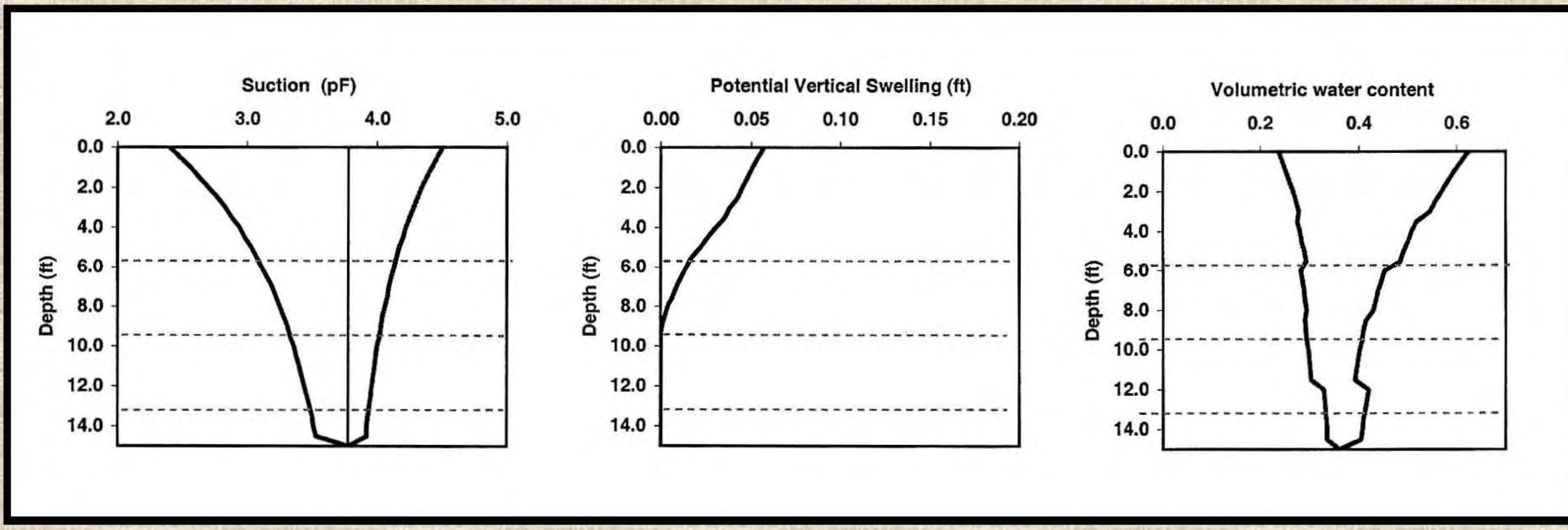
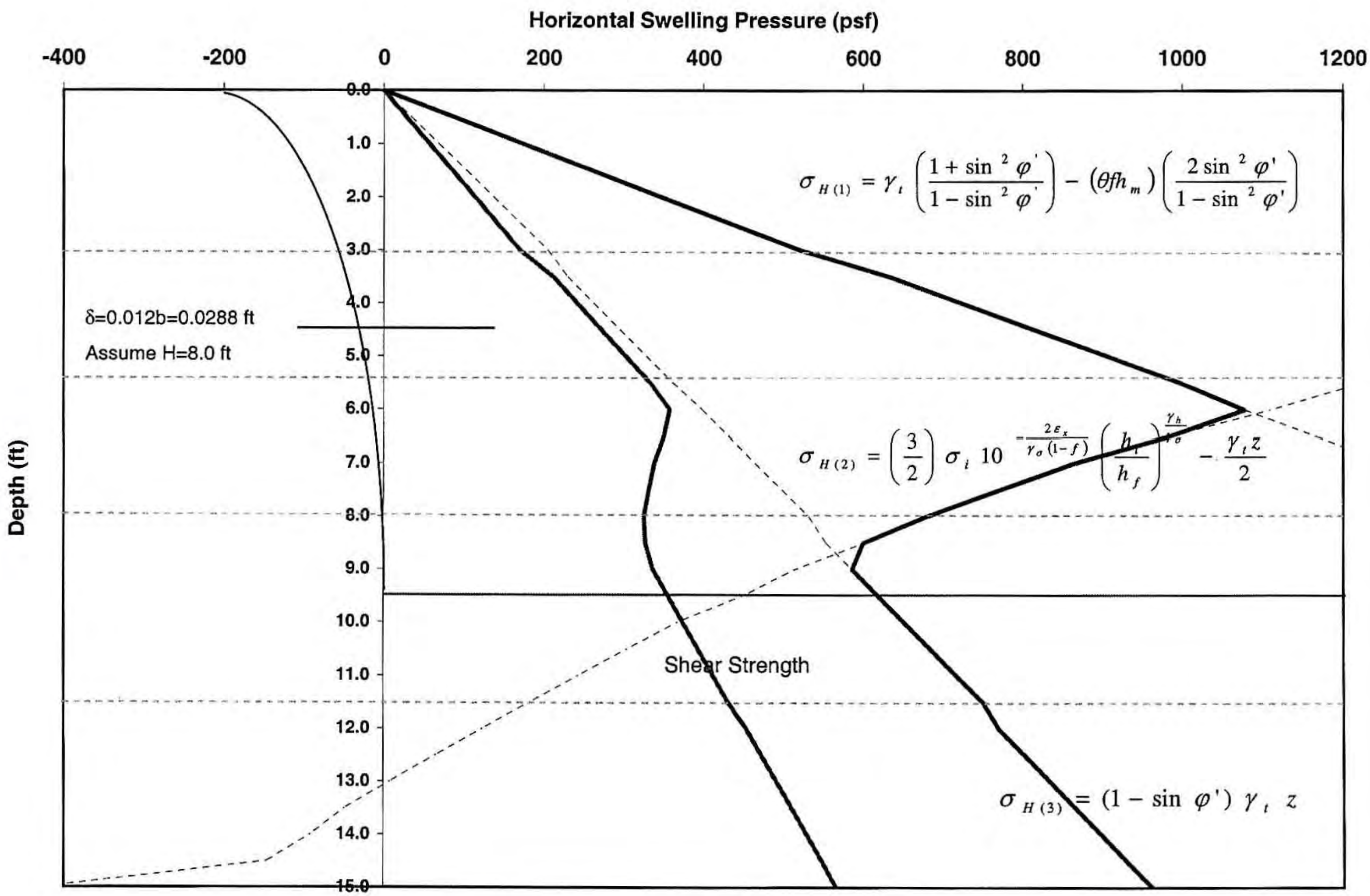


Fig. 11.27 Empirical correlation between ϕ' and PI from triaxial compression tests on normally consolidated undisturbed clays (after U.S. Navy, 1971, and Ladd, et al., 1977).


Suction (pF)
2.0
3.0
4.0
5.0
Depth (ft)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
Potential Vertical Swelling (ft)
0.00
0.05
0.10
0.15
0.20
Volumetric water content
0.0
0.2
0.4
0.6



Horizontal Swelling Pressure (psf)
-400
-200
0
200
400
600
800
1000
1200
Depth (ft)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
δ=0.012b=0.0288 ft
Assume H=8.0 ft
$\sigma_{H(1)} = \gamma_t \left(\frac{1+\sin^2\varphi'}{1-\sin^2\varphi'}\right) - (\theta f h_m)\left(\frac{2\sin^2\varphi'}{1-\sin^2\varphi'}\right)$
$\sigma_{H(2)} = \left(\frac{3}{2}\right)\sigma_i\, 10^{-\frac{2\varepsilon_x}{\gamma_\sigma(1-f)}}\left(\frac{h_i}{h_f}\right)^{\frac{\gamma_h}{\gamma_\sigma}} - \frac{\gamma_t z}{2}$
Shear Strength
$\sigma_{H(3)} = (1-\sin\varphi')\,\gamma_t\, z$

HORIZONTAL EARTH PRESSURE COEFFICIENTS FOR VOLUMETRICALLY ACTIVE SOILS

CONDITION	TYPICAL Ko	FORMULA
Cracked	0	0
Drying (Active)	1/3	$\frac{1-\sin\phi'}{1+\sin\phi'}$
Equilibrium (At Rest)	1/2	$1+\sin\phi'$
Wetting (within movement active zone)	2/3	$\frac{1-\sin\phi'}{1+\sin\phi'}\left(\frac{1+\sin\phi'}{1-\frac{1}{2}\sin\phi'}\right)$
Wetting (below movement active zone)	1	$\frac{1-\sin\phi'}{1+\sin\phi'}\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)$
Swelling Near Surface (Passive Earth Pressure)	3	$1 \quad \frac{1+\sin\phi'}{1-\sin\phi'}$

In general, other than the "cracked" case where Ko = 0,

$$Ko = e\left(\frac{1-\sin\phi'}{1+\sin\phi'}\right)\left(\frac{1+d\sin\phi'}{1-k\sin\phi'}\right)^{n}$$

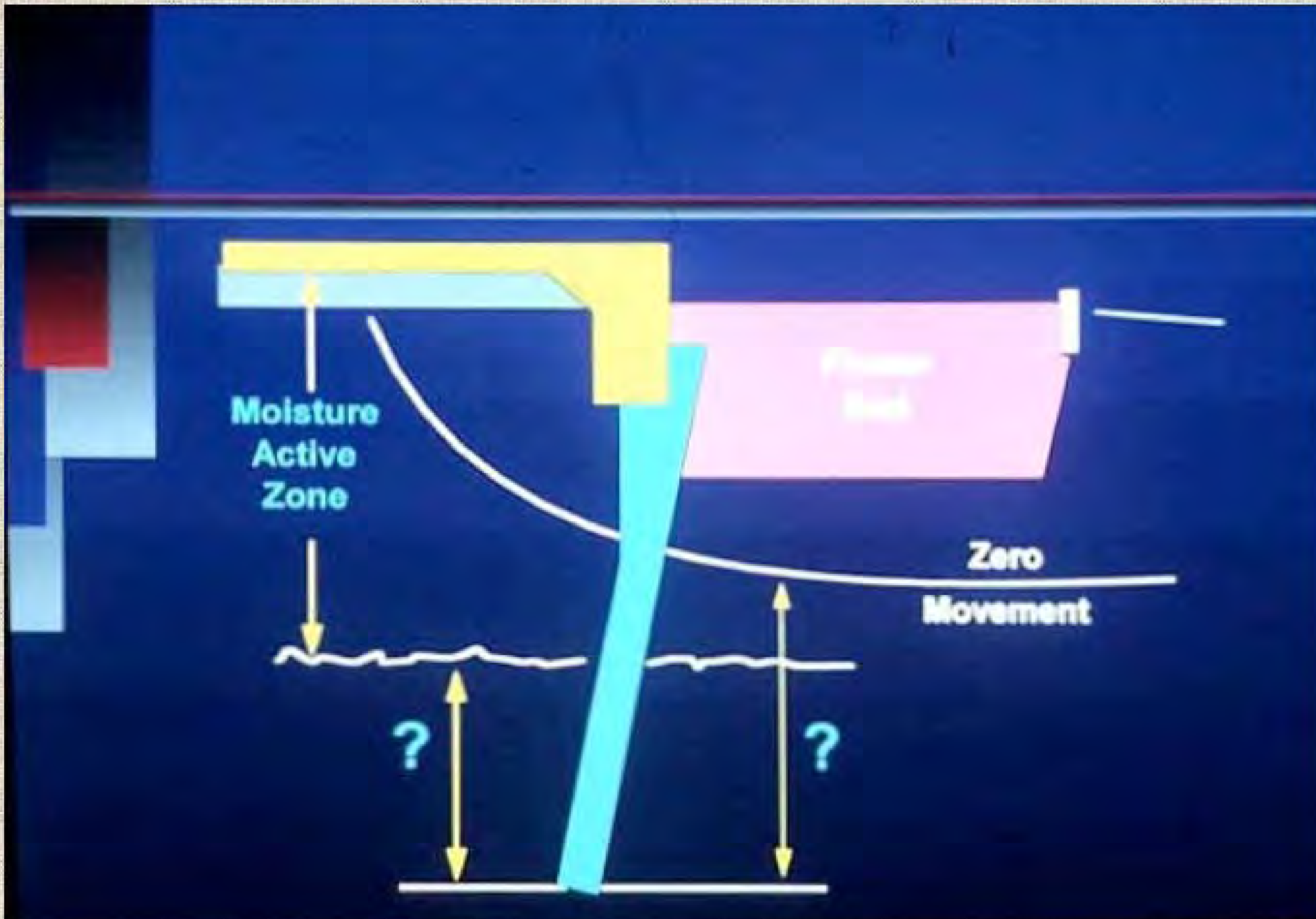
DEPTH	SOIL CONDITION	SUCTION DIFFERENCE, $pF_f - pF_o = \Delta pF$
MOISTURE ACTIVE ZONE, Z_m; MOVEMENT ACTIVE ZONE, Z_A	CRACKED	$1.5 \le \Delta pF$
	DRYING (ACTIVE)	$0.2 \le \Delta pF \le 1.5$
	EQUILIBRIUM	$-0.2 \le \Delta pF \le 0.2$
	WETTING (IN THE MOVEMENT ACTIVE ZONE, Z_A)	$-1.0 \le \Delta pF \le -0.2$
	WETTING (BELOW THE MOVEMENT ACTIVE ZONE, Z_A)	$-1.0 \le \Delta pF \le -0.2$
	SWELLING (NEAR THE SURFACE)	$\Delta pF < -1.0$

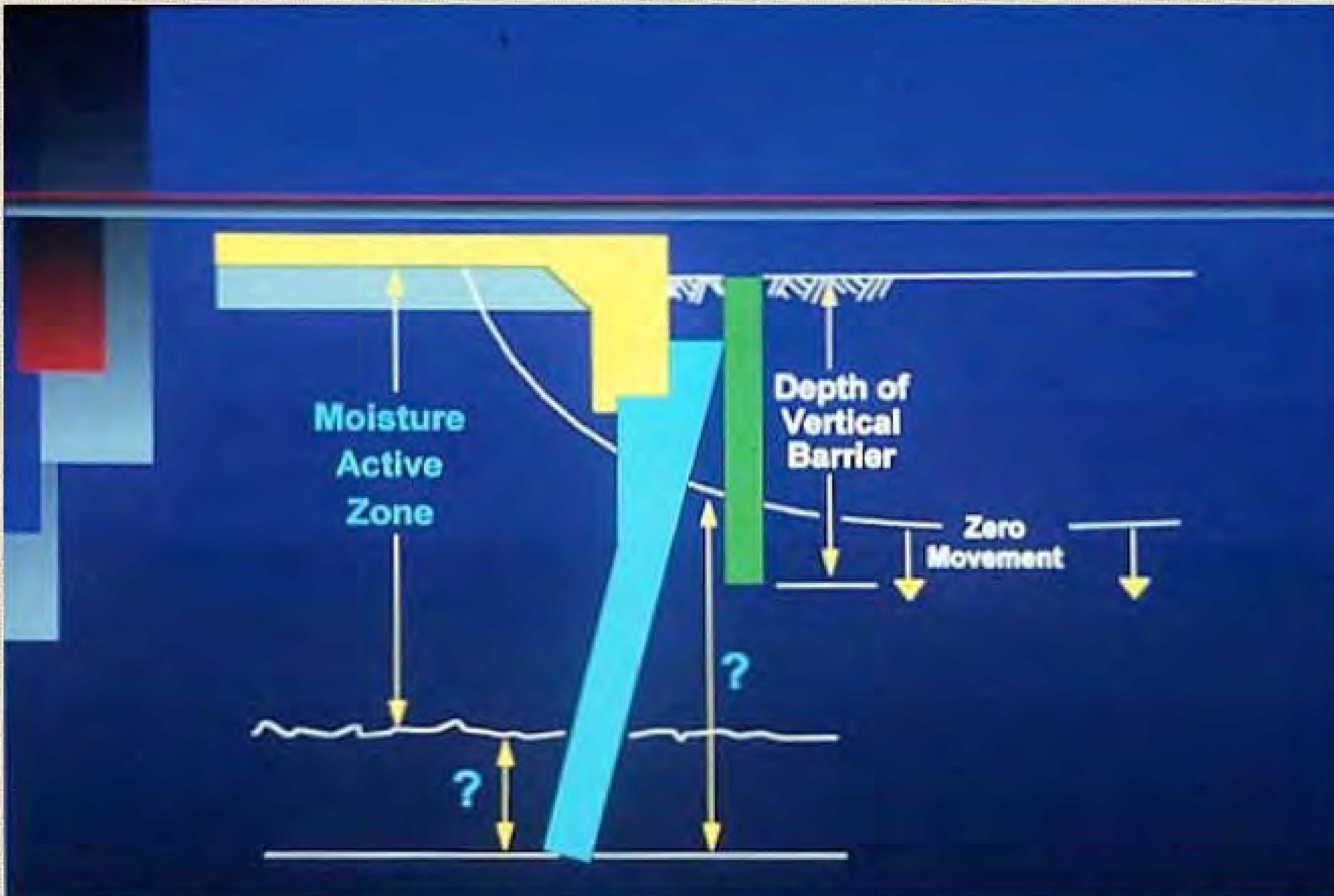
pF_o = SUCTION BELOW THE MOISTURE ACTIVE ZONE

HORIZONTAL EARTH PRESSURE COEFFICIENTS

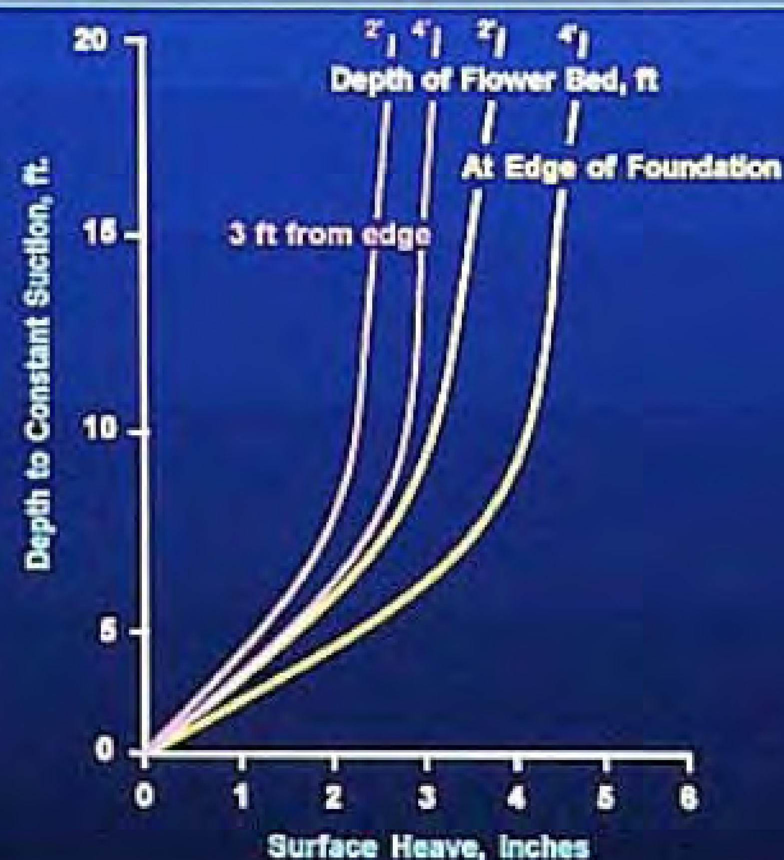
TYPICAL

K_o	d	k	n	c
0	0	0	1	0
1/3	0	0	1	1
1/2	1	0	1	1
2/3	1	1/2	1	1
1	1	1	1	1
3	1	1	2	1

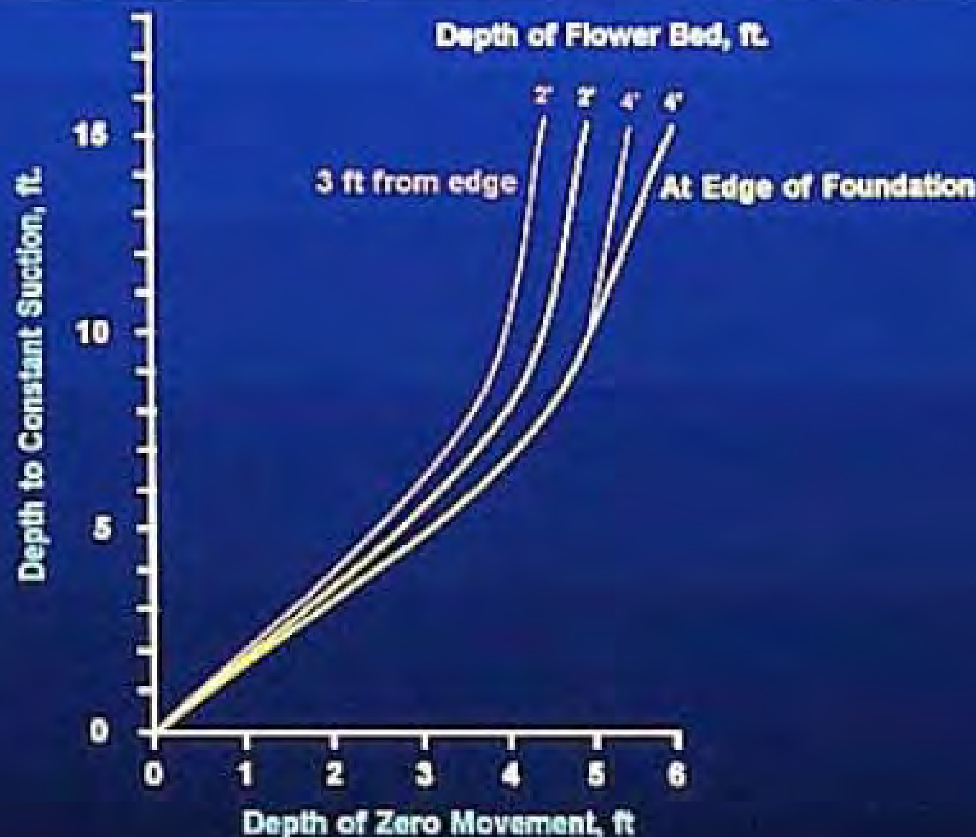

Moisture
Active
Zone
Zero
Movement
?
?

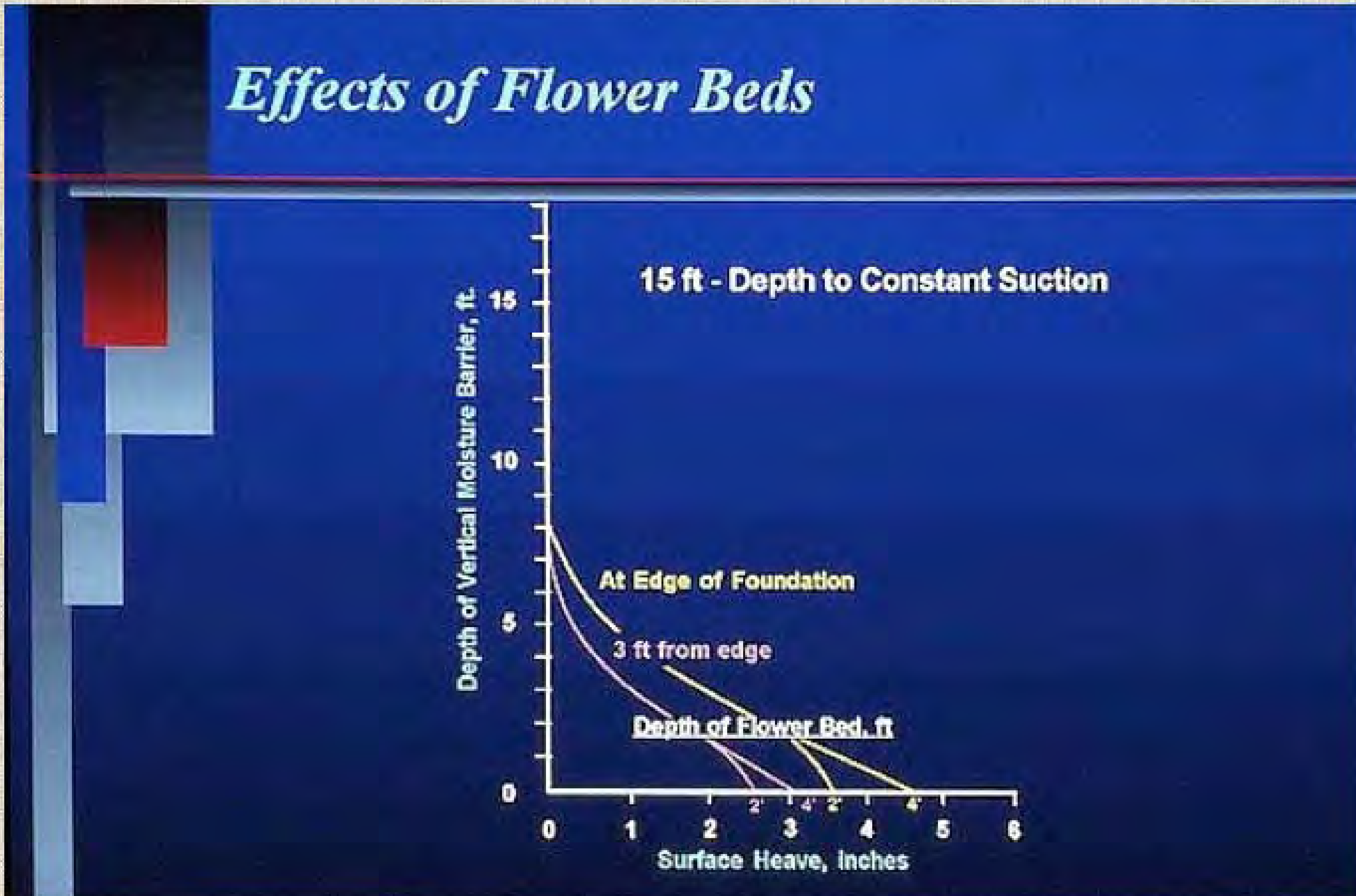

Moisture
Active
Zone
Depth of
Vertical
Barrier
Zero
Movement
?
?

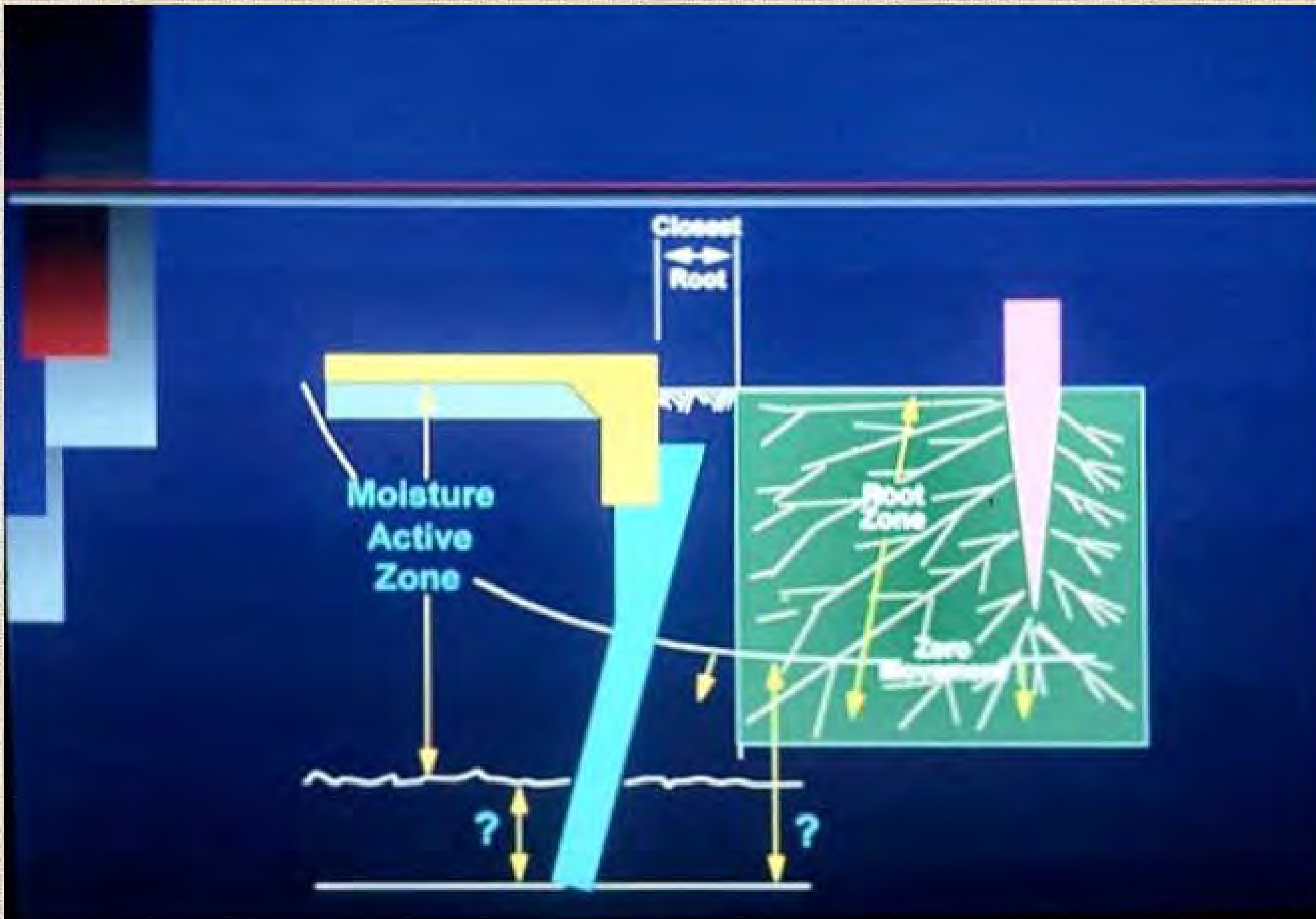
Effects of Flower Beds



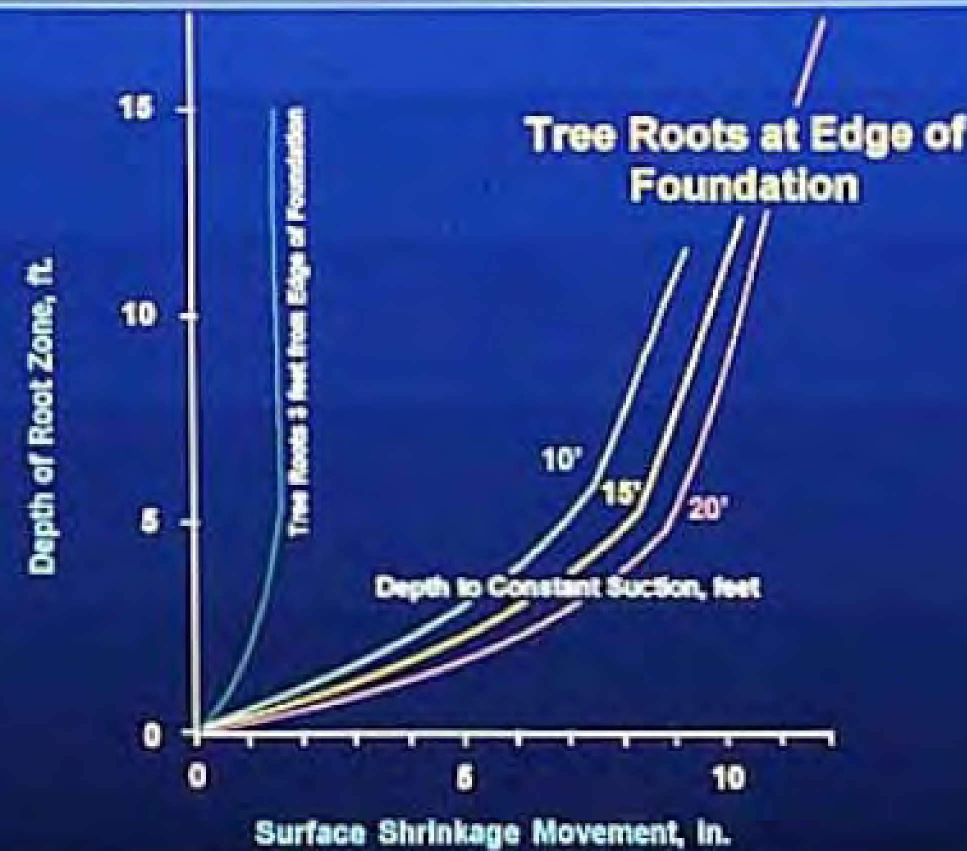
Effects of Flower Beds

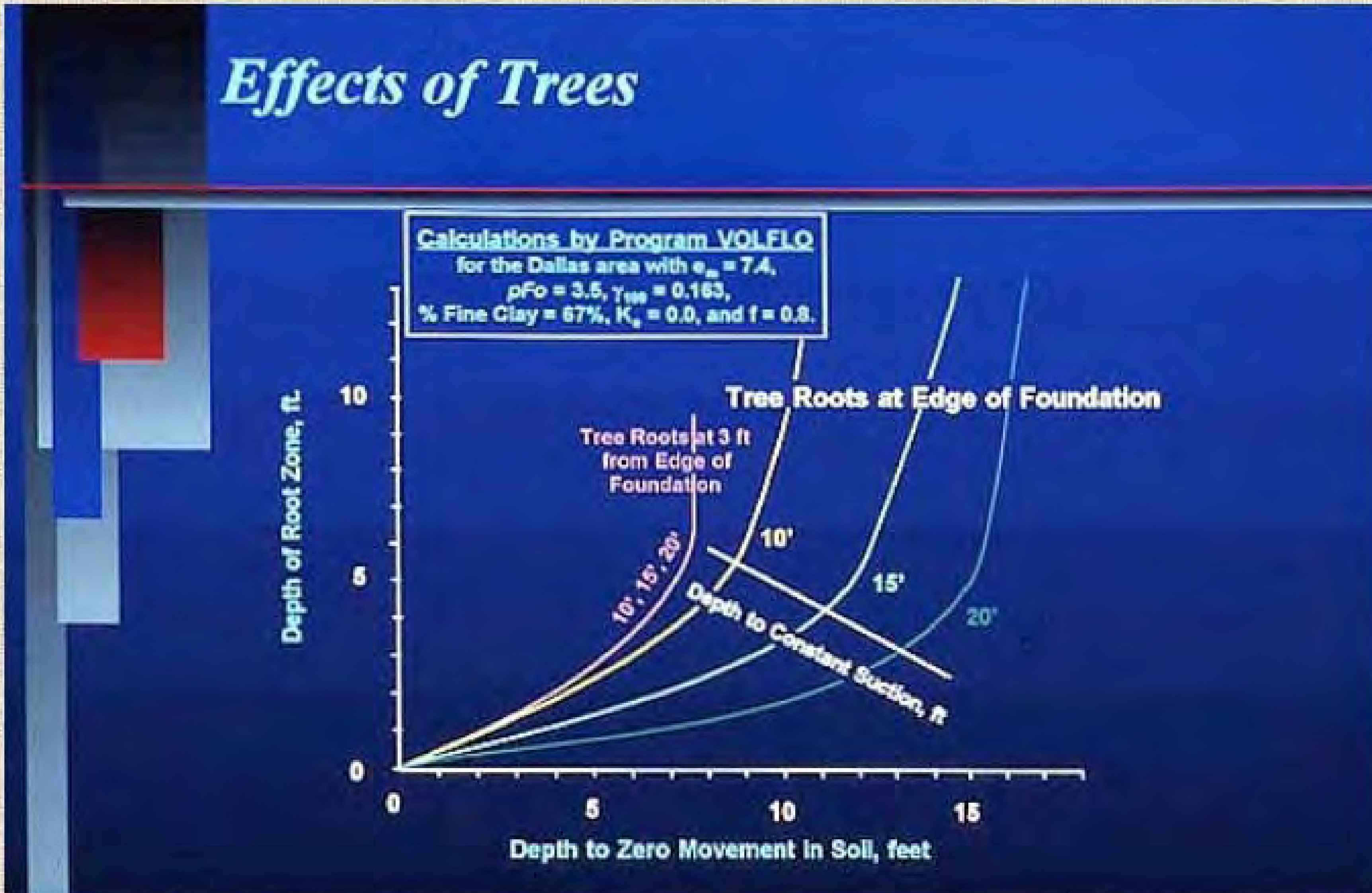



Effects of Flower Beds
15 ft - Depth to Constant Suction
Depth of Vertical Moisture Barrier, ft.
15
10
5
0
At Edge of Foundation
3 ft from edge
Depth of Flower Bed, ft
2'
4'
2'
4'
0
1
2
3
4
5
6
Surface Heave, Inches

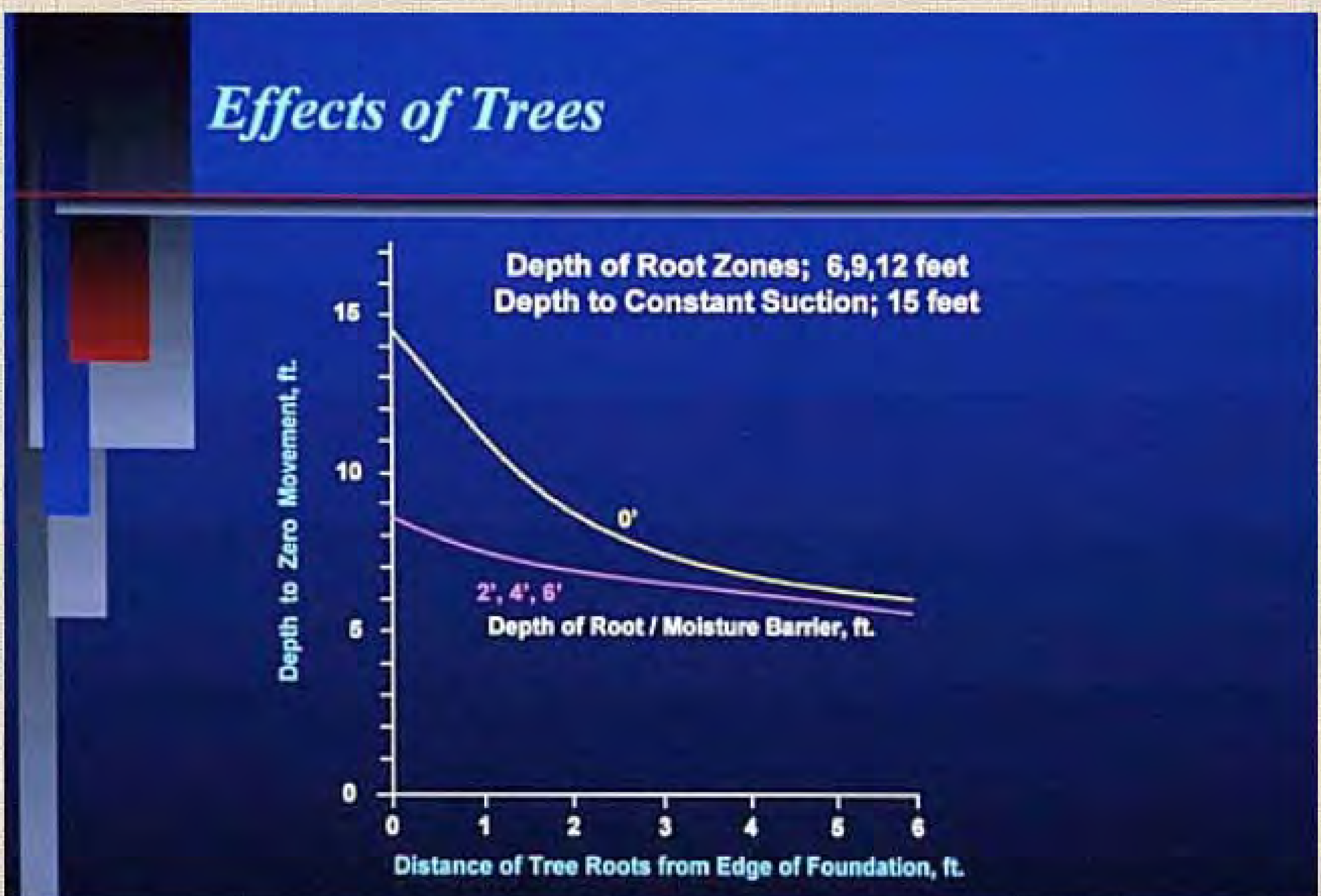

Closest
Root
Moisture
Active
Zone
Root
Zone
?
?

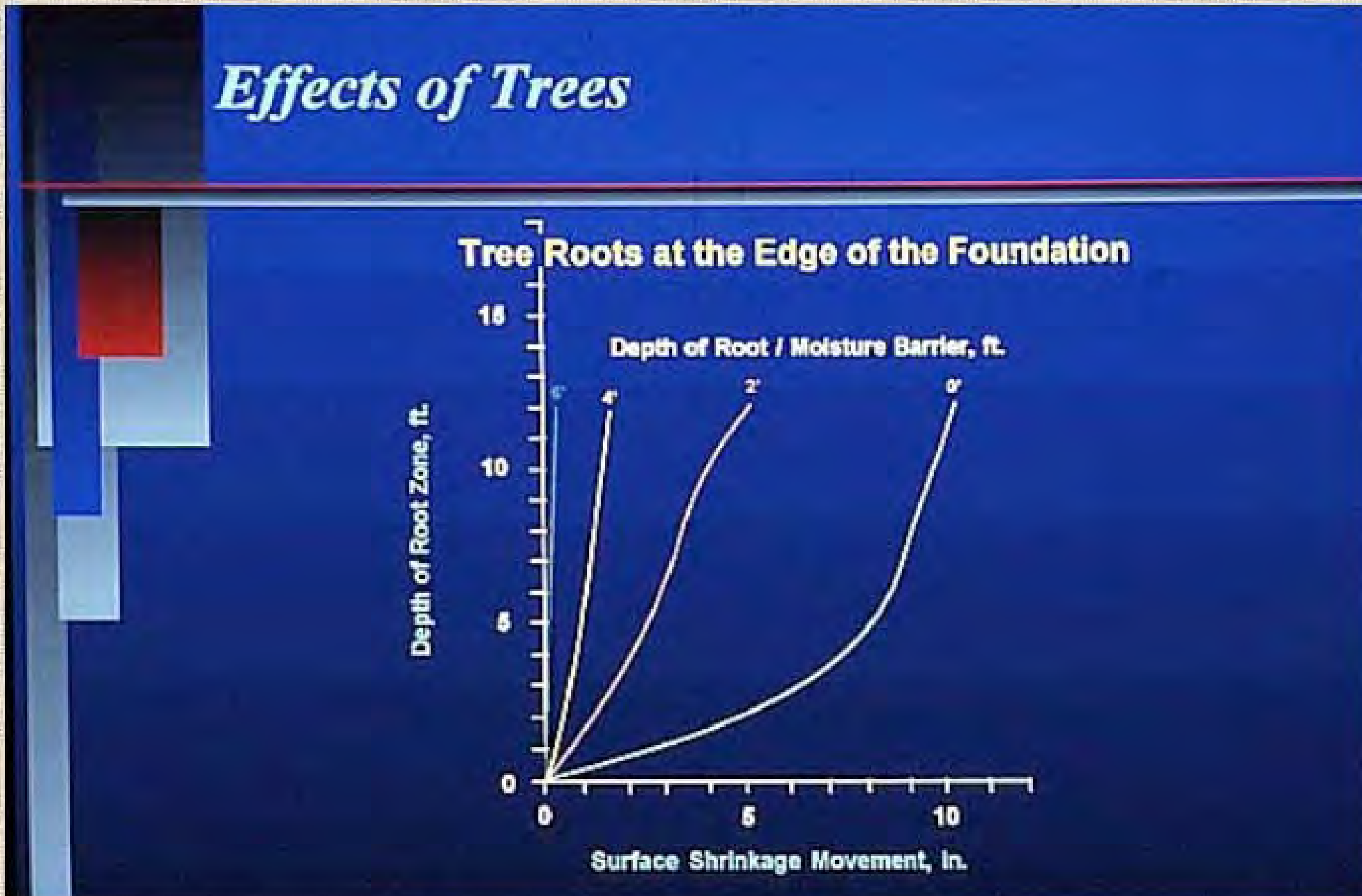
Effects of Trees



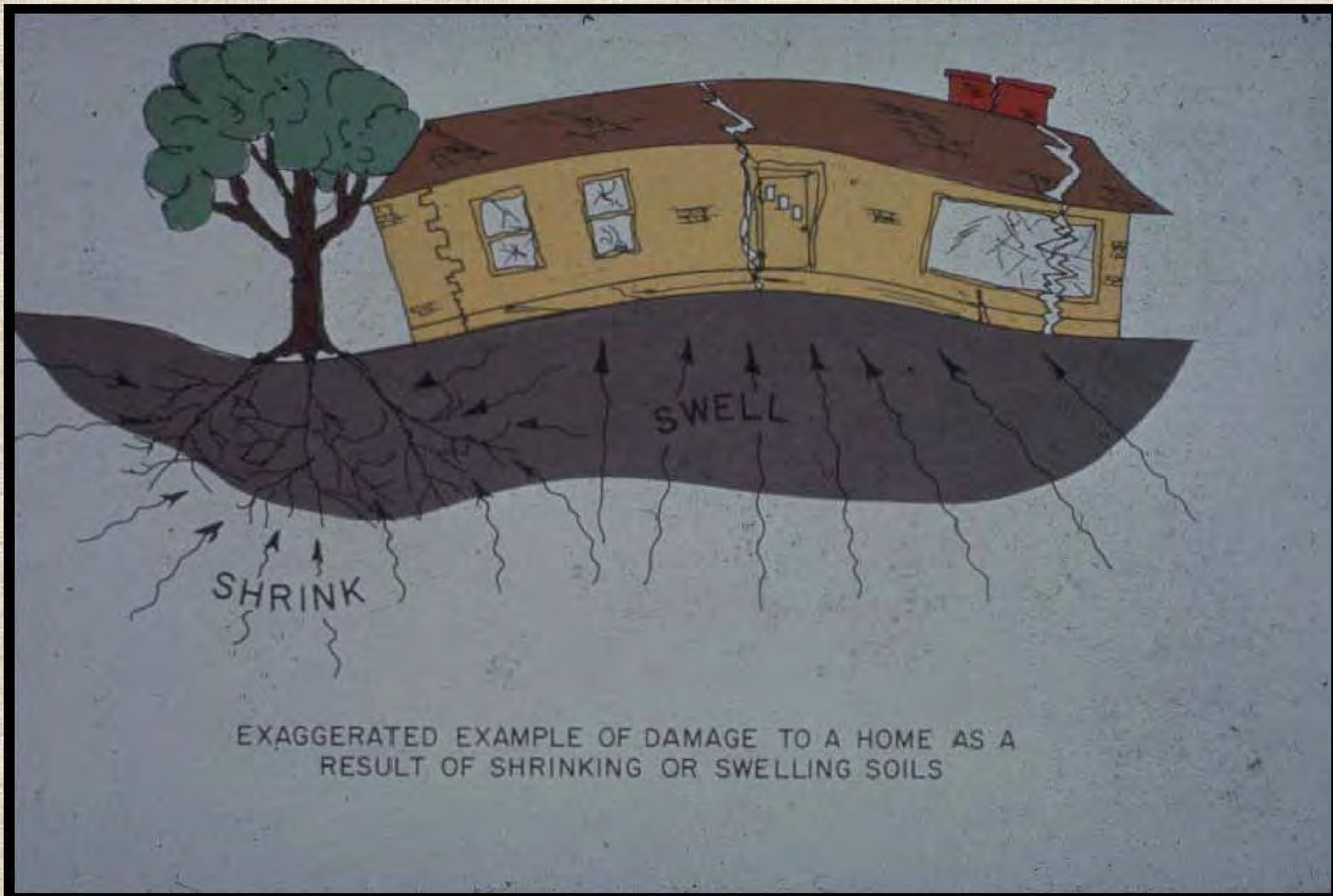


Effects of Trees
Calculations by Program VOLFLO
for the Dallas area with e = 7.4,
pFo = 3.5, γ = 0.163,
% Fine Clay = 67%, K = 0.0, and f = 0.8.
Tree Roots at Edge of Foundation
Tree Roots at 3 ft from Edge of Foundation
10', 15', 20'
10'
15'
20'
Depth to Constant Suction, ft
Depth of Root Zone, ft.
Depth to Zero Movement in Soil, feet
0
5
10
15

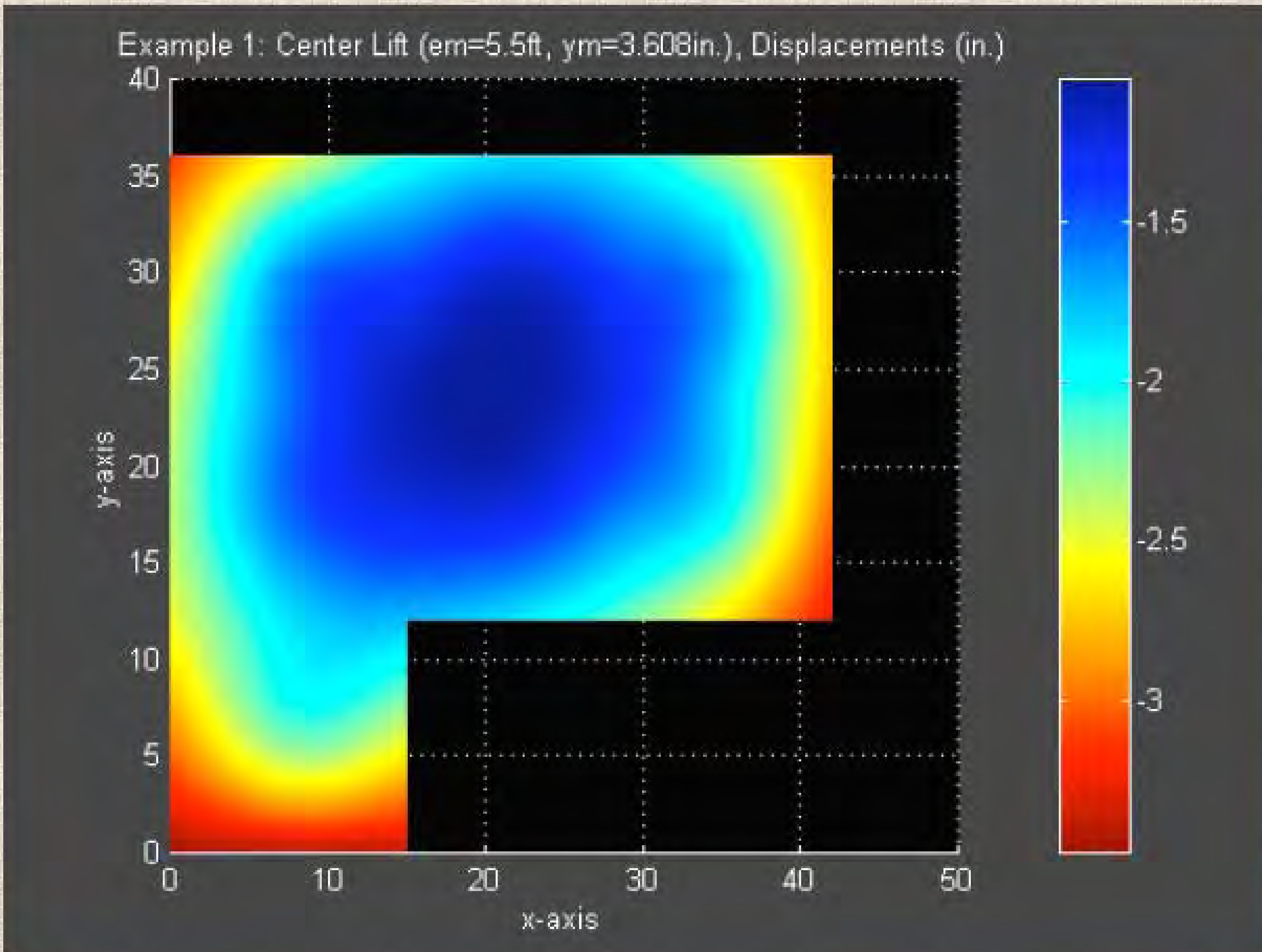

Effects of Trees
Depth of Root Zones; 6,9,12 feet
Depth to Constant Suction; 15 feet
Depth to Zero Movement, ft.
15
10
5
0
0'
2', 4', 6'
Depth of Root / Moisture Barrier, ft.
0
1
2
3
4
5
6
Distance of Tree Roots from Edge of Foundation, ft.

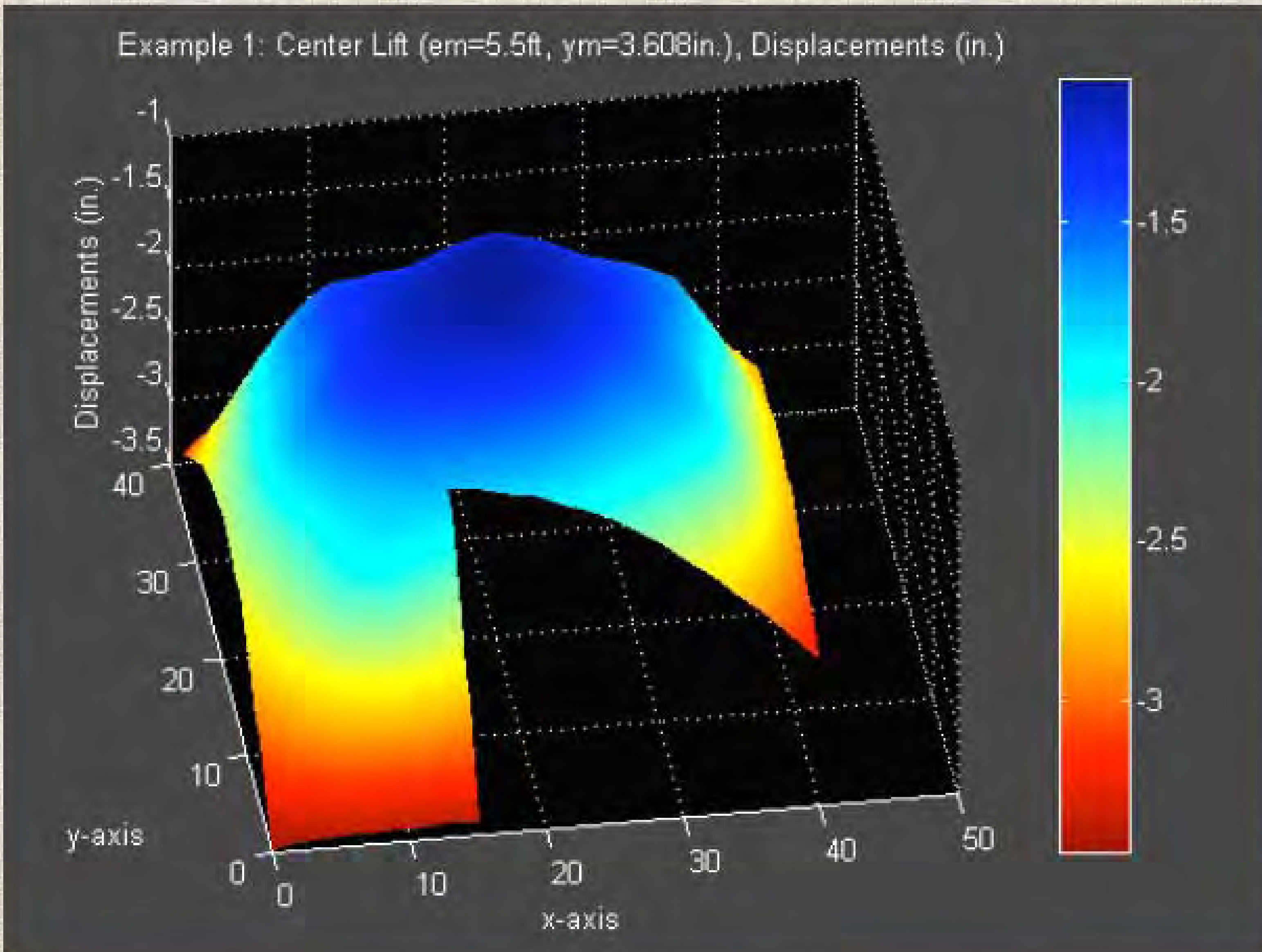

Effects of Trees
Tree Roots at the Edge of the Foundation
Depth of Root / Moisture Barrier, ft.
4'
2'
0'
Depth of Root Zone, ft.
15
10
5
0
0
5
10
Surface Shrinkage Movement, in.

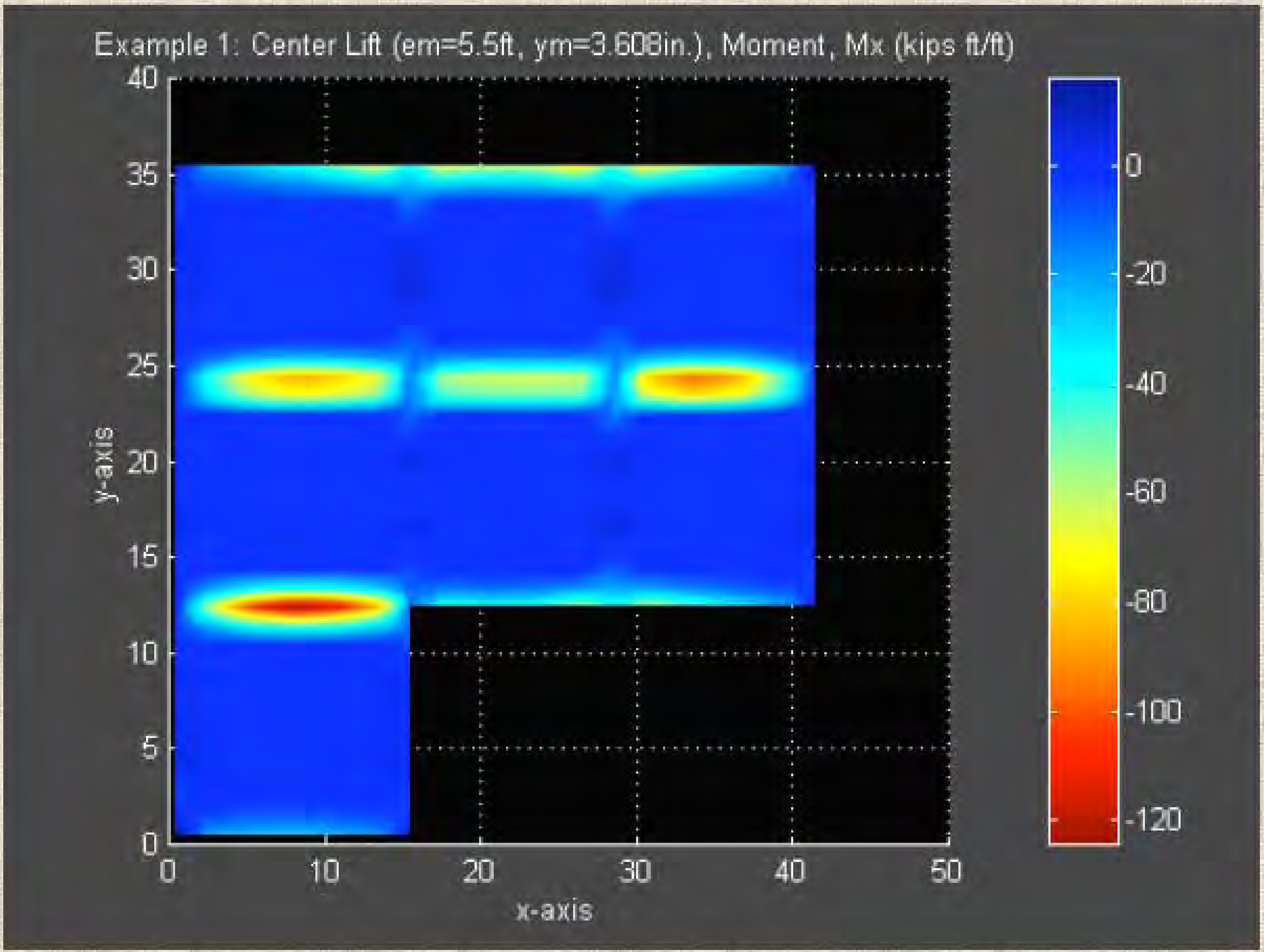
FLOATING SLABS

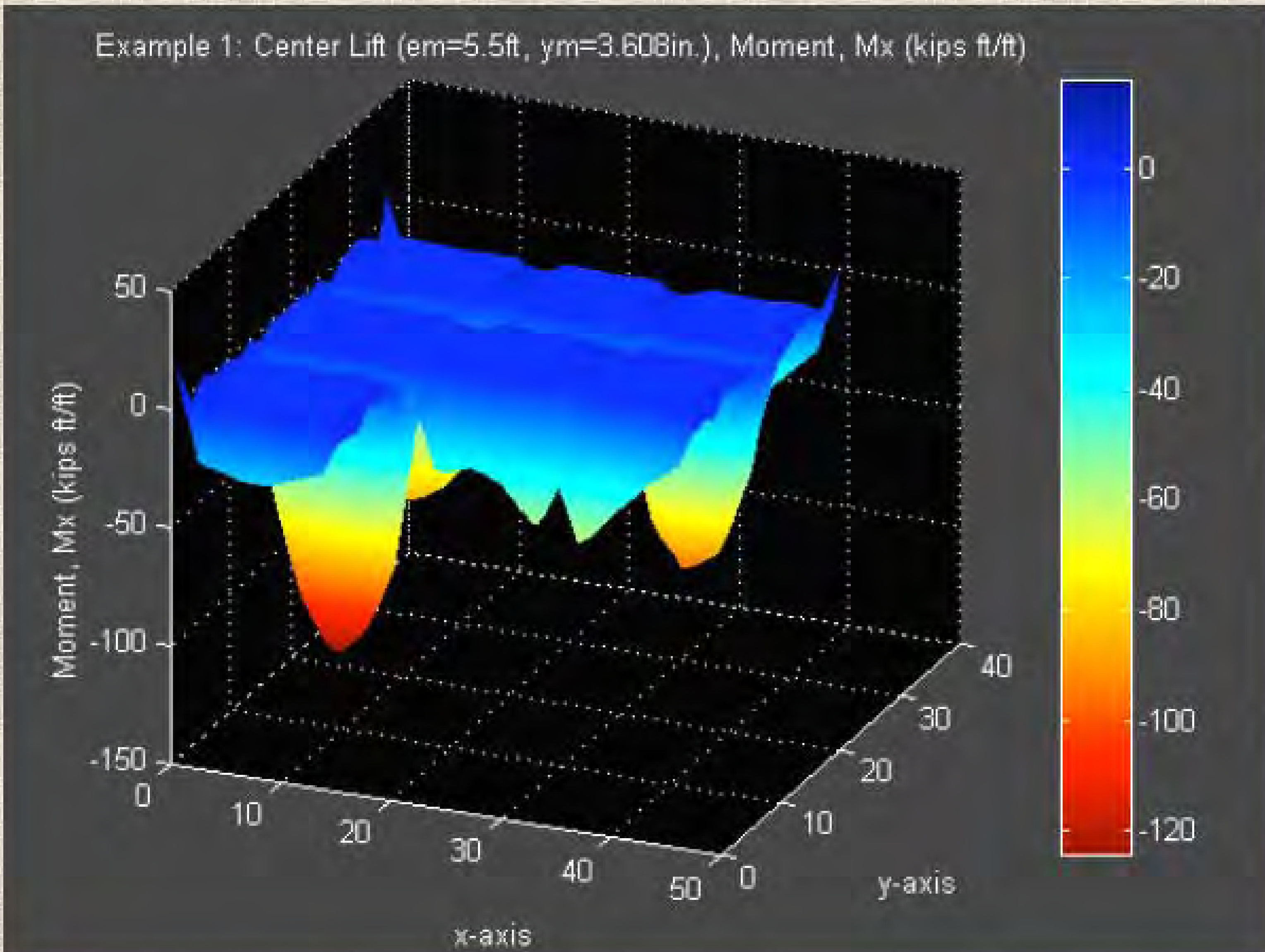

SWELL
SHRINK
EXAGGERATED EXAMPLE OF DAMAGE TO A HOME AS A
RESULT OF SHRINKING OR SWELLING SOILS

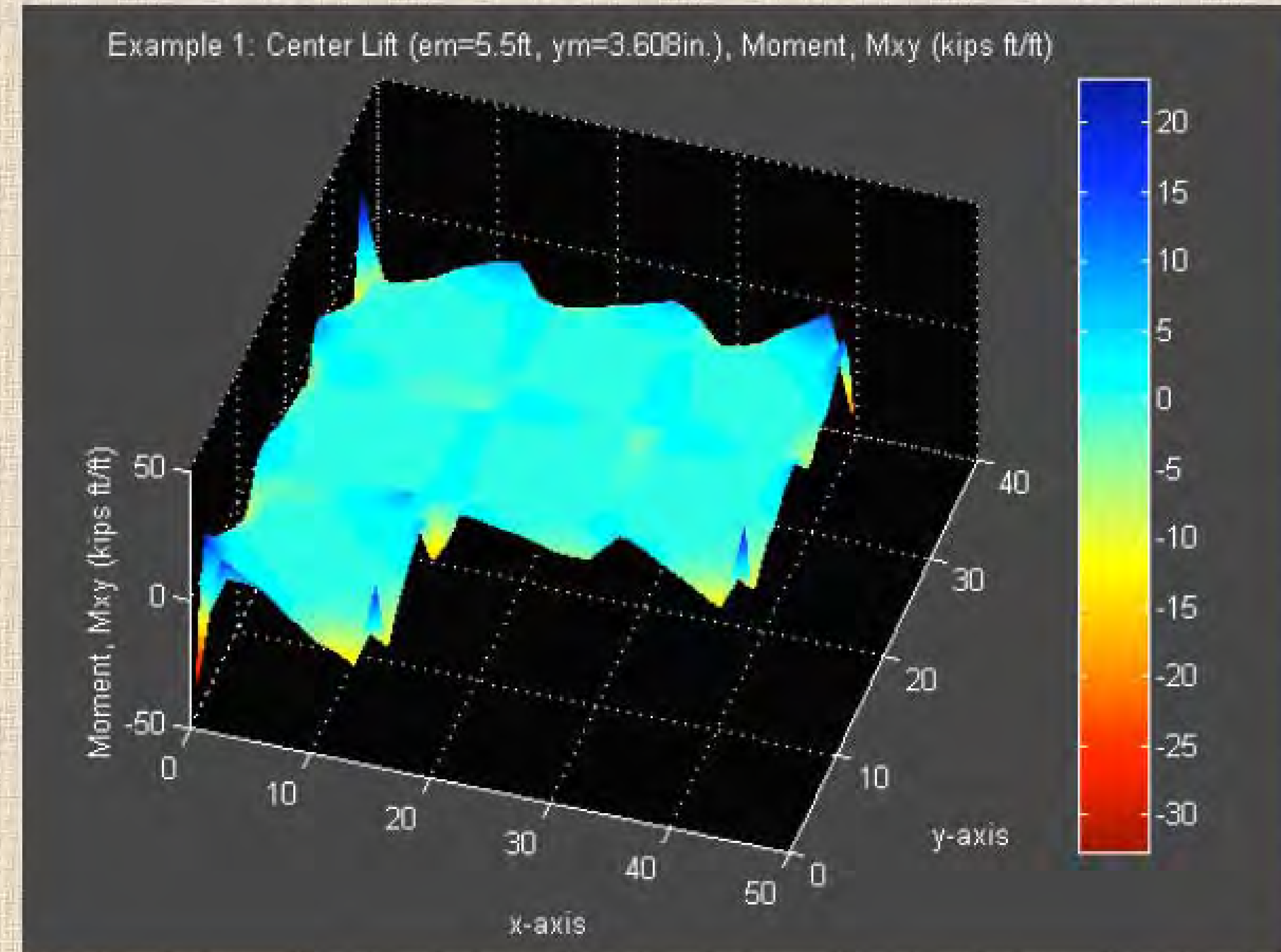

0.04
0.06
0.08
0.10
0.12
0.14
0.16
0.18
0.20
0.22
0.24
0.26
0.28
0.30
TYPICAL CONTOURS FOR RESIDENTIAL SLAB EXPERIENCING CENTER LIFT OR EDGE DRYING CONDITIONS

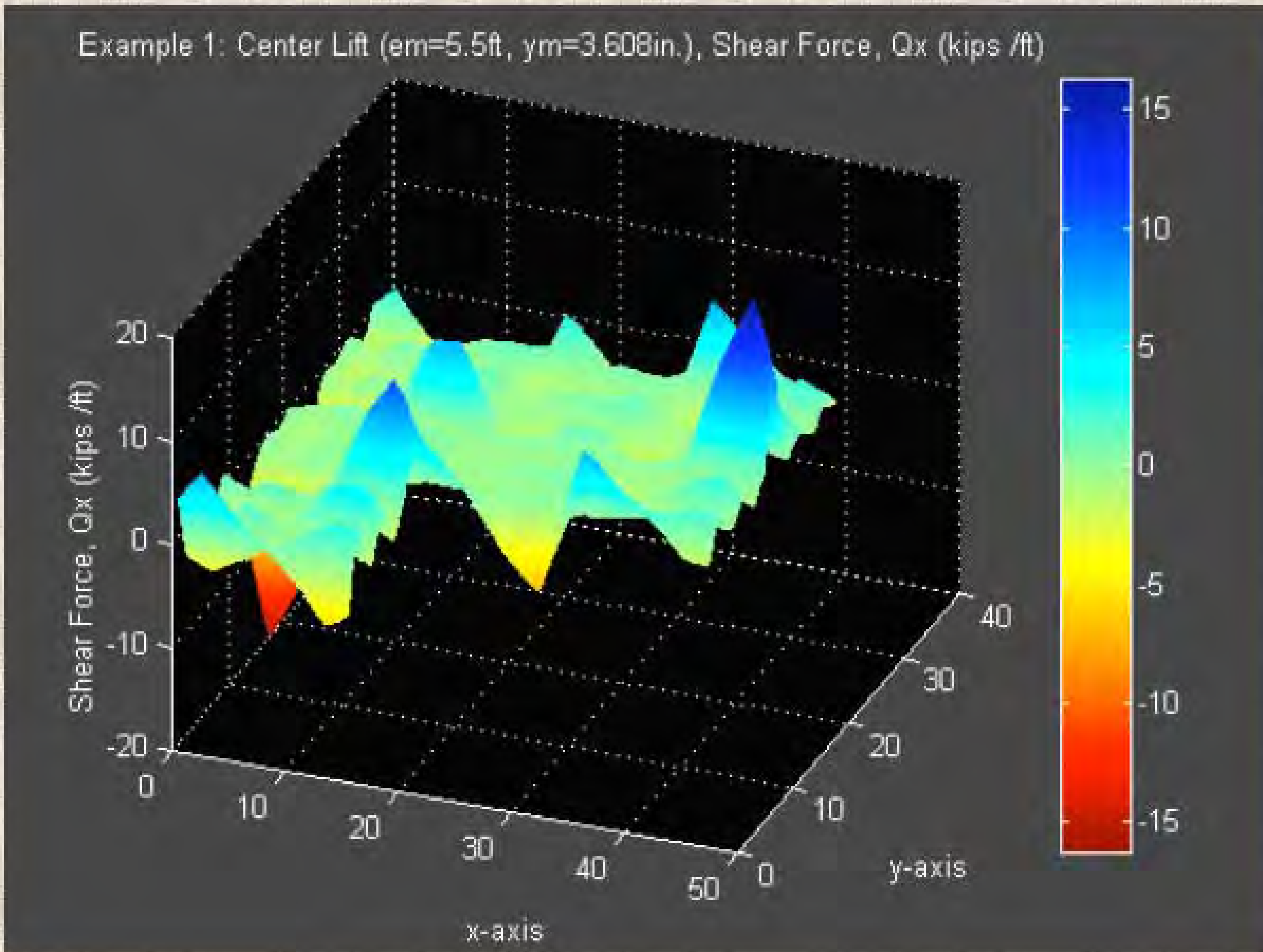

Example 1: Center Lift (em=5.5ft, ym=3.608in.), Displacements (in.)
y-axis
x-axis
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
-1.5
-2
-2.5
-3

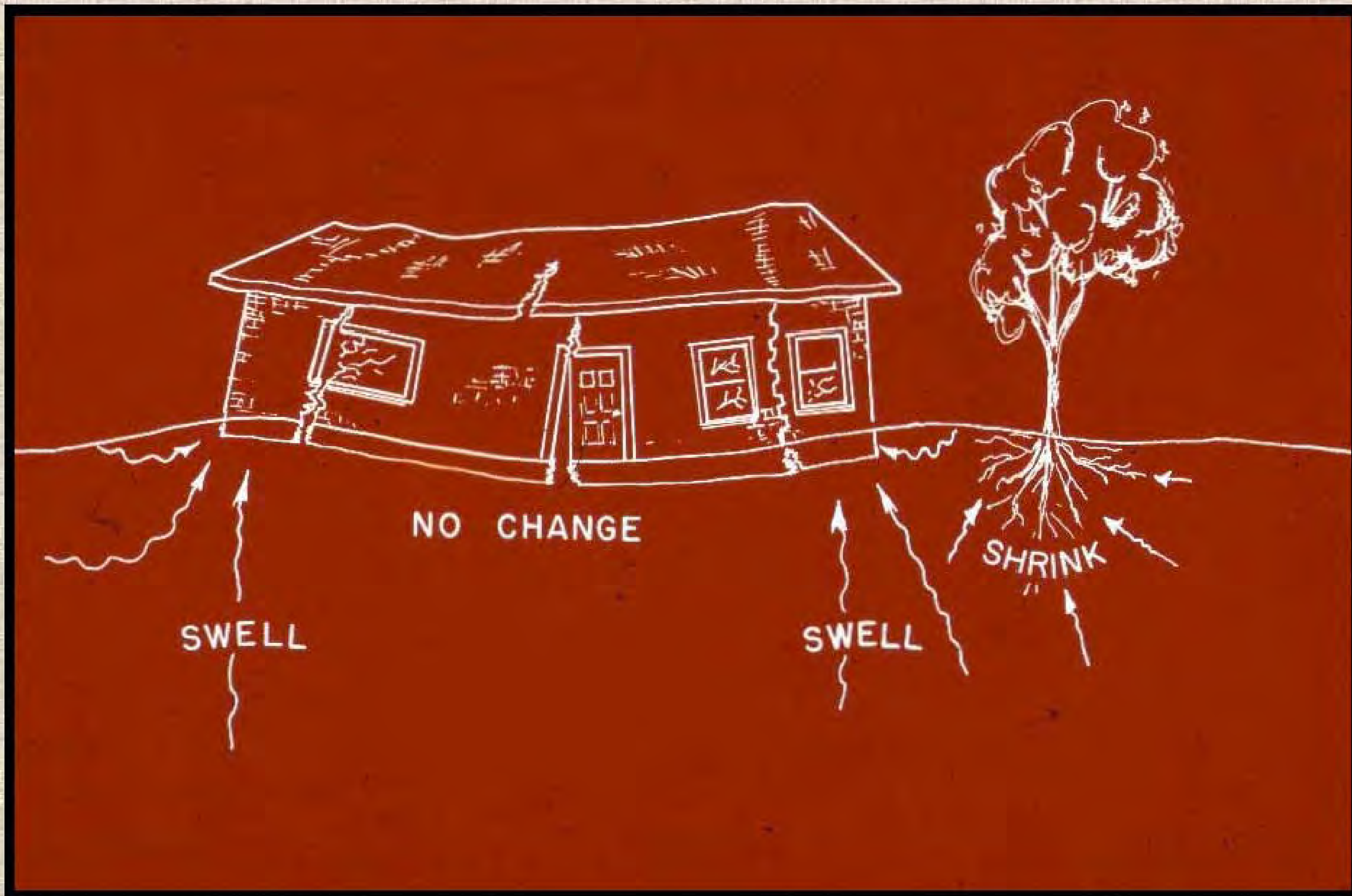

Example 1: Center Lift (em=5.5ft, ym=3.608in.), Displacements (in.)
Displacements (in.)
-1
-1.5
-2
-2.5
-3
-3.5
40
30
20
10
0
y-axis
0
10
20
30
40
50
x-axis
-1.5
-2
-2.5
-3

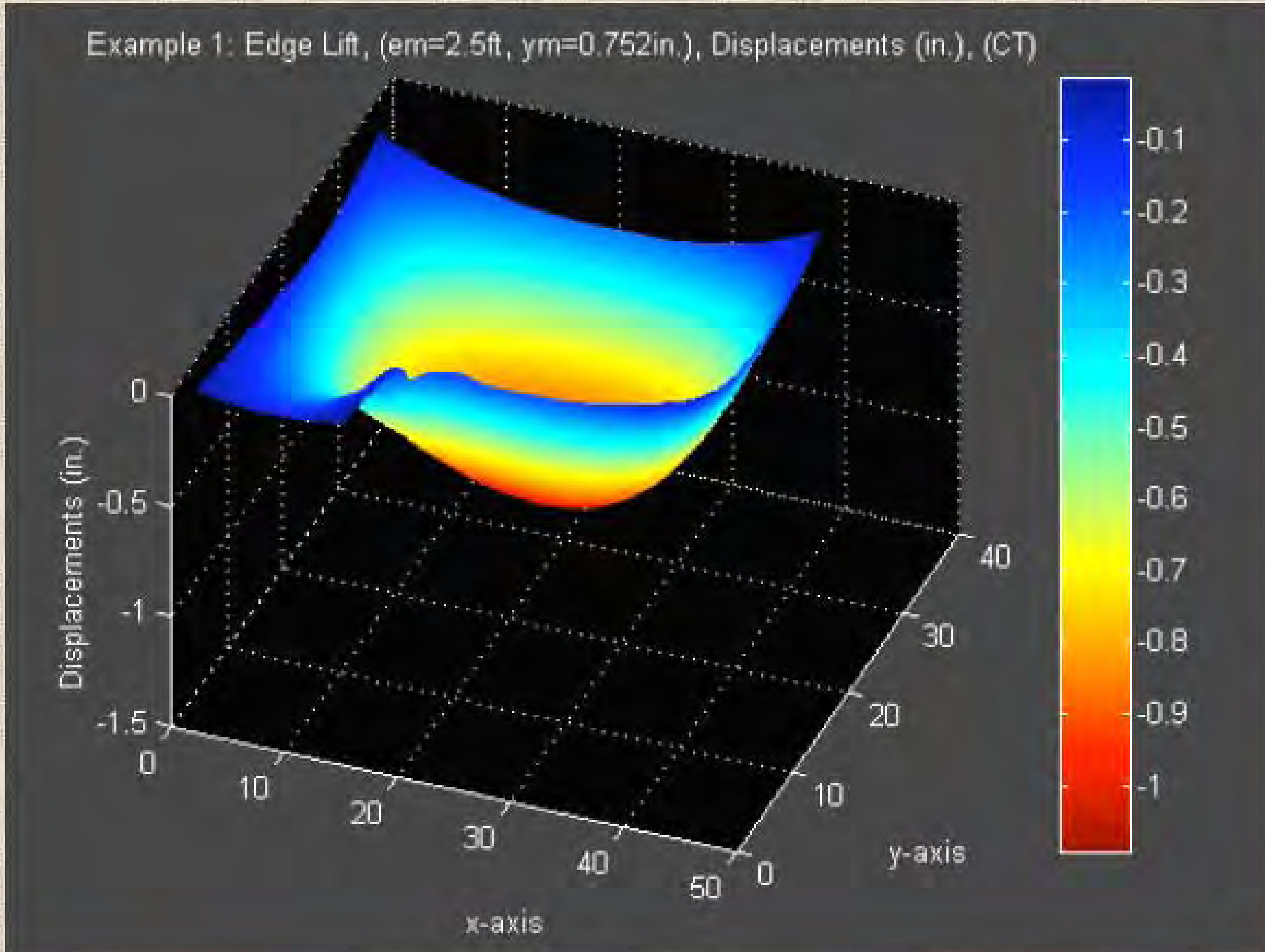

Example 1: Center Lift (em=5.5ft, ym=3.608in.), Moment, Mx (kips ft/ft)
40
35
30
25
20
15
10
5
0
y-axis
0
10
20
30
40
50
x-axis
0
-20
-40
-60
-80
-100
-120


Example 1: Center Lift (em=5.5ft, ym=3.608in.), Moment, Mx (kips ft/ft)
Moment, Mx (kips ft/ft)
50
0
-50
-100
-150
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
0
-20
-40
-60
-80
-100
-120

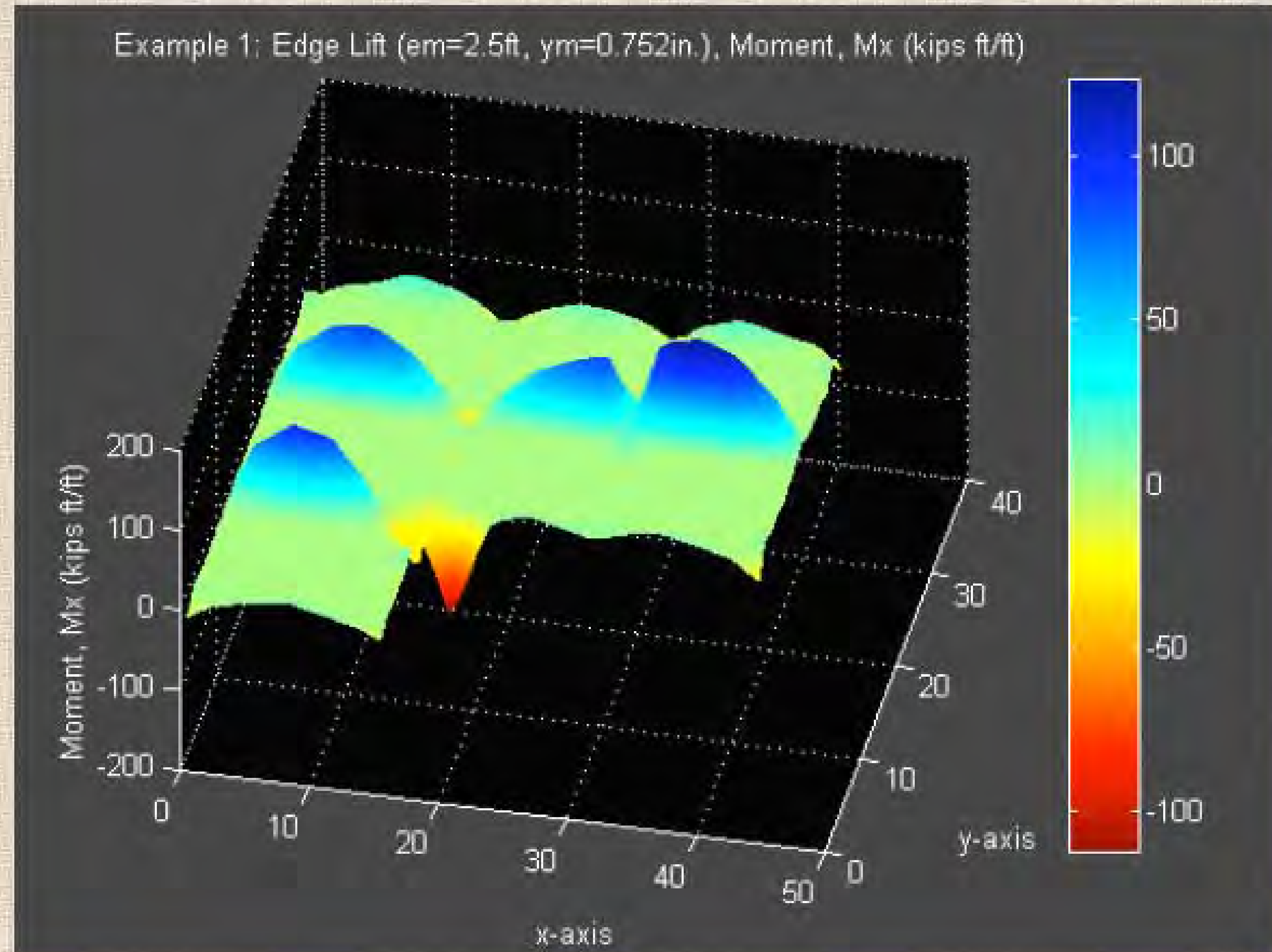

Example 1: Center Lift (em=5.5ft, ym=3.608in.), Moment, Mxy (kips ft/ft)
Moment, Mxy (kips ft/ft)
50
0
-50
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
20
15
10
5
0
-5
-10
-15
-20
-25
-30


Example 1: Center Lift (em=5.5ft, ym=3.608in.), Shear Force, Qx (kips /ft)
Shear Force, Qx (kips /ft)
20
10
0
-10
-20
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
15
10
5
0
-5
-10
-15

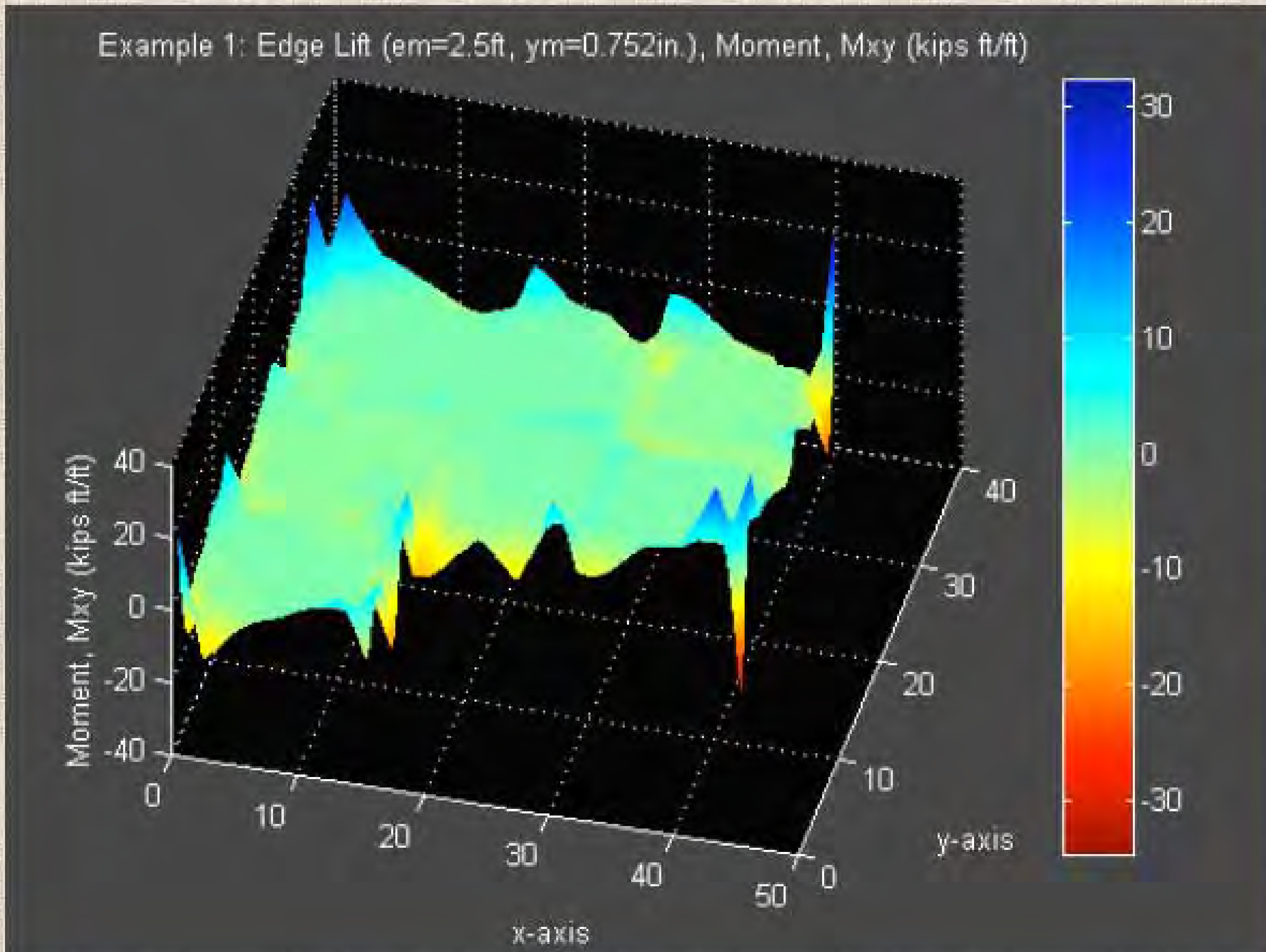

NO CHANGE
SWELL
SWELL
SHRINK

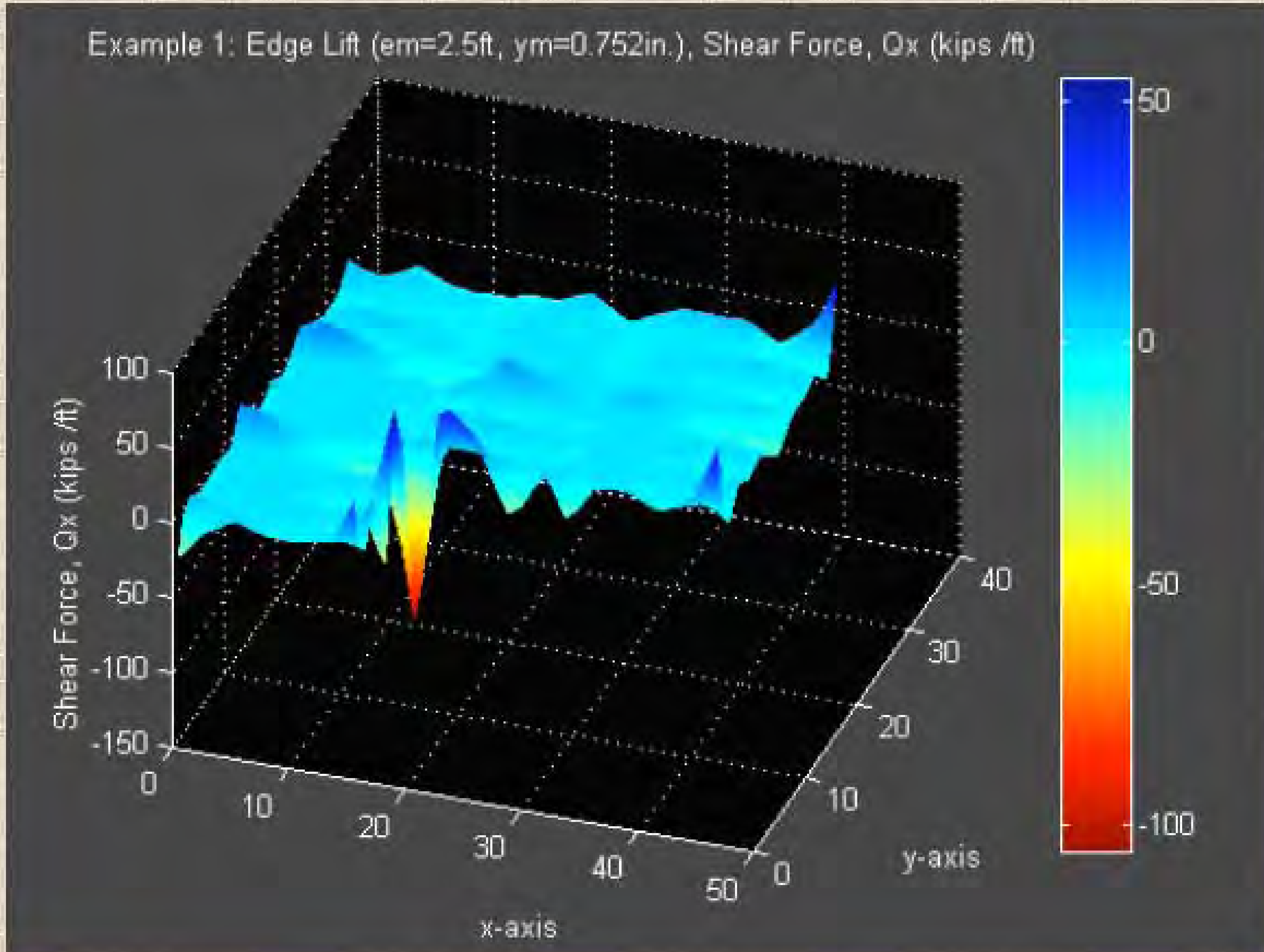


Example 1: Edge Lift, (em=2.5ft, ym=0.752in.), Displacements (in.), (CT)
Displacements (in.)
0
-0.5
-1
-1.5
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1



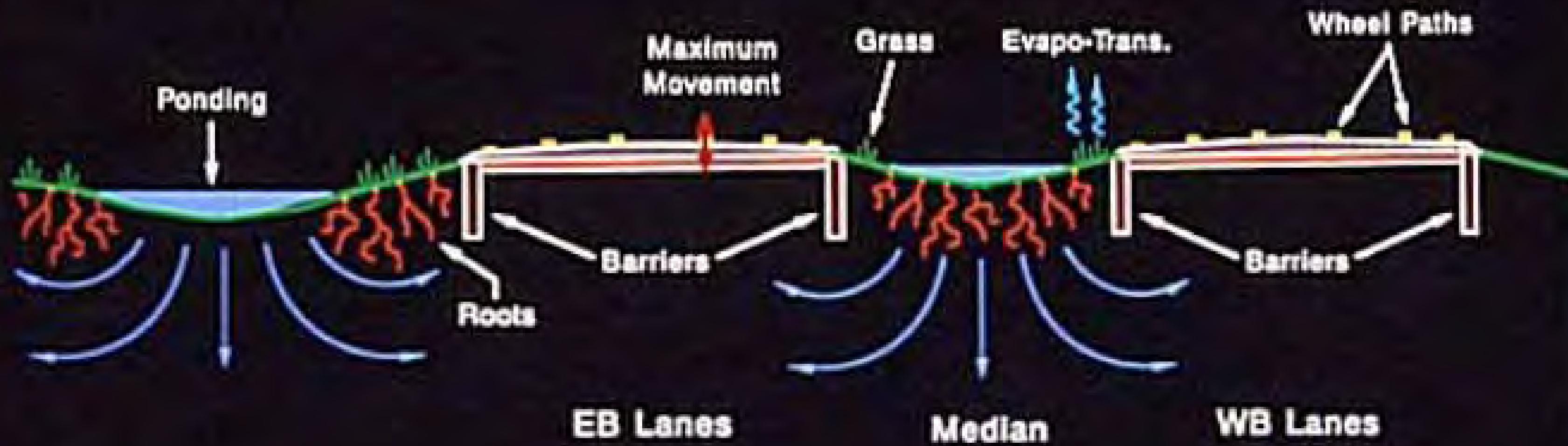
Example 1: Edge Lift (em=2.5ft, ym=0.752in.), Moment, Mx (kips ft/ft)
Moment, Mx (kips ft/ft)
200
100
0
-100
-200
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
100
50
0
-50
-100

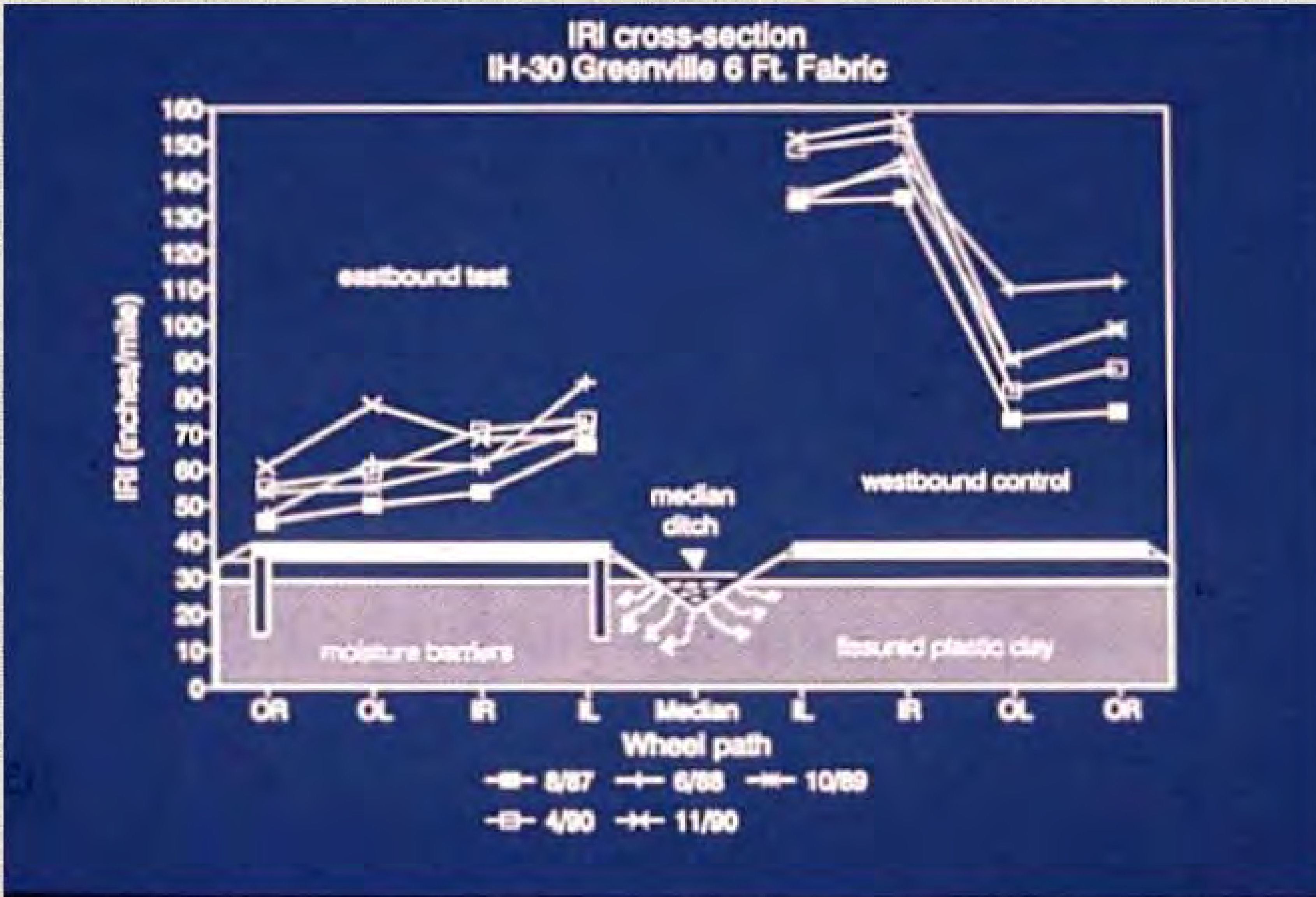

Example 1: Edge Lift (em=2.5ft, ym=0.752in.), Moment, Mxy (kips ft/ft)
Moment, Mxy (kips ft/ft)
40
20
0
-20
-40
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
30
20
10
0
-10
-20
-30

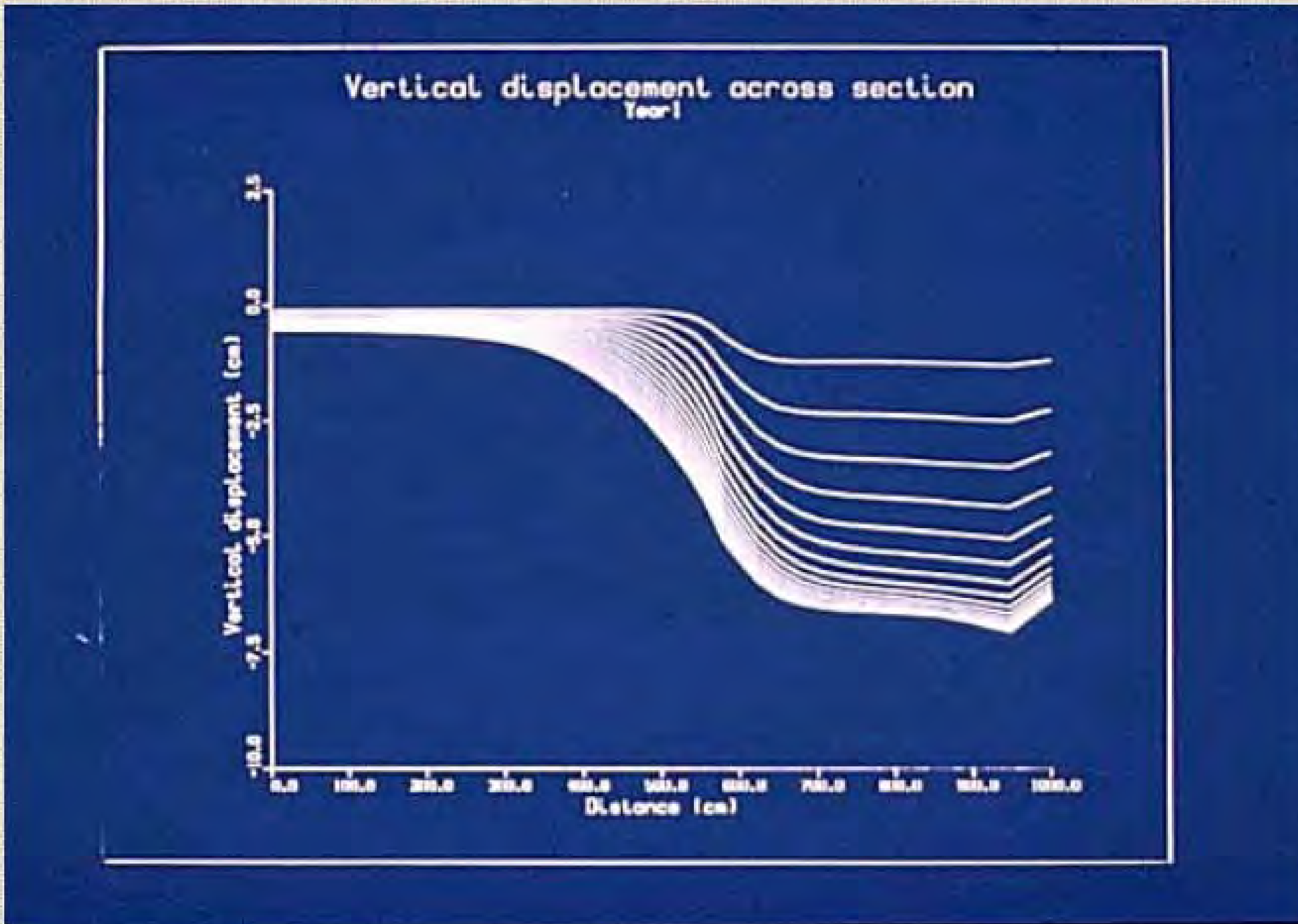


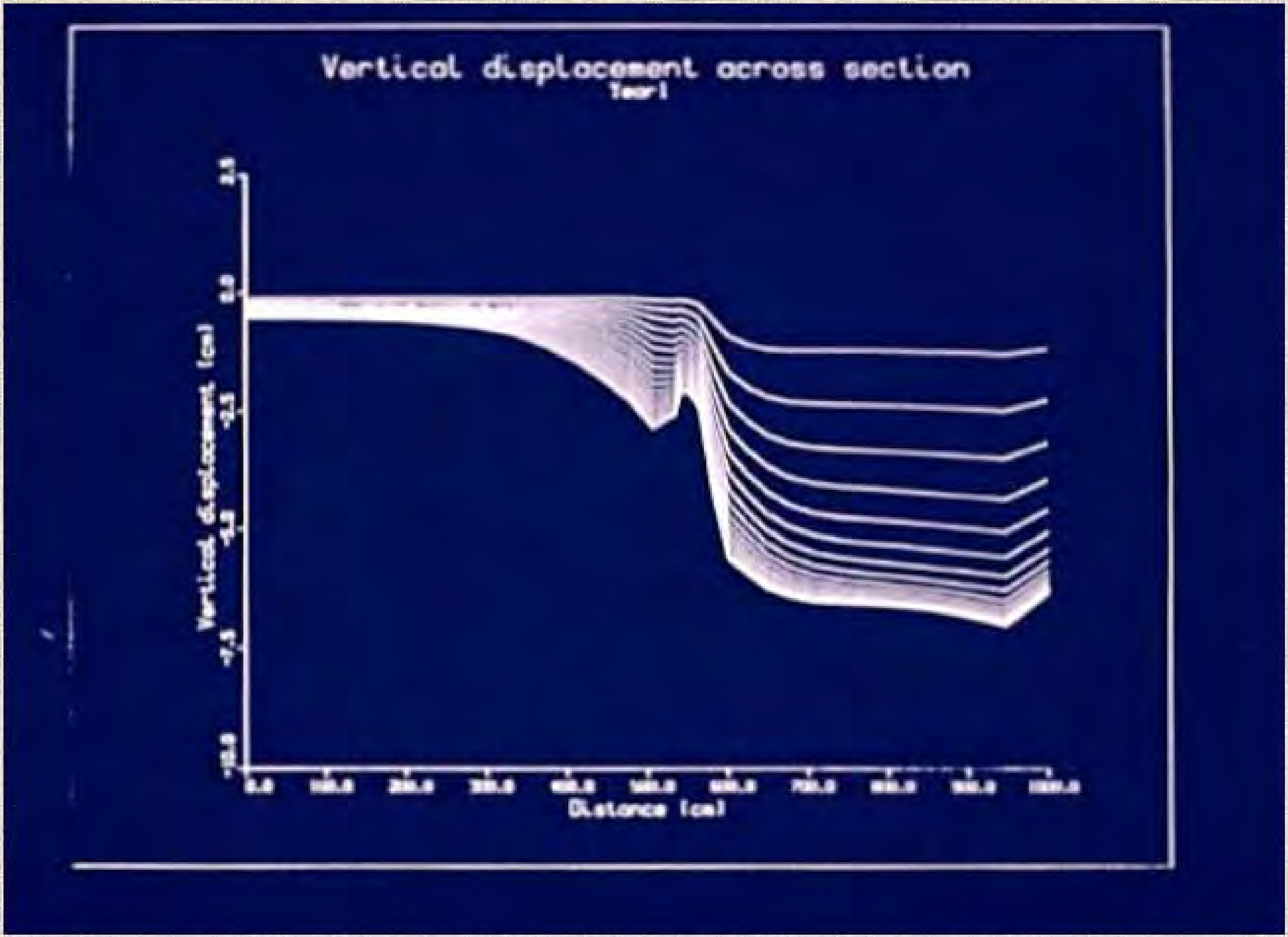
Example 1: Edge Lift (em=2.5ft, ym=0.752in.), Shear Force, Qx (kips /ft)
Shear Force, Qx (kips /ft)
100
50
0
-50
-100
-150
0
10
20
30
40
50
x-axis
0
10
20
30
40
y-axis
50
0
-50
-100

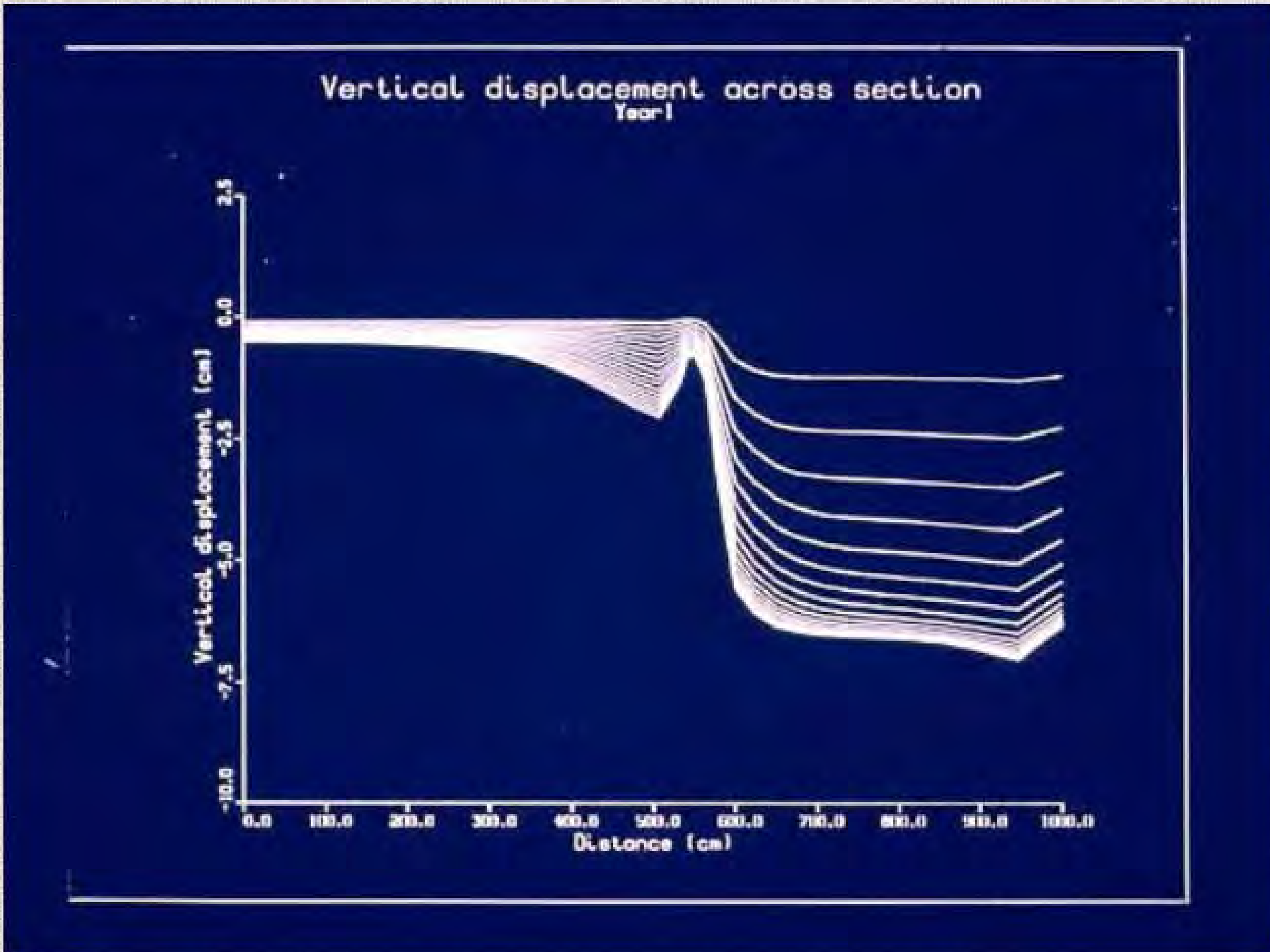
MONITORING
ANALYSIS OF CROSS SECTION



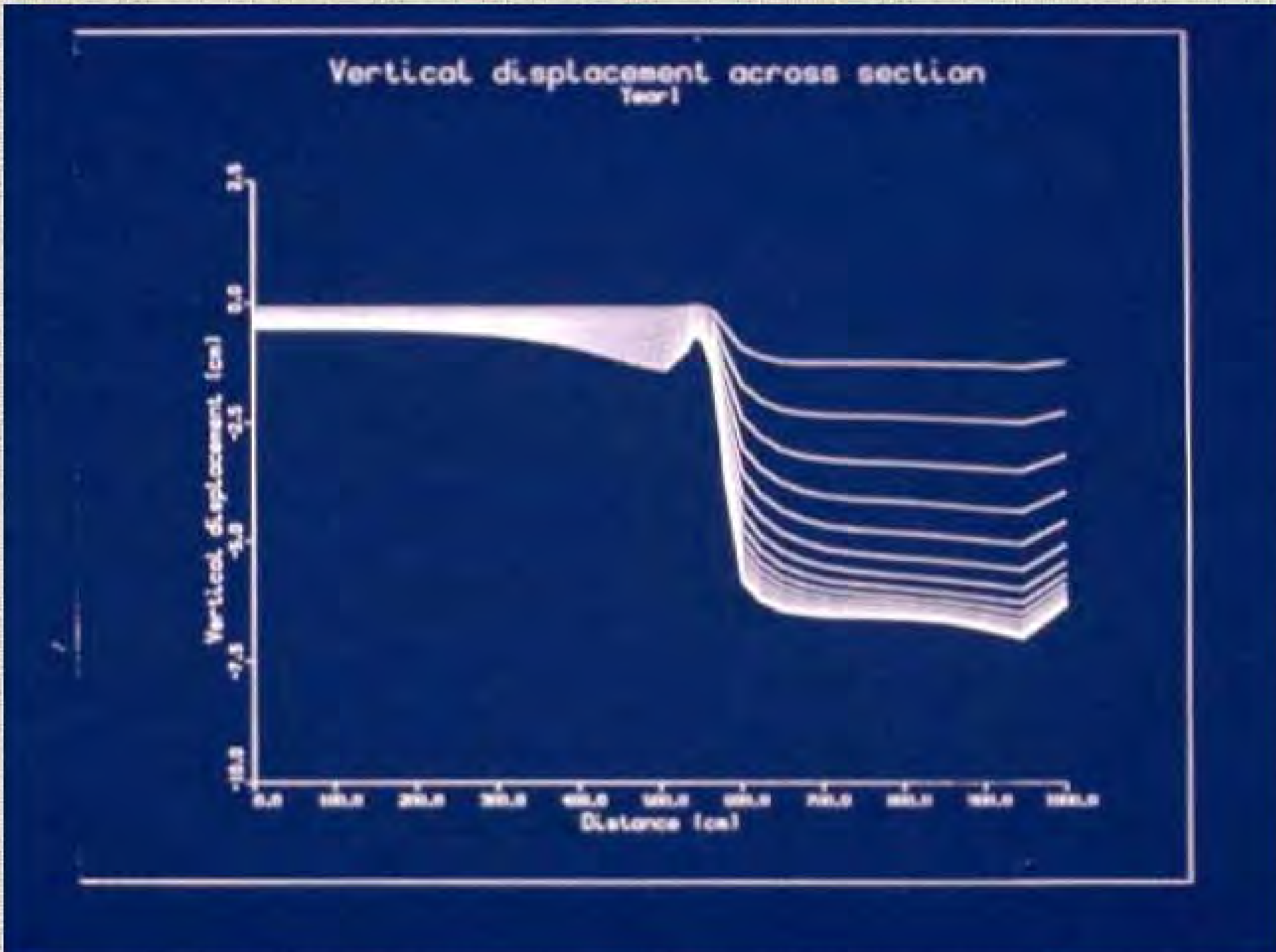

IRI cross-section
IH-30 Greenville 6 Ft. Fabric
IRI (inches/mile)
eastbound test
westbound control
median ditch
moisture barriers
fissured plastic clay
OR
OL
IR
IL
Median
IL
IR
OL
OR
Wheel path
8/87
6/88
10/89
4/90
11/90

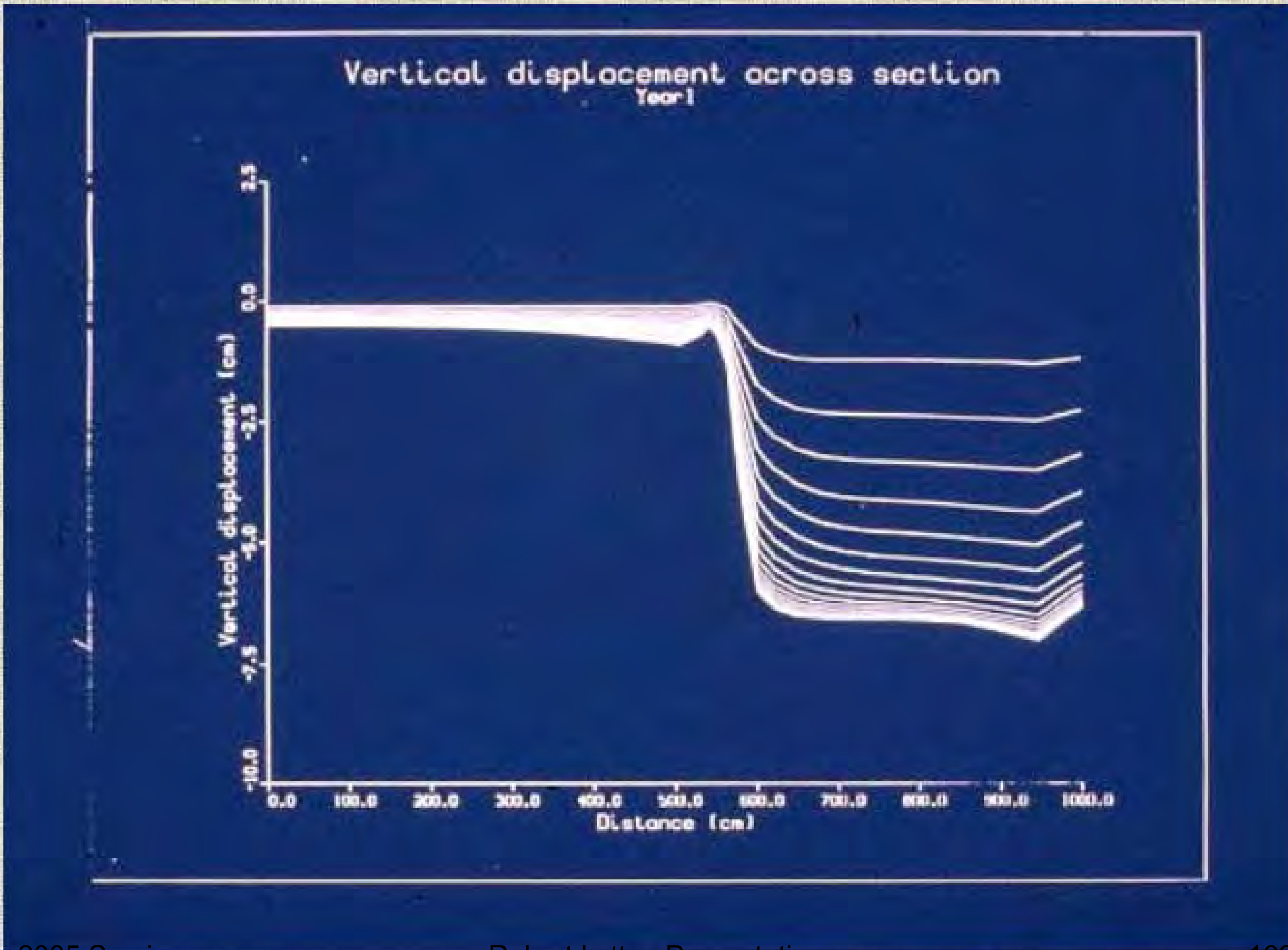

Vertical displacement across section
Vertical displacement (cm)
Distance (cm)

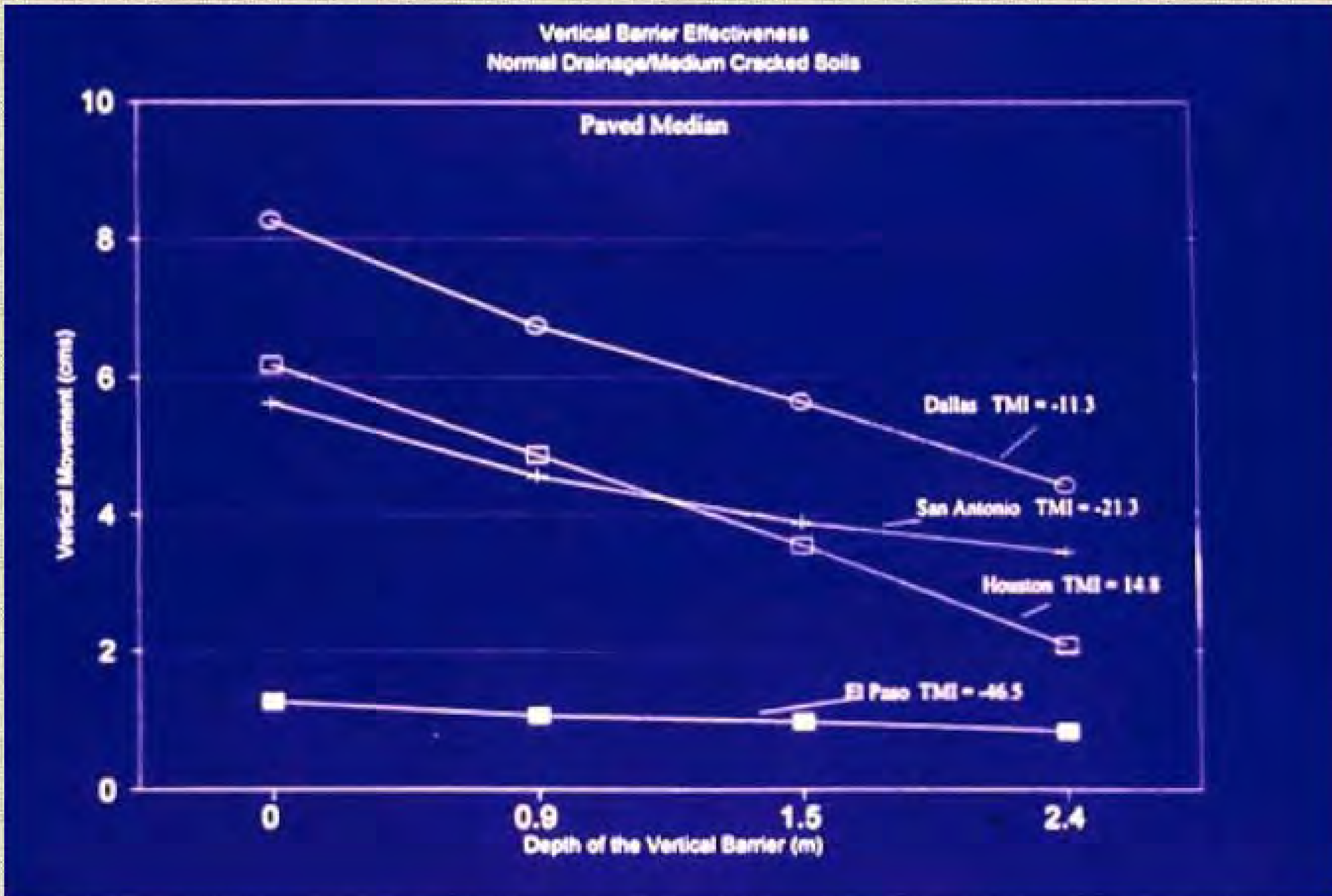

Vertical displacement across section
Vertical displacement (cm)
Distance (cm)

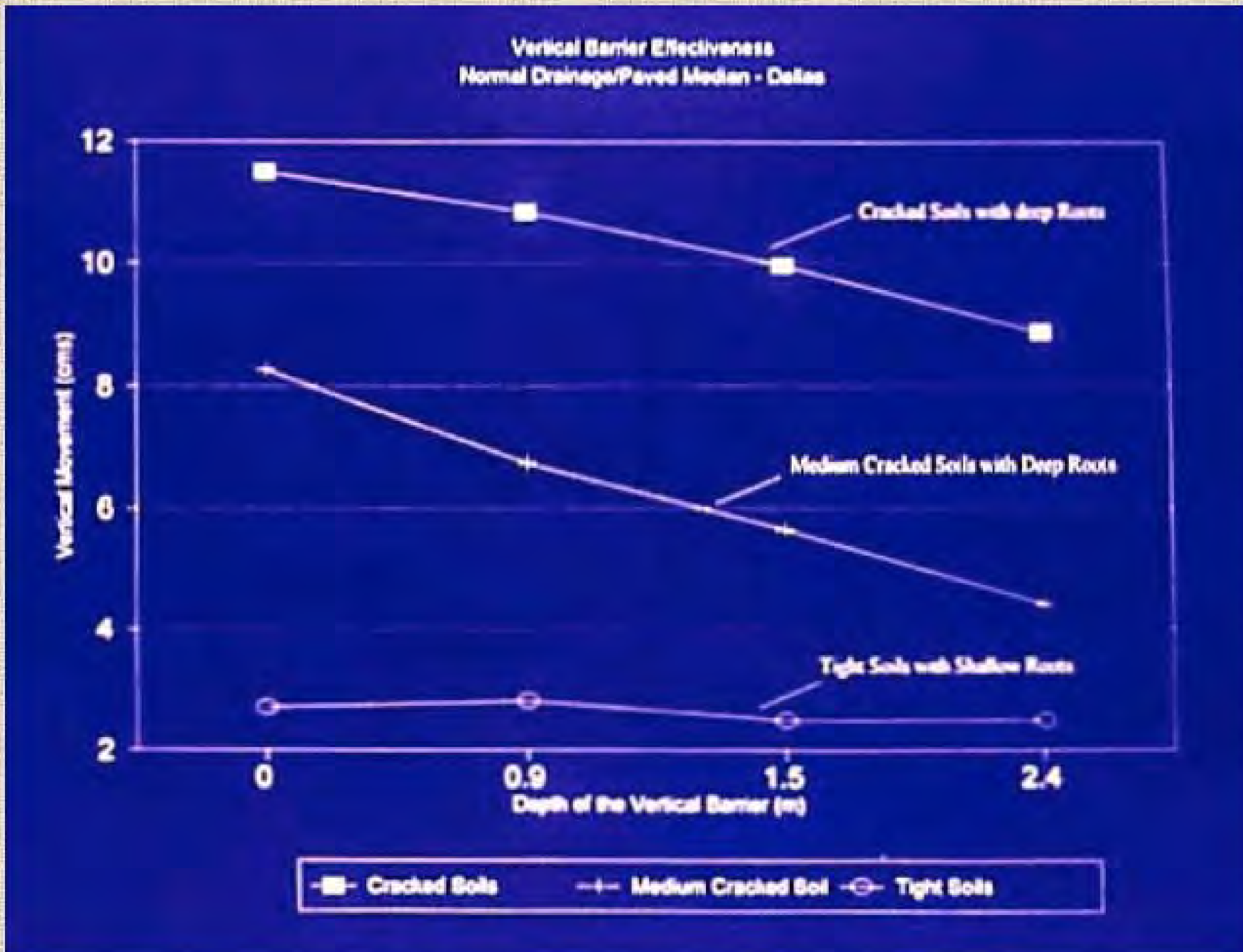


Vertical displacement across section
Year1
Vertical displacement (cm)
2.5
0.0
-2.5
-5.0
-7.5
-10.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
800.0
900.0
1000.0
Distance (cm)


Vertical displacement across section
Vertical displacement (cm)
Distance (cm)


Vertical displacement across section
Year 1
Vertical displacement (cm)
2.5
0.0
-2.5
-5.0
-7.5
-10.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
800.0
900.0
1000.0
Distance (cm)


Vertical Barrier Effectiveness
Normal Drainage/Medium Cracked Soils
Paved Median
Vertical Movement (cms)
Depth of the Vertical Barrier (m)
Dallas TMI = -11.3
San Antonio TMI = -21.3
Houston TMI = 14.8
El Paso TMI = -46.5
10
8
6
4
2
0
0
0.9
1.5
2.4


Vertical Barrier Effectiveness
Normal Drainage/Paved Median - Dallas
Vertical Movement (cm)
12
10
8
6
4
2
Cracked Soils with deep Roots
Medium Cracked Soils with Deep Roots
Tight Soils with Shallow Roots
0
0.9
1.5
2.4
Depth of the Vertical Barrier (m)
Cracked Soils
Medium Cracked Soil
Tight Soils

Condition of Soil	Range of P (cm^2/sec)
Cracked, Pervious	> 0.001
Medium Cracked	0.00005 - 0.001
Tight	< 0.00005

UNSATURATED PERMEABILITY

LL, PI, PL, % - #200, % - 2μ

Dry Density

$$P = \frac{\alpha \quad \gamma_d}{|S| \quad \gamma_w}$$

Unit Wt. of Water

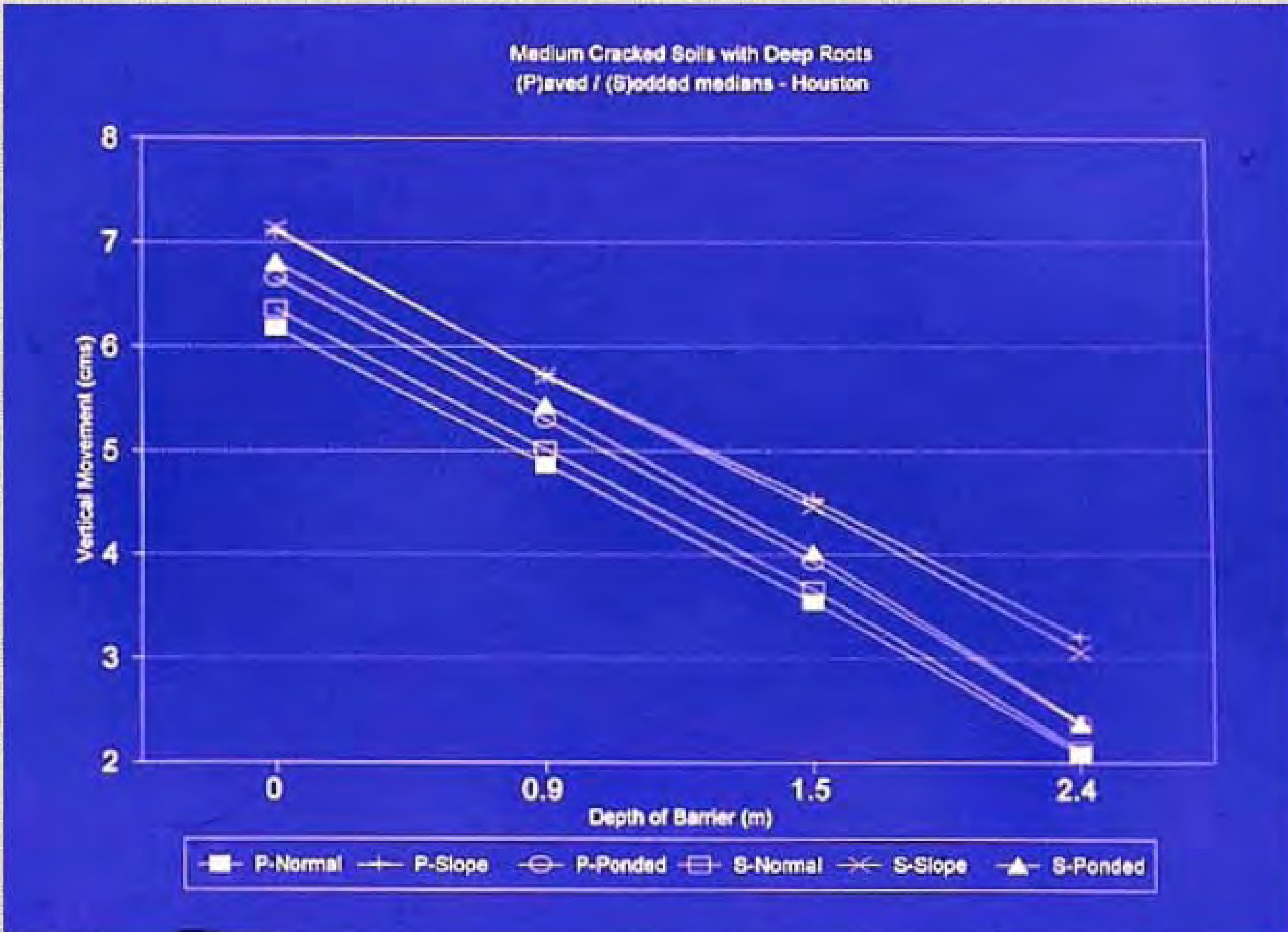
LL, PI, % - #200

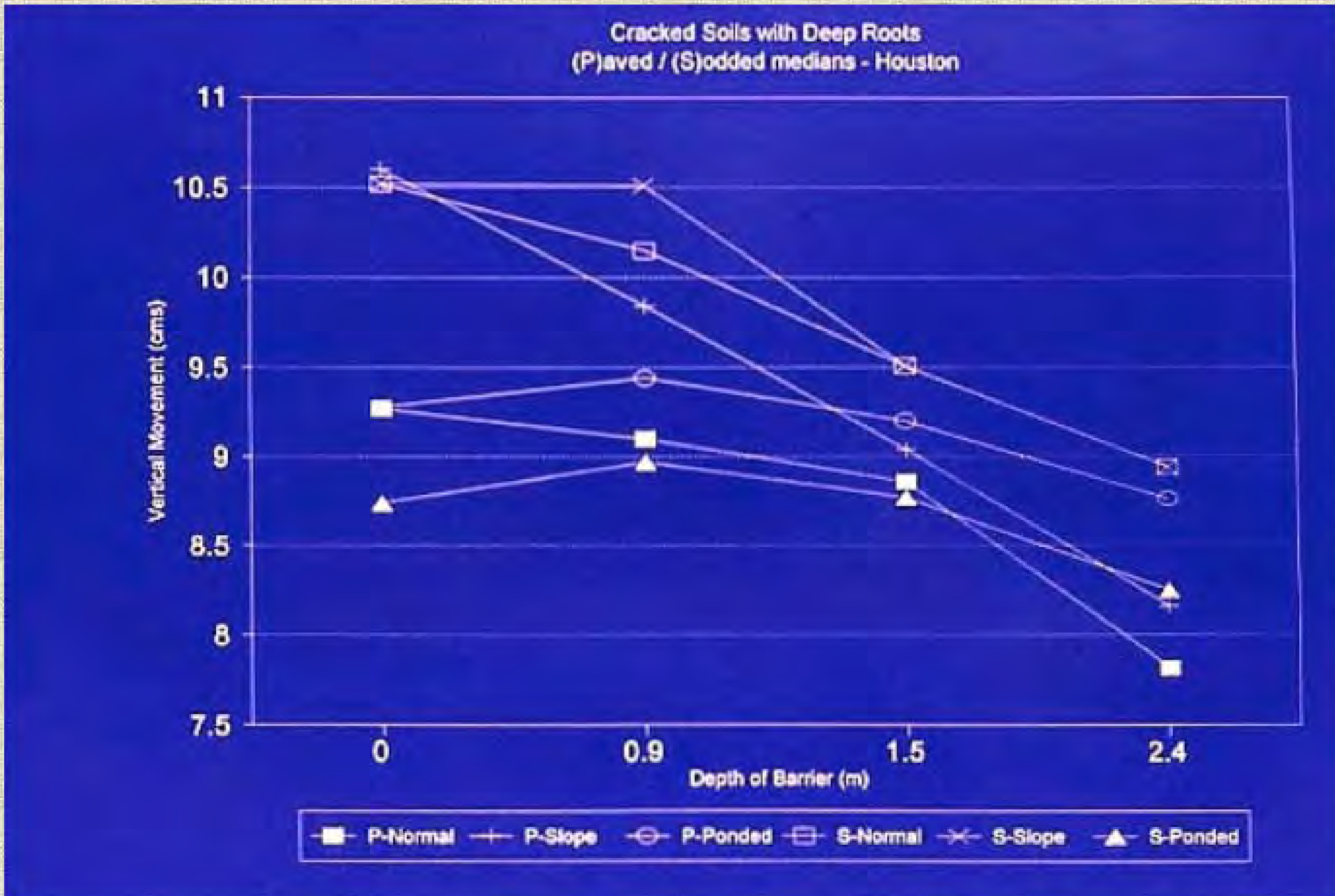
DIFFUSION COEFFICIENT

$$\alpha = 0.0029 - 0.000162\,(S) - 0.0122\,(SCI)$$

α = Diffusion Coefficient

SCI = Suction Compression Index


Medium Cracked Soils with Deep Roots
(P)aved / (S)odded medians - Houston
Vertical Movement (cms)
8
7
6
5
4
3
2
0
0.9
1.5
2.4
Depth of Barrier (m)
P-Normal
P-Slope
P-Ponded
S-Normal
S-Slope
S-Ponded

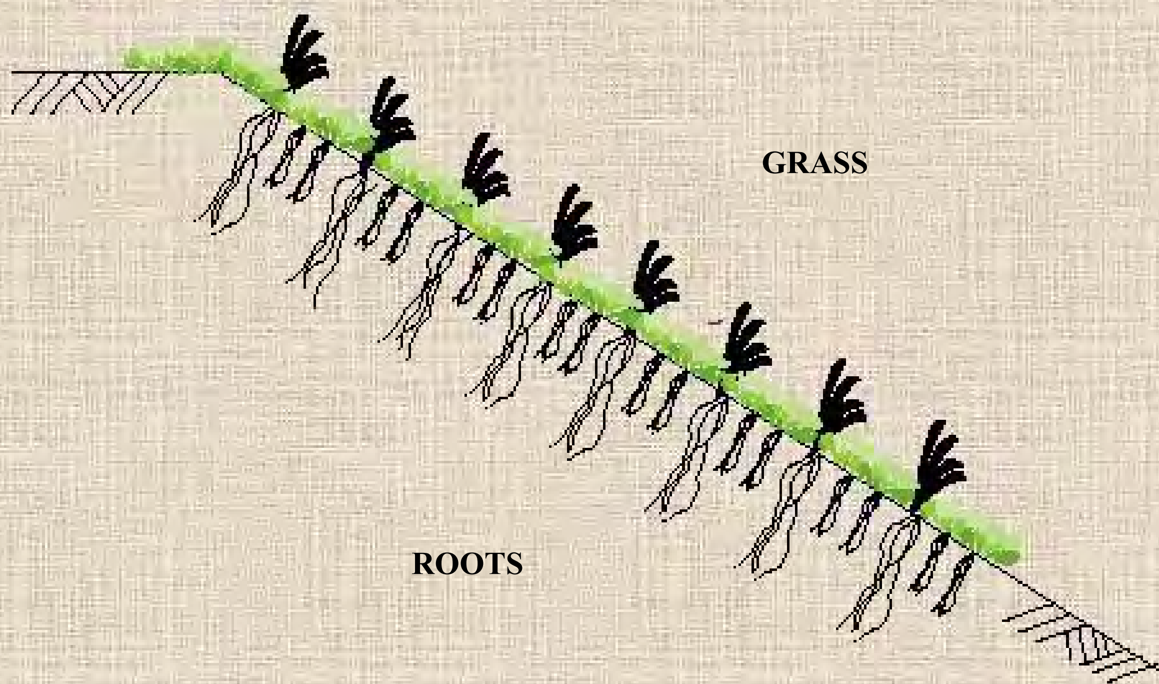

Cracked Soils with Deep Roots
(P)aved / (S)odded medians - Houston
Vertical Movement (cms)
11
10.5
10
9.5
9
8.5
8
7.5
0
0.9
1.5
2.4
Depth of Barrier (m)
P-Normal
P-Slope
P-Ponded
S-Normal
S-Slope
S-Ponded

SLOPES

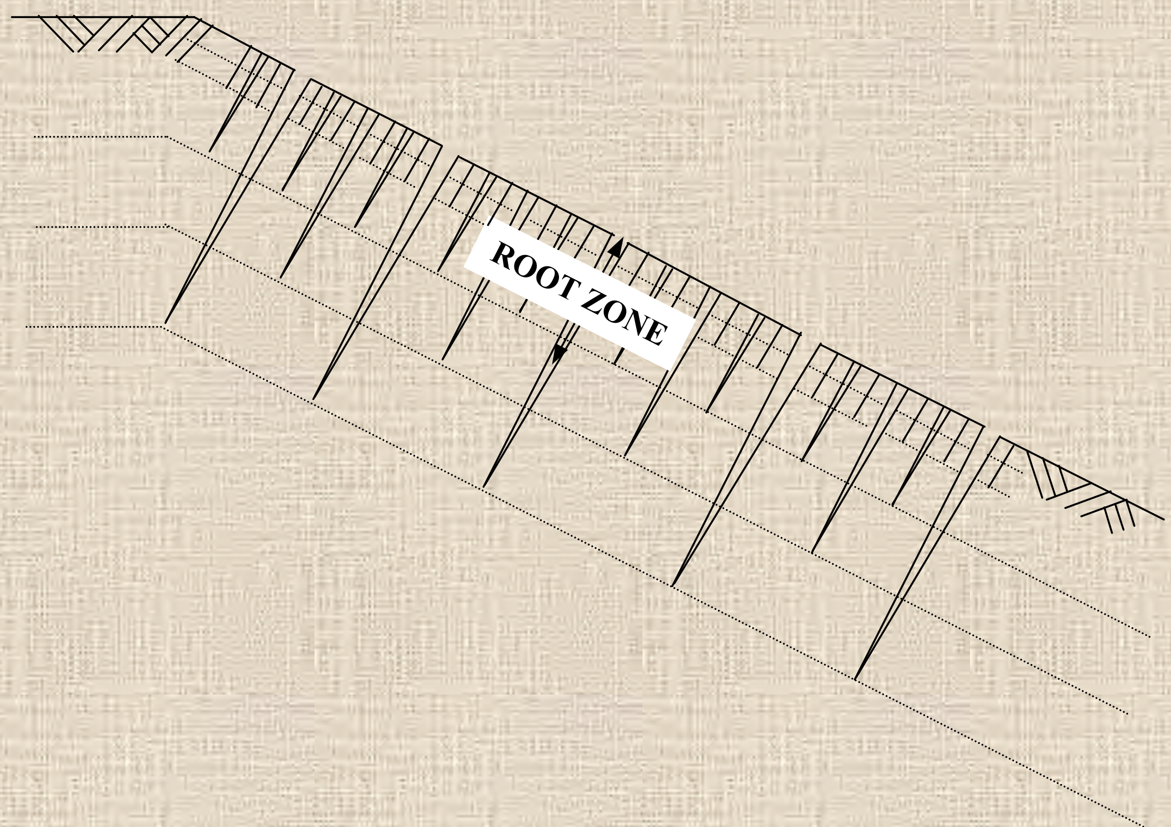
IN

EXPANSIVE SOILS

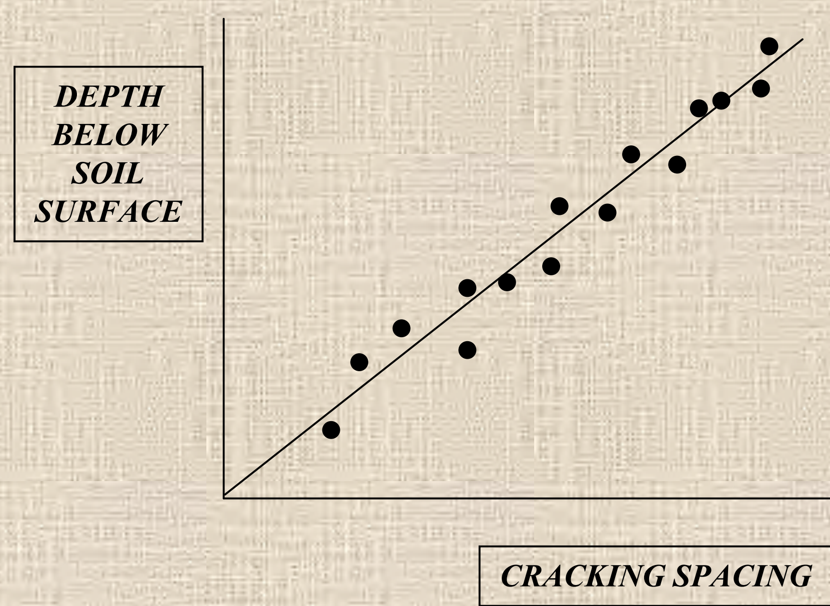
SHALLOW SLOPE FAILURE



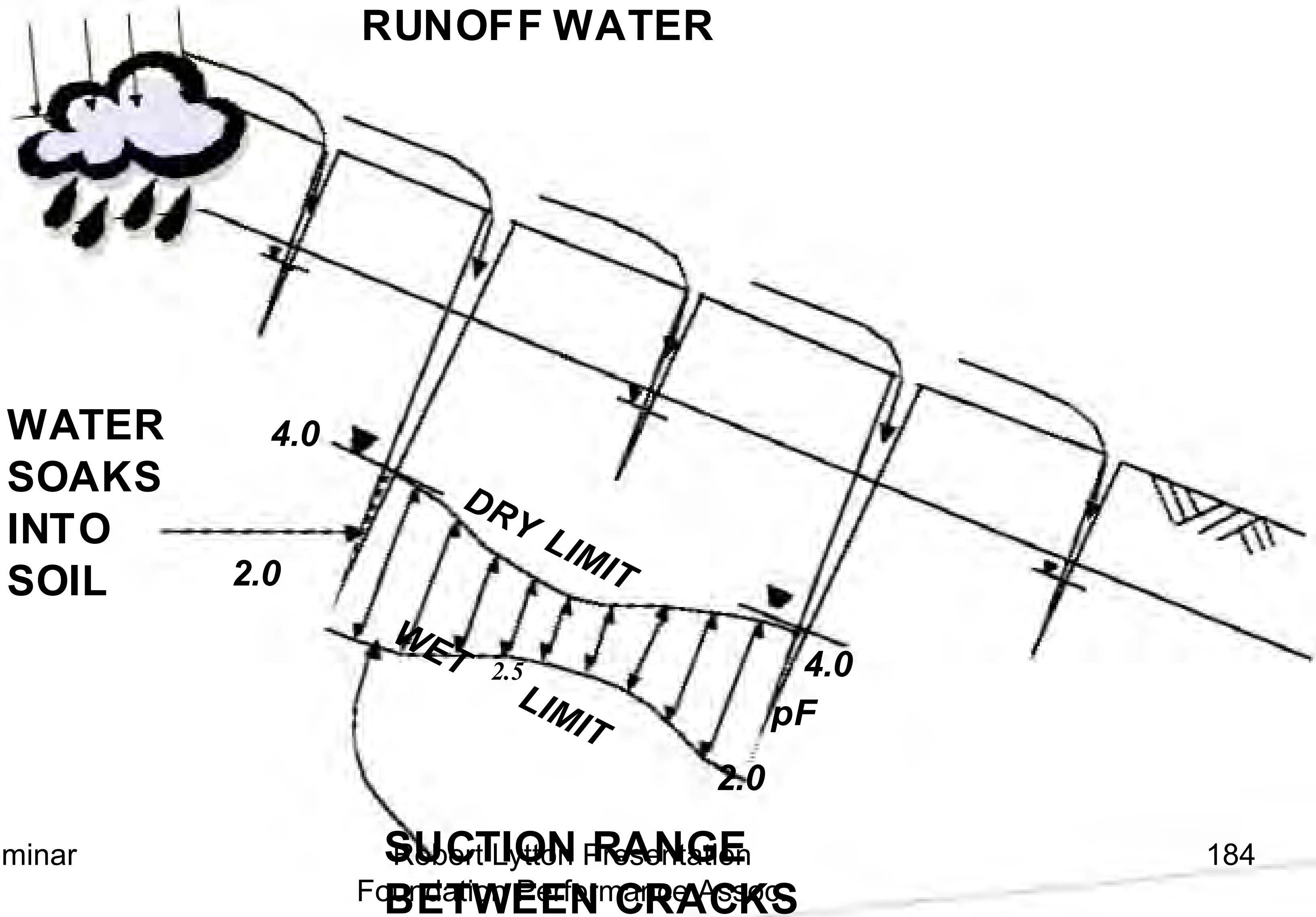
DURING DRY PERIODS ROOTS EXTRACT WATER FROM THE SOIL AND CAUSE SHRINKAGE CRACKS

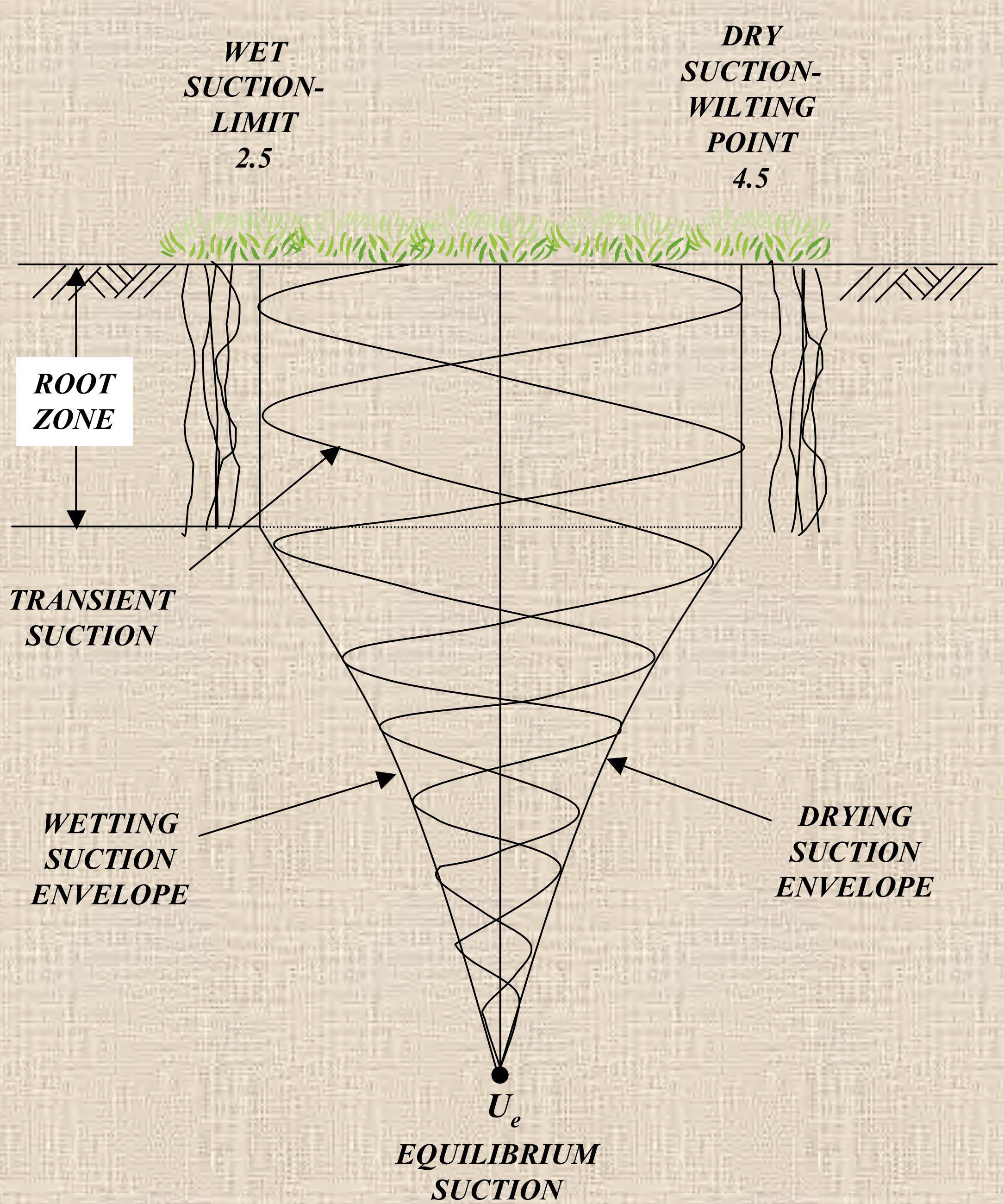


Crack Spacing Gets Larger with Depth



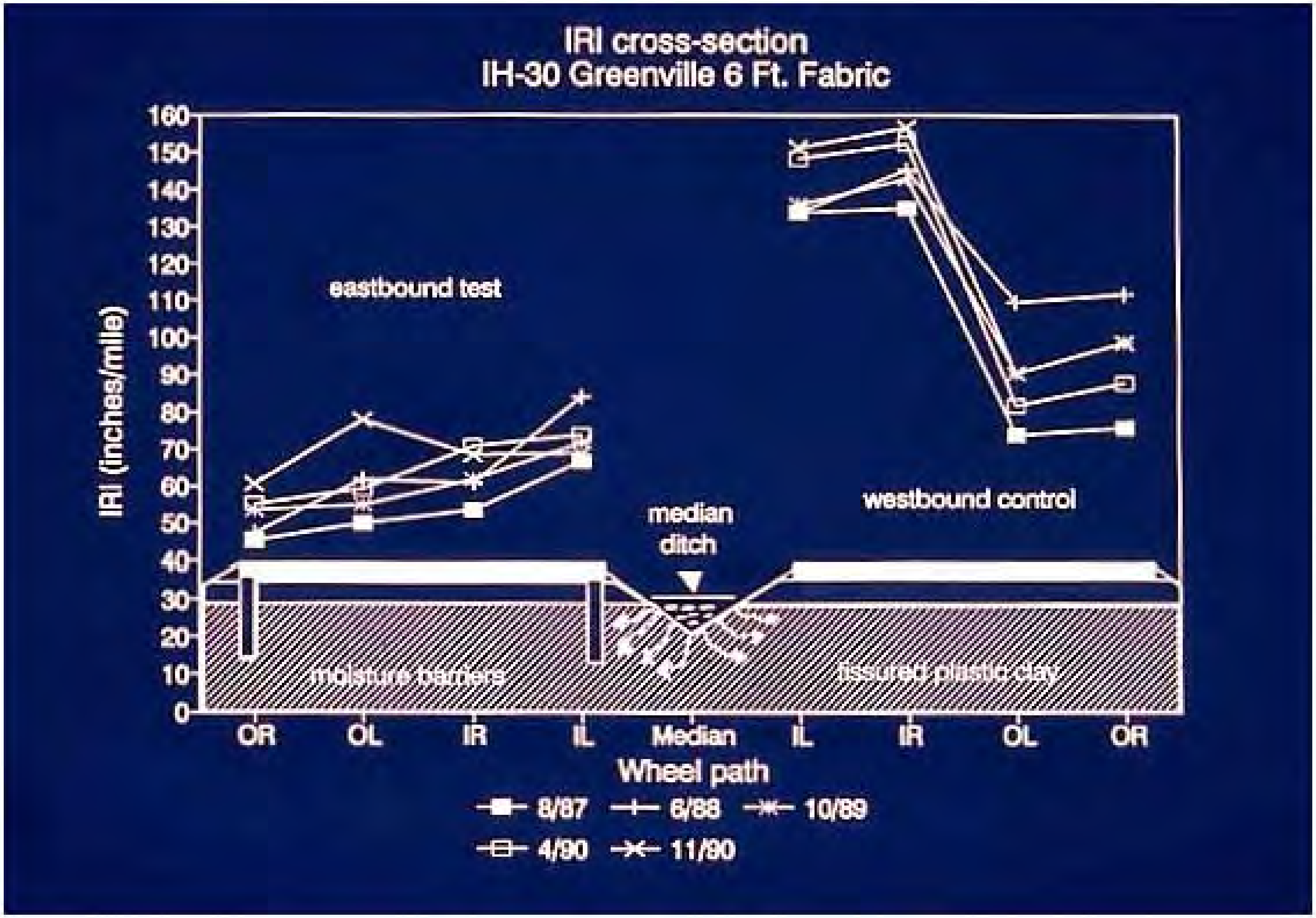
SOURCE : MICHAEL KNIGHT
PH. D. DISSERTATATION, GEOLOGY
UNIVERSITY OF MELBOURNE (AUSTRALIA)
1972


RUNOFF WATER
WATER SOAKS INTO SOIL
4.0
2.0
DRY LIMIT
WET LIMIT
2.5
4.0
pF
2.0
SUCTION RANGE BETWEEN CRACKS

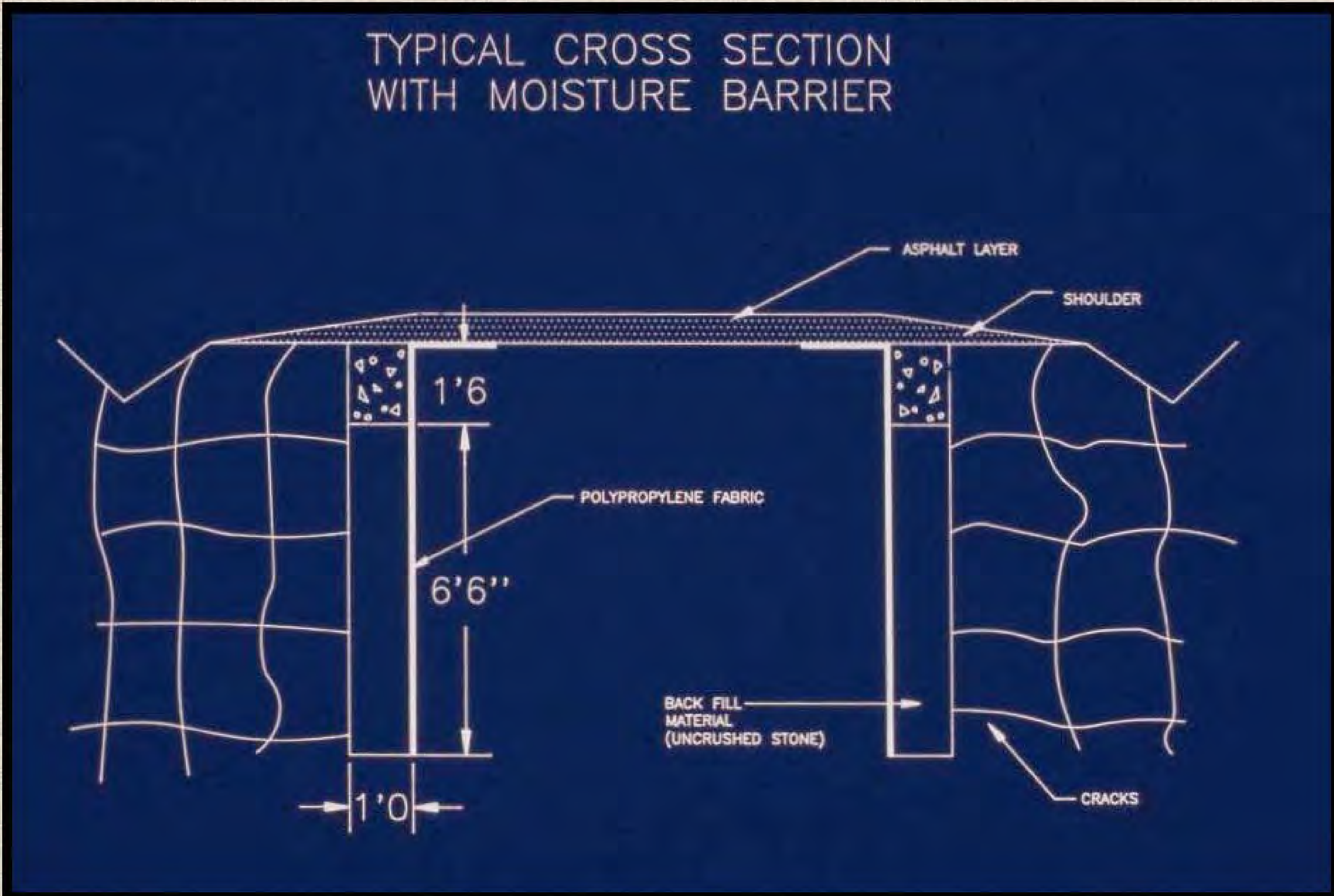

WET SUCTION-LIMIT 2.5
DRY SUCTION-WILTING POINT 4.5
ROOT ZONE
TRANSIENT SUCTION
WETTING SUCTION ENVELOPE
DRYING SUCTION ENVELOPE
U_e
EQUILIBRIUM SUCTION

DESIGN AND CONSTRUCTION

- PAVEMENTS ON EXPANSIVE SOILS
- SULFATE SWELLING PROBLEMS


IRI cross-section
IH-30 Greenville 6 Ft. Fabric
IRI (inches/mile)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
eastbound test
westbound control
median
ditch
moisture barriers
fissured plastic clay
OR
OL
IR
IL
Median
IL
IR
OL
OR
Wheel path
8/87
6/88
10/89
4/90
11/90


CEM. STAB. BACKFILL
GRAVEL BACKFILL
VERTICAL MOISTURE SEAL
PROPOSED
TYPICAL MAINLANE PAVEMENT SECTION


TYPICAL CROSS SECTION
WITH MOISTURE BARRIER
ASPHALT LAYER
SHOULDER
1'6
POLYPROPYLENE FABRIC
6'6"
BACK FILL
MATERIAL
(UNCRUSHED STONE)
CRACKS
1'0

SULFATE SWELLING PROBLEMS

LIME +

SULFATE +

WATER +

CLAY = PAVEMENT BUCKLING

QUESTION:

HOW CAN YOU TELL WHERE YOU HAVE A SULFATE SWELLING PROBLEM?

Approach

- **Establish "decision tree" approach**
- **Use GIS to organize geological, pedological, topographical, and test data to assess sulfate potential**
- **Use magnetometer to screen for threshold presence of sulfates**
- **Use stability models to validate decision thresholds**

Consider geological and pedological facts

- **Sulfate concentrations are generally low in surface soils and rocks**
- **Gypsum is present in soils developed from montmorillonitic Eagle Ford shale**
- **Sulfate induced heave observed most frequently in Eagle Ford where roads follow streams, or run across low-lying areas or hillside slopes**

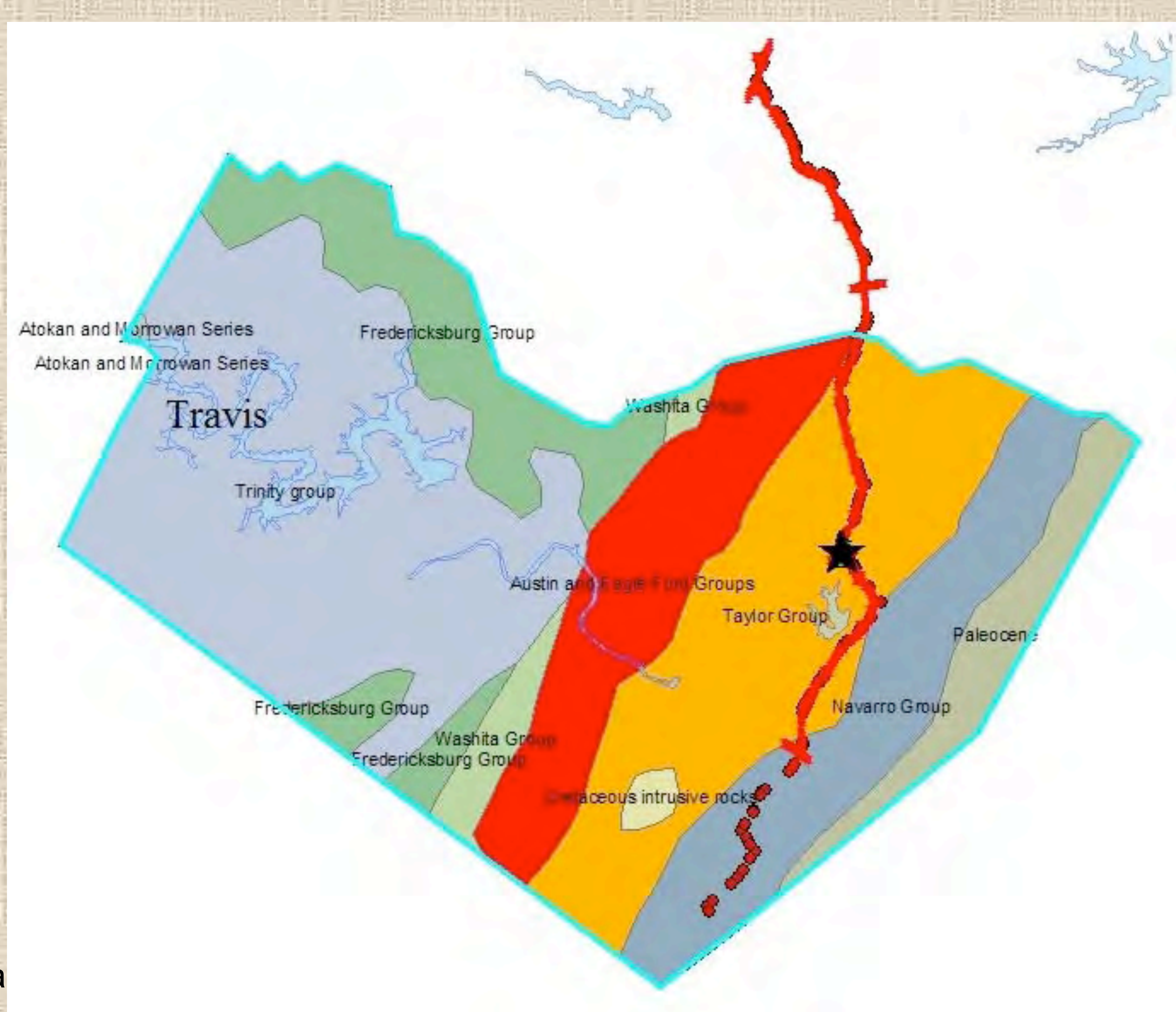
Data Sources

- **Bedrock geology and Soil Report data are used to assess sulfate potential**
- **Geological databases were accessed from The Texas Natural Resources Information System (TNRIS)**
- **The Soil Survey Geographic Database (SSURGO) was accessed regarding soils data**
- **Aerial photographs from TNRIS were accessed for topographical analysis**

Integrated GIS approach considers

- Sulfates (including pyritic sulfur) in bedrock
- Sulfates in soil profile
- Shrink-swell potential
- Permeability
- Topography

Geology Map of Travis County

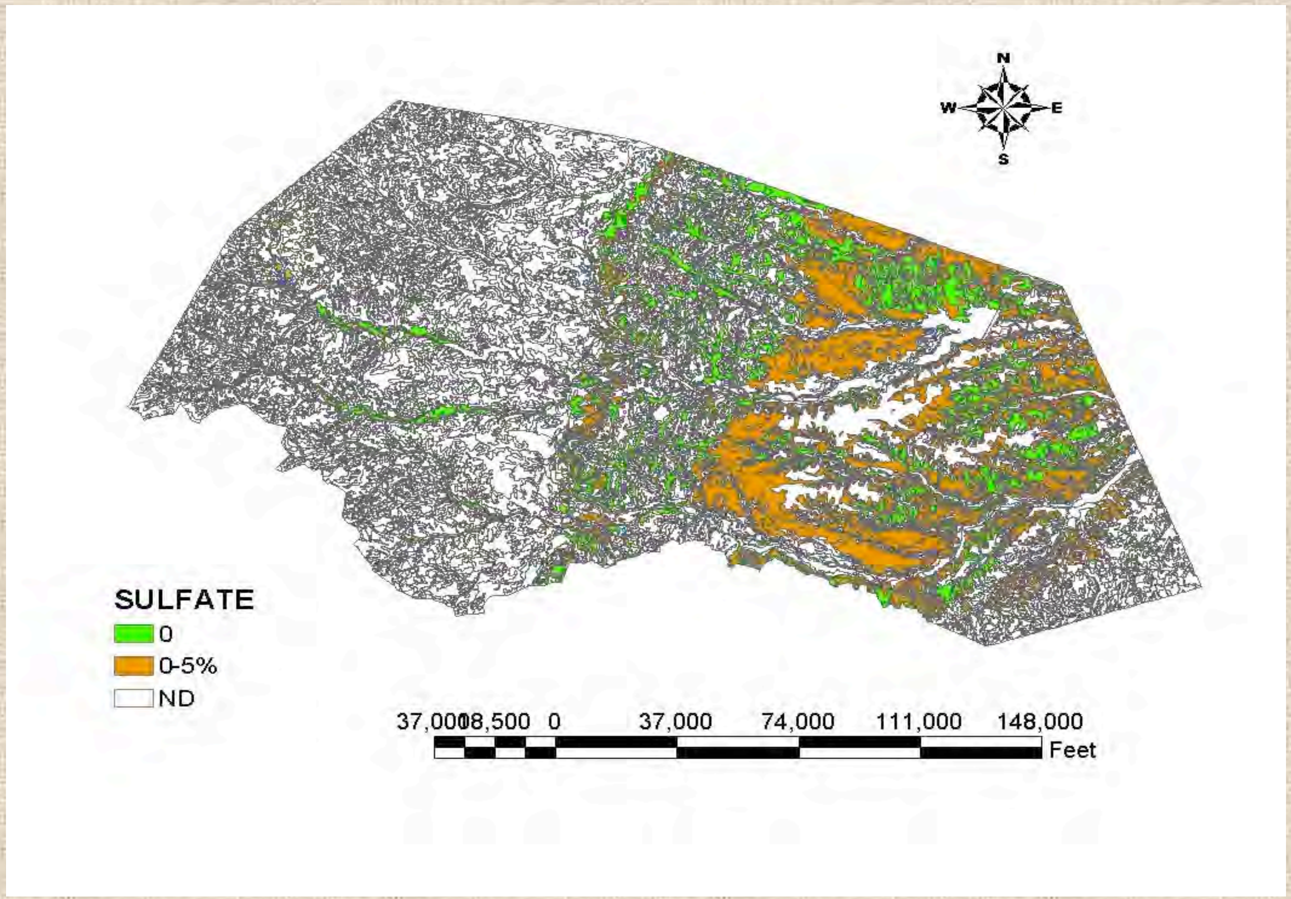


Attribute Table for Bedrock

AREA	UNIT	ROCK DESCRIPTION	COMMENTS
6.551	Te2	Eocene Claibome Group	Sediments primarily consists of sandstones, c
0.001	uK2	Austin and Eagle Ford Groups	Includes Pepper Shale,Cloice Shale,Bouldin
1.453	uK3	Taylor Group	Marine calcareous clay.Likely to contain high
6.551	Te2	Eocene Claibome Group	Sediments primarily consists of sandstones, c
1.453	uK3	Taylor Group	Marine calcareous clay,Likely to contain high
0.001	Tx	Paleocene	Highly glauconitic sands and sandy clays
1.56	Te1	Eocene Wilcox Group	Primarily composed of mud with various amou
1.56	Te1	Eocene Wilcox Group	Primarily composed of mud with various amou
0.753	Tx	Paleocene	Highly glauconitic sands and sandy clays
0.753	Tx	Paleocene	Highly glauconitic sands and sandy clays
0.001	Ki	Cretaceous intrusive rocks	Sulfate: No-data
0.008	IK2	Fredericksburg Group	Sulfate: No-data
0.081	IK2	Fredericksburg Group	Sulfate: No-data
0.026	IK3	Washita Group	Sulfate: No-data
0.018	PP1	Atokan and Morrowan Series	<Null>
0.018	PP1	Atokan and Morrowan Series	<Null>
1.56	Te1	Eocene Wilcox Group	Primarily composed of mud with various amou
1.019	IK1	Trinity group	Include interfingering carbonates deposited in
0.753	Tx	Paleocene	Highly glauconitic sands and sandy clays
0.753	uK4	Navarro Group	Marine marl and carbonaceous shale
0.008	IK1	Trinity group	Include interfingering carbonates deposited in
0.646	IK2	Fredericksburg Group	Sulfate: No-data
0.023	IK1	Trinity group	Include interfingering carbonates deposited in
1.453	uK3	Taylor Group	Marine calcareous clay.Likely to contain high
1.327	uK2	Austin and Eagle Ford Groups	Includes Pepper Shale,Cloice Shale,Bouldin
0.945	IK3	Washita Group	Sulfate: No-data
0.646	IK2	Fredericksburg Group	Sulfate: No-data

The comments section
Identifies if sulfates and/or
Pyritic sulfur is present or not

Williamson County Soil Map



Attribute Table For Williamson Soil

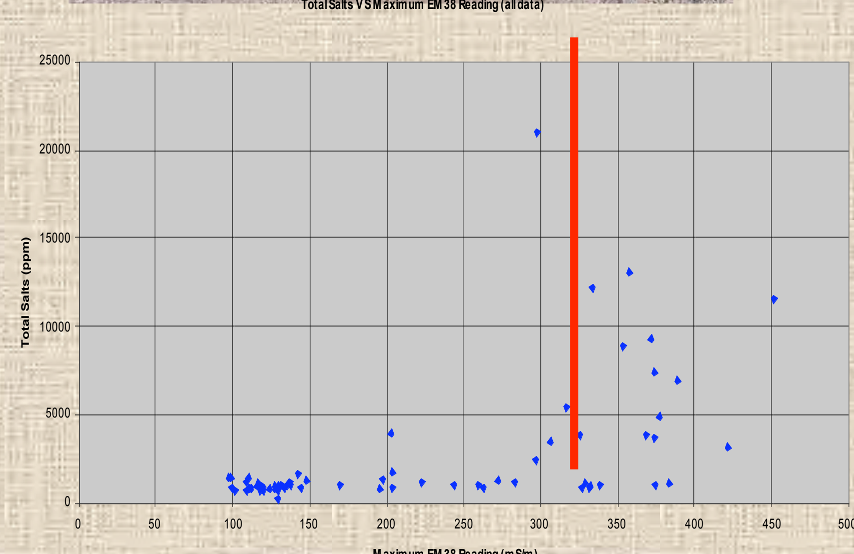
AREA	MUSYM	SULFATE	SHRINK SWELL	PERMEABILITY
0.00001	BkE	ND	low	moderately slow
0	DnC	ND	high	slow
0.00012	BkE	ND	low	moderately slow
0.00004	DnB	ND	high	slow
0.00004	EaD	ND	moderate	moderately slow
0.00002	BkE	ND	low	moderately slow
0	EaD	ND	moderate	moderately slow
0.00001	DoC	ND	moderate	moderately slow
0	DoC	ND	moderate	moderately slow
0.00009	BkE	ND	low	moderately slow
0.00003	DnC	ND	high	slow
0.00017	EaD	ND	moderate	moderately slow
0.00001	BkE	ND	low	moderately slow
0	DnB	ND	high	slow
0	DnB	ND	high	slow
0.00003	DoC	ND	moderate	moderately slow
0	BkE	ND	low	moderately slow
0	DoC	ND	moderate	moderately slow
0.00001	BkE	ND	low	moderately slow
0.00001	BkG	ND	low	ND
0.00001	BkE	ND	low	moderately slow
0	BkE	ND	low	moderately slow
0.00001	DoC	ND	moderate	moderately slow
0	SuB	0	moderate	moderate
0.0001	Of	ND	moderate	moderate
0	BkG	ND	low	ND
0.00001	BkE	ND	low	moderately slow
0.00001	BkE	ND	low	moderately slow
0	BkG	ND	low	ND
0.00003	BkE	ND	low	moderately slow
0.00003	EaD	ND	moderate	moderately slow
0.00009	BkE	ND	low	moderately slow
0	BkG	ND	low	ND
0.00004	DoC	ND	moderate	moderately slow
0	BkE	ND	low	moderately slow
0.00043	BkE	ND	low	moderately slow
0.00005	BkG	ND	low	ND
0.00002	BkE	ND	low	moderately slow
0	BkC	ND	low	moderately slow
0.00001	DoC	ND	moderate	moderately slow
0	BkE	ND	low	moderately slow
0.00041	GsB	ND	high	slow
0.00002	BkE	ND	low	moderately slow
0	FaB	ND	very high	moderate
0.00011	GsB	ND	high	slow
0.00004	GeB	ND	high	slow

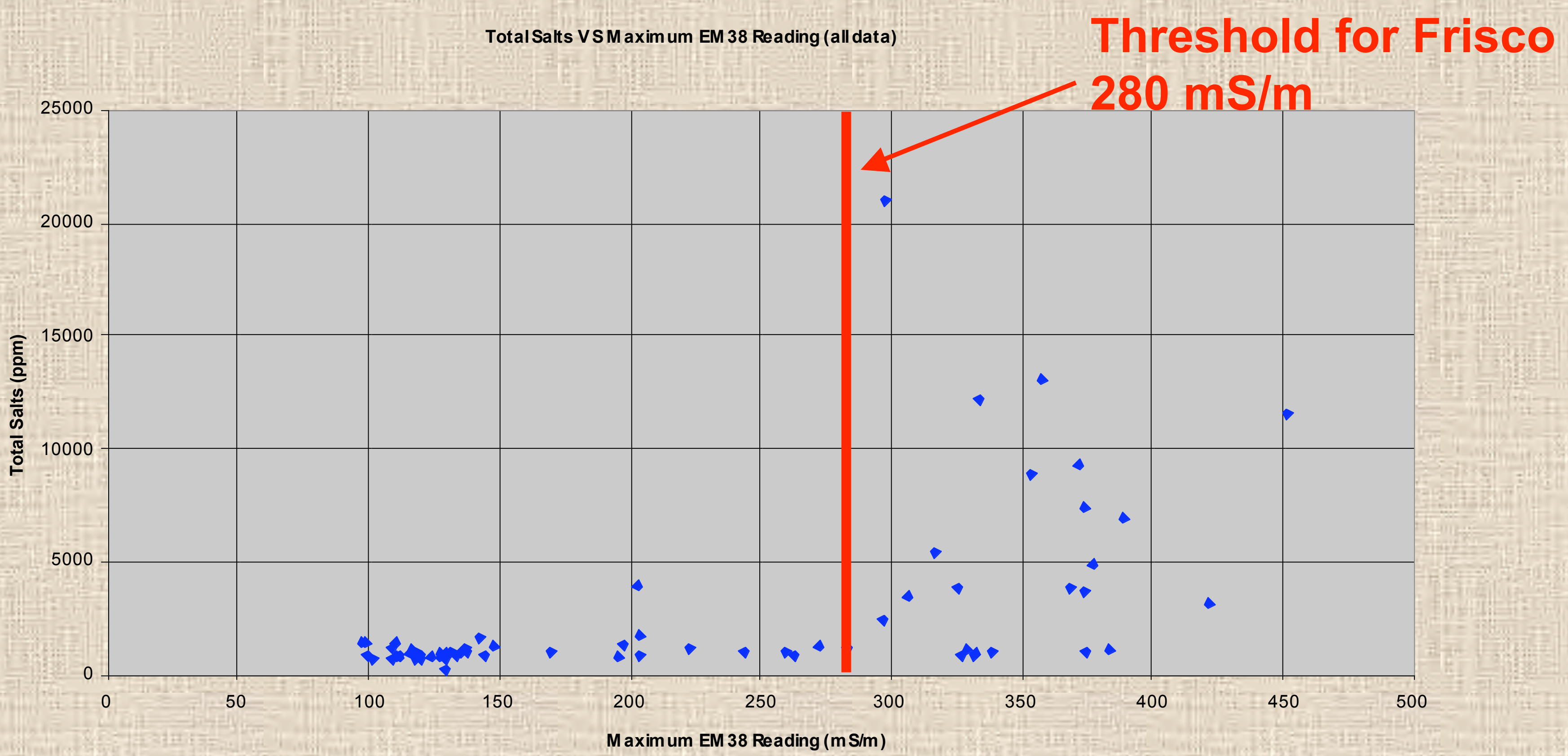
Magnetometer

- **Based on electrical conductivity (EC) as a soil property that can be measured rapidly across a large area**
- **Threshold sulfate level can be determined in the field using magnetometer**
- **The magnetometer is used to measure sulfate contents in the direction perpendicular to the long axis of the instrument, which means that it can canvass an area about 1 meter wide and 1.5 meter deep**
- **Threshold level should be carefully validated for SH 130 corridor**

Sampling and Testing when Sulfates are Suspected: Petry and Berger

- Sulfates occur in variable locations, seams
- Cannot detect with routine testing protocols
- Reliable screening tool needed
- Threshold reading of 280 mS/m established to relate to threshold soluble sulfate level (3,000 ppm).



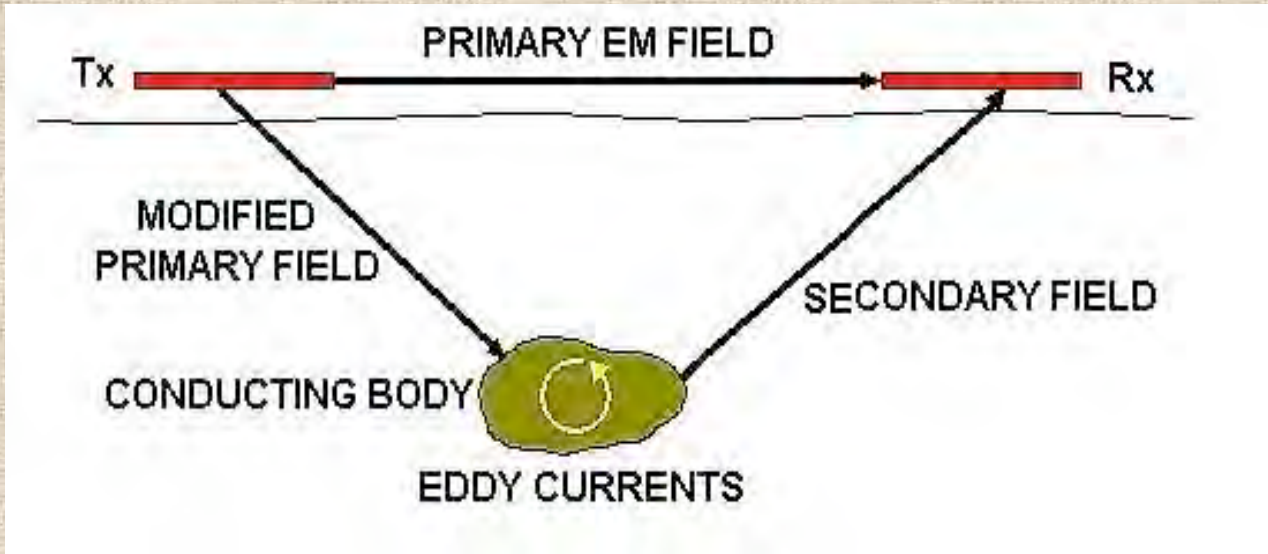

Total Salts VS Maximum EM38 Reading (all data)
Threshold for Frisco
280 mS/m
Total Salts (ppm)
25000
20000
15000
10000
5000
0
0
50
100
150
200
250
300
350
400
450
500
Maximum EM38 Reading (mS/m)

Magnetometer

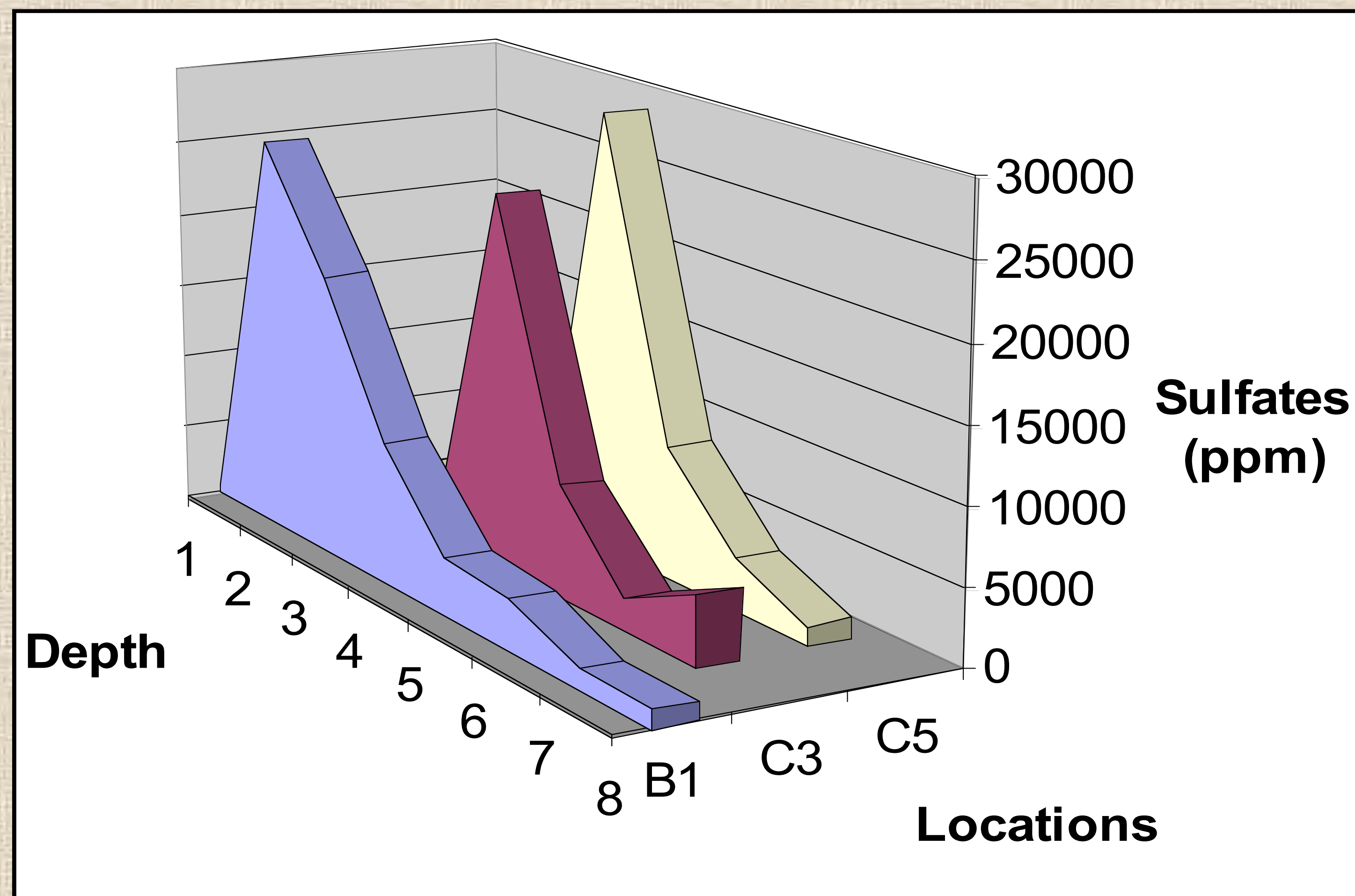
Operation of EM38

- **The EM38 device has a transmitting and a receiving coil**
- **The transmitting coil generates a secondary magnetic field that varies in strength with depth of soil, known as electromagnetic induction**
- **The conductivity readings obtained in milliSeimens/meter is the apparent conductivity**
- **3000 data points can easily be obtained in one hour using the DL720 data logger**

Magnetometer



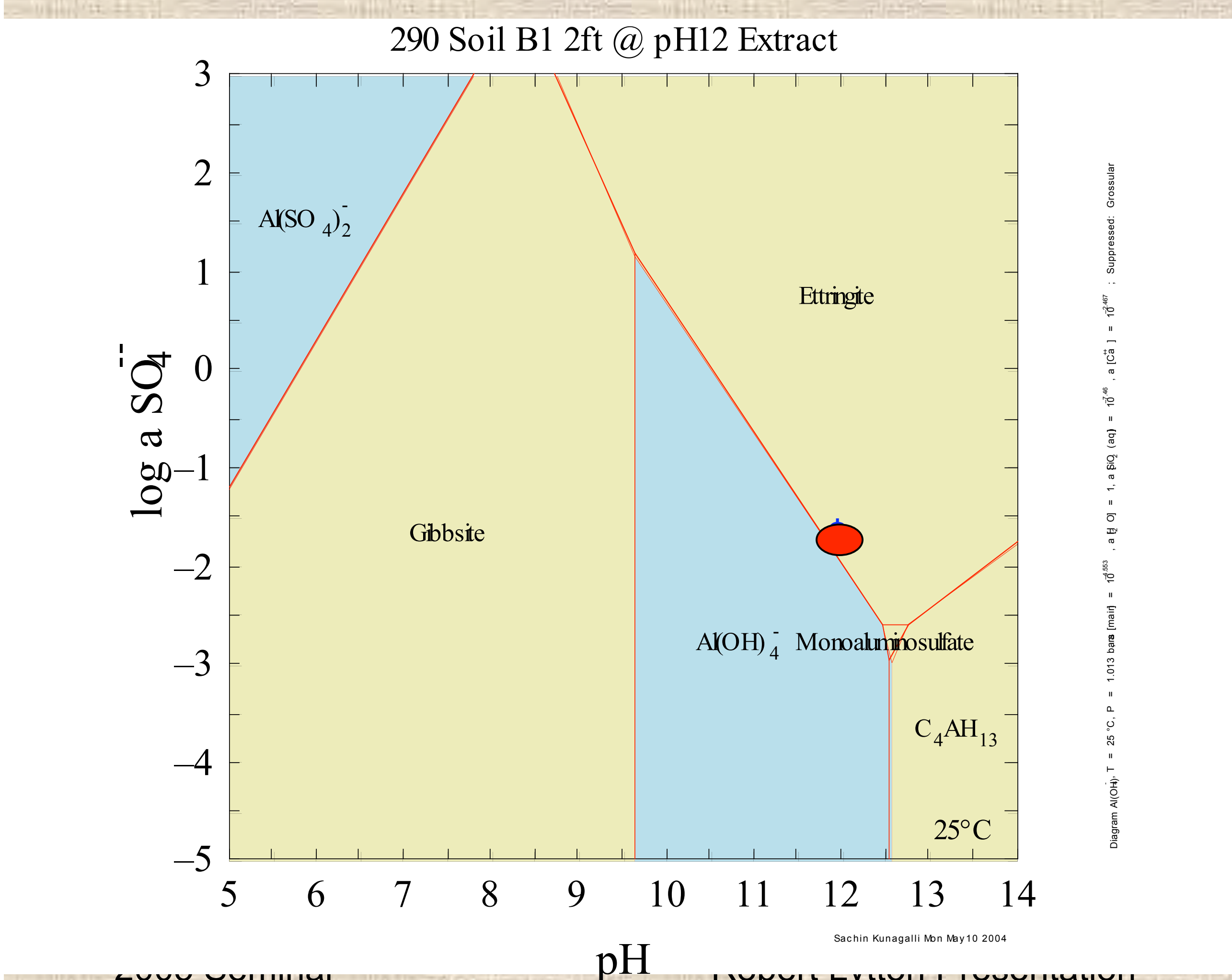
Variation of Sulfate along slope of the surface



Magnetometer Criteria

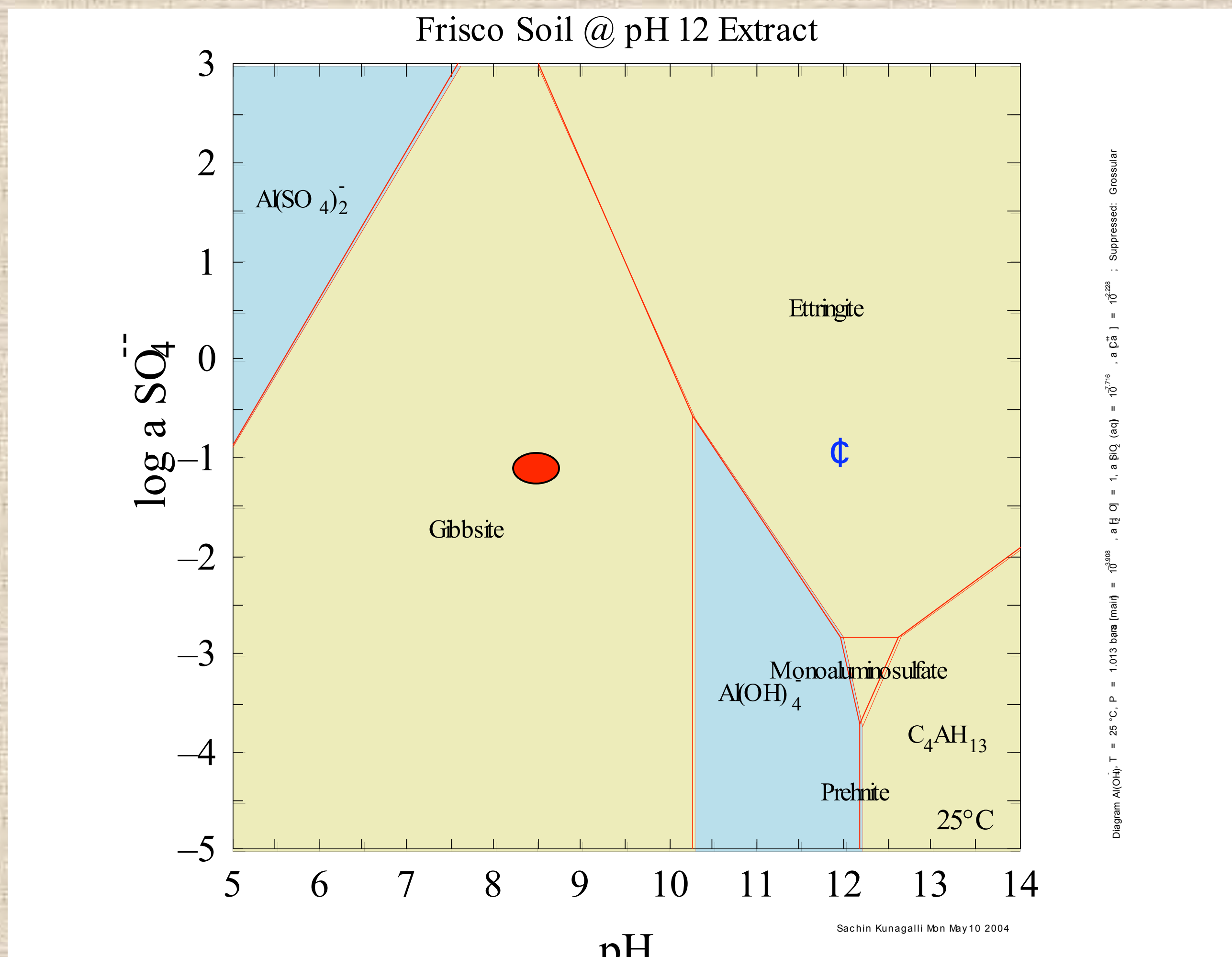
- **The threshold level of sulfates to form ettringite is in the range of 3300 ppm**
- **In the field, any conductivity value exceeding 230 ms/m should be considered problematic and associated with soluble sulfate levels exceeding 3300 ppm**
- **EM 38 conductivity meter should never be used in a trench less than a meter wide to avoid interference between dipoles**

Stability Models or Phase Diagrams
290 Soil - Depth of 24-inches



Soluble Sulfates = 18,700 ppm

Stability Model for Frisco Soil



PROGRAM AGENDA

1:00 pm	Geotechnical and Structural Design of Post-Tensioned Slabs-on-Ground using PTI 2004 Manual and Computer Programs VOLFLO 1.5 and PTISLAB 3.0 – Meyer, Read
2:30 pm	Break
2:40 pm	Design Concepts of Various Foundation Systems – Lytton
	Drilled Footings
	Floating Slabs
	Moisture Barrier
	Root Barrier
	Slopes
	Pavements
	Sulfates
➢4:00 pm	**Forensic Evaluation of Foundations – Dr. David Eastwood, P.E.**
4:30 pm	Legal Issues – Mr. David Dorr, Esquire
5:00 pm	Panel Discussion
	Questions and Answers
6:00 pm	Adjourn

SEMINAR

FOUNDATIONS ON EXPANSIVE SOILS

SHERATON – NORTH HOUSTON

HOUSTON, TEXAS

SEPTEMBER 23, 2005

LABORATORY TESTS

- SUCTION
- DIFFUSION COEFFICIENT

PAVEMENT ANALYSIS AND DESIGN

- ANALYSIS PROGRAM - FLODEF
- DESIGN PROGRAM - WINPRES

APPENDIX: TOTAL AND MATRIC SUCTION MEASUREMENTS WITH THE FILTER PAPER METHOD

Texas Transportation Institute

Texas A&M University System

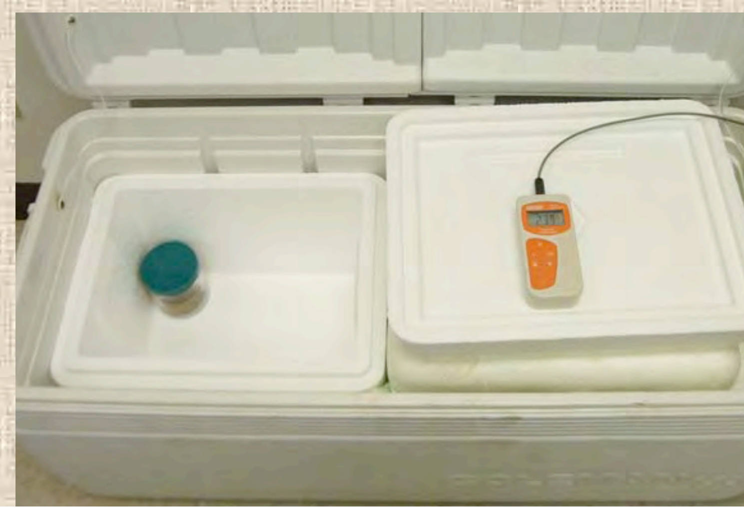
College Station, Texas

APPARATUS

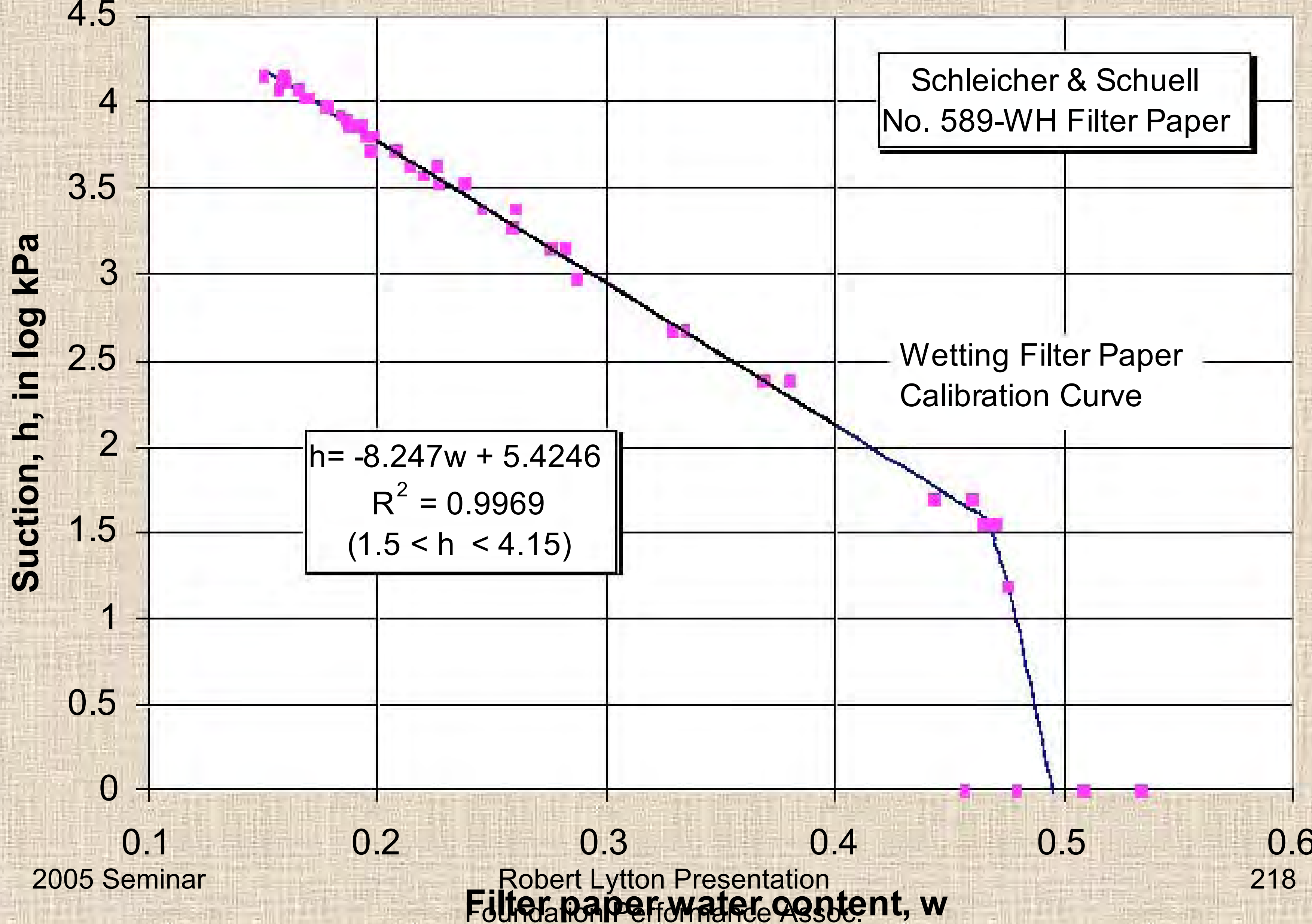
- ✓ Filter Papers
- ✓ Glass Jars
- ✓ Moisture Tins
- ✓ Tweezers
- ✓ Latex Gloves
- ✓ PVC-Rings
- ✓ Electrical Tape
- ✓ Aluminum Block

APPARATUS

- ✓ Sensitive Balance (0.0001 g)
- ✓ Constant Temperature Container (stability less than ±1°C)
- ✓ Oven (110 ± 5°C)



CALIBRATION CURVE



BEFORE COMMENCING THE TESTING, MAKE SURE THAT ALL ITEMS RELATED TO FILTER PAPER METHOD ARE CLEAN, MOISTURE, OIL, AND DUST FREE!

NOTE:

- *Make sure that tweezers are used to handle filter papers*
- *Make sure that moisture tins, o-rings are handled with gloves*

• Use a container that a Shelby-tube soil sample can be fit into easily without the disturbance of the soil sample.

• Cut the soil sample into two halves for matric suction measurements.

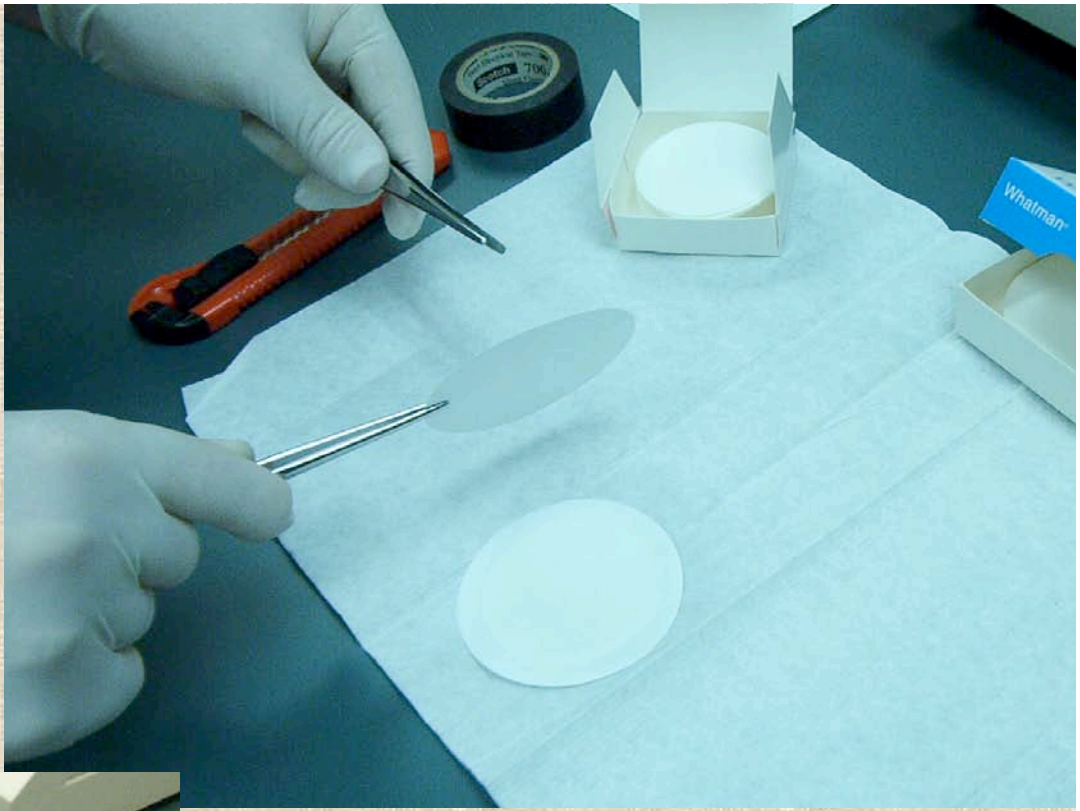
• Make sure that the surfaces of the soil samples are smooth and flat for establishing an intimate contact between the soil sample and the filter paper for matric suction measurements.

NOTE: When preparing the soil samples make sure that sample disturbance is minimal.

• Remove a *Schleicher & Schuell No. 589-WH* filter paper from the box using tweezers (5.5 cm in diameter)

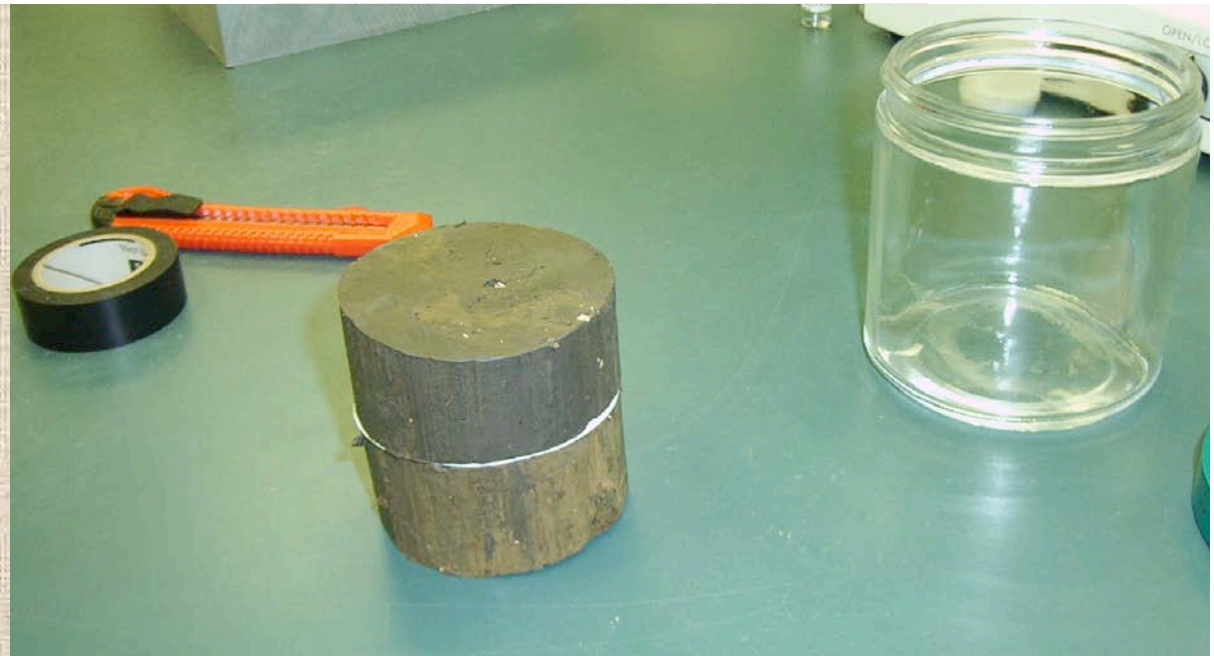
Almost any brand of high permeability and larger diameter filter paper can be used as protective filter papers for matric suction measurements (as shown in the lower left box and in the picture, about 70 mm in diameter)

• For matric suction measurements, insert a single Schleicher & Schuell No. 589-WH filter paper in between two larger in diameter protective filter papers as shown on the right



• Using tweezers put the sandwiched filter papers on top of the soil sample as shown on the left

• Put the other half of the soil sample on top, keeping the sandwiched filter papers in between and in intimate contact with the soil samples



• Tape the two pieces of the soil sample together

NOTE: Electrical tape works nicely for this purpose

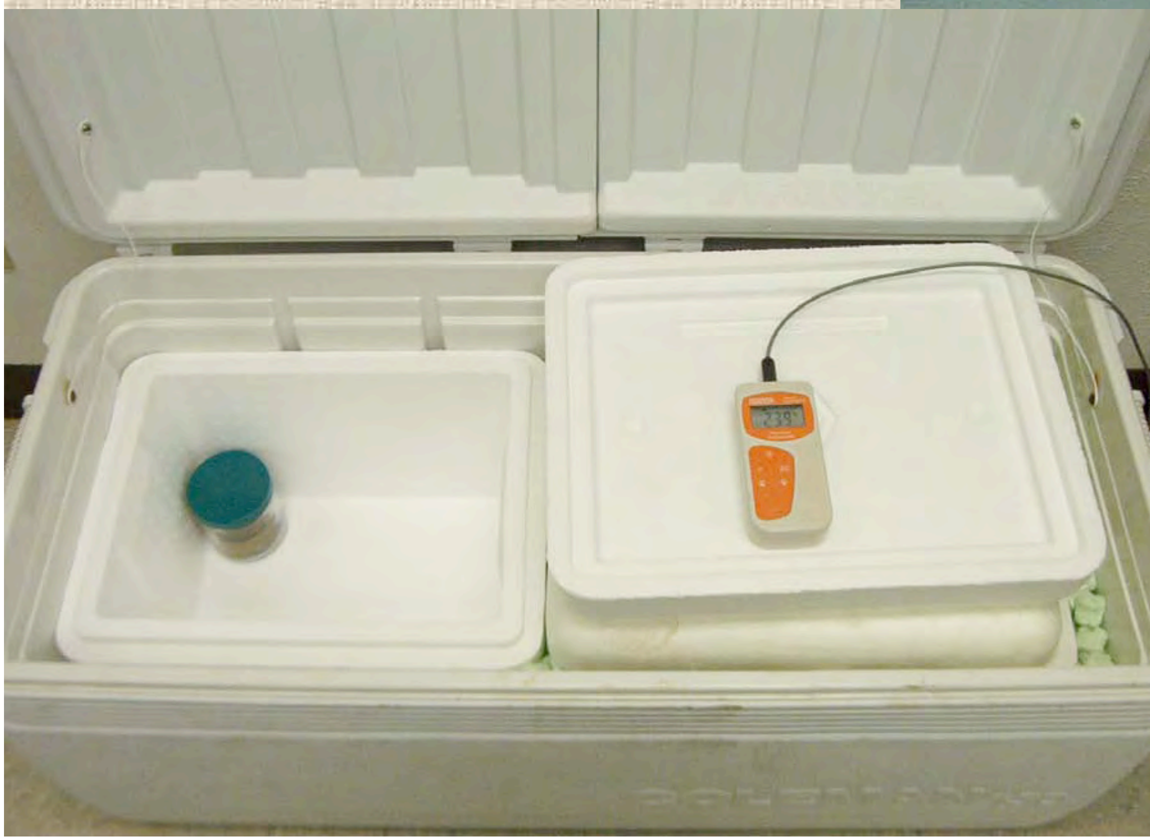
• Insert a clean PVC O-ring, with the sharp edge facing up, on top of the soil sample for total suction measurements

• Place two *Schleicher & Schuell No. 589-WH* filter papers on top of the ring as shown on the left

NOTE: Bend the edge of the top filter paper up a little so that it will be easy to remove them later!

- Put the lid on and tape it tight to prevent any moisture exchange between the air inside and the air outside of the jar

- Label the jar as necessary



- Insert the glass jar into a well-insulated container for suction equilibrium

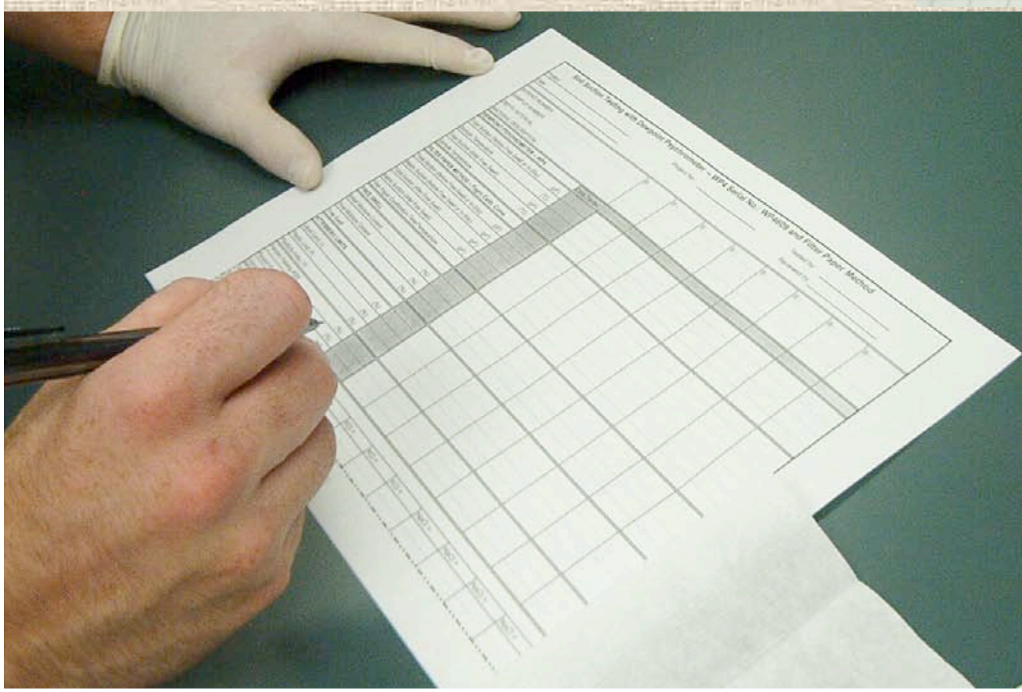
NOTE: Temperature control is critically important

✓ The setup, as described in the previous slides, will be kept in a temperature-controlled environment for at least one week

✓ Temperature fluctuations should be kept as low as possible, preferably below ± 1^{o}C

AT THE END OF AT LEAST ONE WEEK OF EQUILIBRIUM PERIOD:

- Before opening the lid of the temperature-controlled container, take the dry, cold weight of the moisture tins



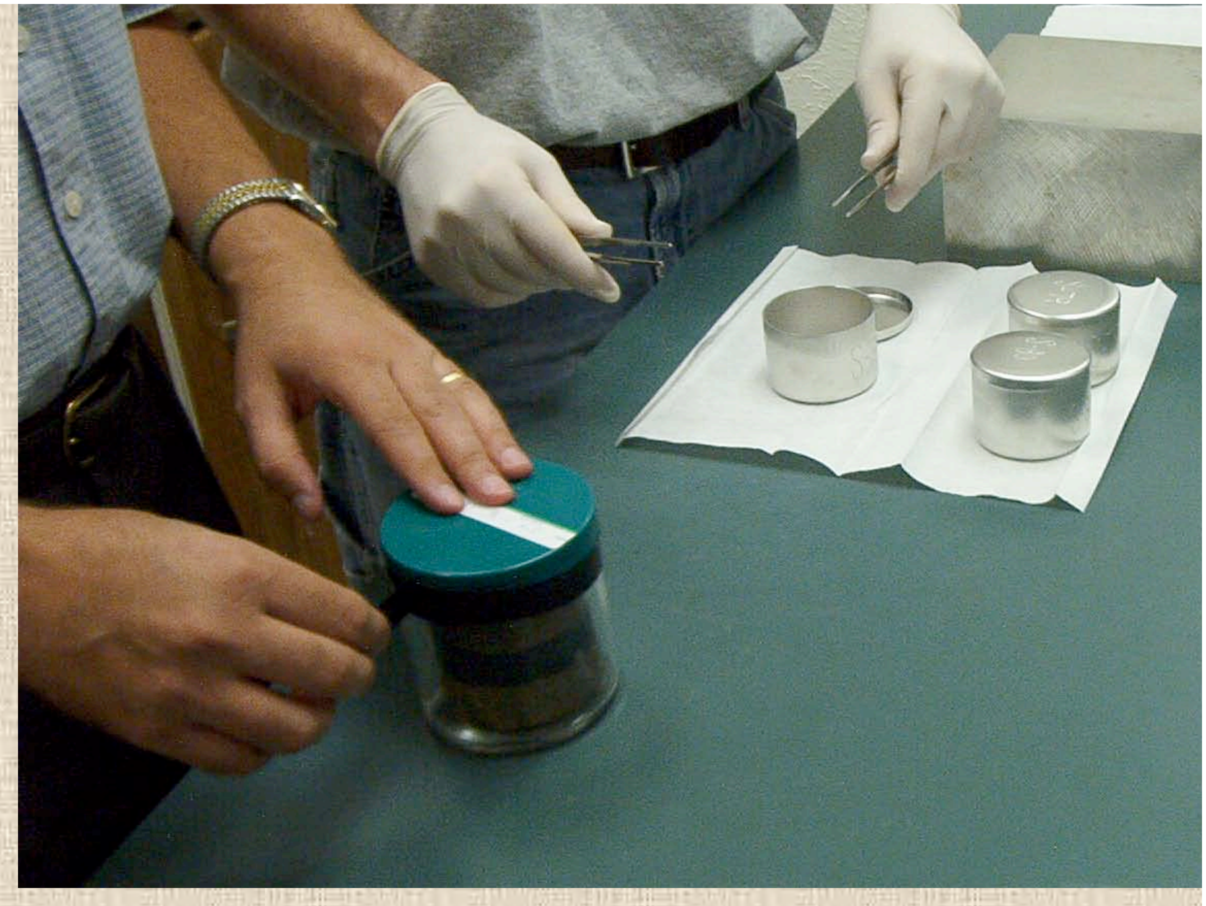
- Record all the weights with their corresponding tin numbers

NOTE: Use a balance at least to the nearest 0.0001 g. accuracy

• Remove a glass jar from the temperature-controlled container

• Time is critical at this stage and thus it is suggested that two people share the work

• The time that the filter papers are exposed to the lab environment should be minimal, preferably less than a few seconds



Note that while one person is opening the glass jar the other person is ready to pick up the filter papers

• Open the glass jar and quickly carry the filter paper to the moisture tin using tweezers, in less than a few seconds

• Immediately close the lid of the moisture tin with the wet filter paper inside

- After closing the lid of the moisture tin, immediately weigh the tin with the wet filter paper inside

- Record the weight as cold tare plus wet filter paper mass

Note that this is a total suction measurement

• Continue with the matric suction measurement by removing the tape that was holding the soil samples together

• Remove the filter paper that was sandwiched between the two protective filter papers

• Immediately carry the filter paper to the moisture tin

• Immediately close the lid of the moisture tin and weigh the tin with the wet filter paper inside

• Record the weight as cold tare plus wet filter paper mass

Note that this is a matric suction measurement

- After opening all the glass jars and recording the weight of the moisture tins with the wet filter papers inside, carry them to a hot oven with the lids half open
- Leave them in the oven for at least 10 hours

- Before taking them out from the oven, close their lids for equilibrium and leave them in the oven for about 5 minutes

• Remove a hot tin from the oven and put on a large aluminum block

NOTE: The aluminum block will expedite the process of the cooling

• Leave the tin on the block for about 20 seconds

• Weigh the hot tin with the dry filter paper inside

• Record the weight as hot tare plus dry filter paper mass

- Finally, take the dry filter paper out of the tin
- Weigh the empty hot tin

- Record the weight as hot tare mass
- Repeat the above process for other tins in the oven

- Calculate the moisture content of each filter paper for both total and matric suction measurements. A calculation work sheet as given in the next slide can be used

- Obtain the suction value from the calibration curve that was provided above

SUCTION CALCULATION WORKSHEET

THE FILTER PAPER METHOD SUCTION MEASUREMENT S WORKSHEET									
Date Sampled:								Date Tested:	
Boring No.:								Tested By:	
Sample No.:									
Depth									
Moisture Tin No.:									
Total or Matric Suction Top or Bottom Filter Paper									
Cold tare mass, g	T_c								
Mass of wet filter paper + cold tare mass, g	M_1								
Mass of dry filter paper + hot tare mass, g	M_2								
Hot tare mass, g	T_h								
Mass of dry filter paper, g ($M_2 - T_h$)	M_f								
Mass of water in filter paper, g ($M_1 - M_2 - T_c + T_h$)	M_w								
Water content of filter paper, g (M_w / M_f)	W_f								
Suction, log kPa	h_1								
Suction, pF	h_2								

APPENDIX: LABORATORY MEASUREMENTS OF ALPHA DIFFUSION COEFFICIENTS

Texas Transportation Institute

Texas A&M University System

College Station, Texas

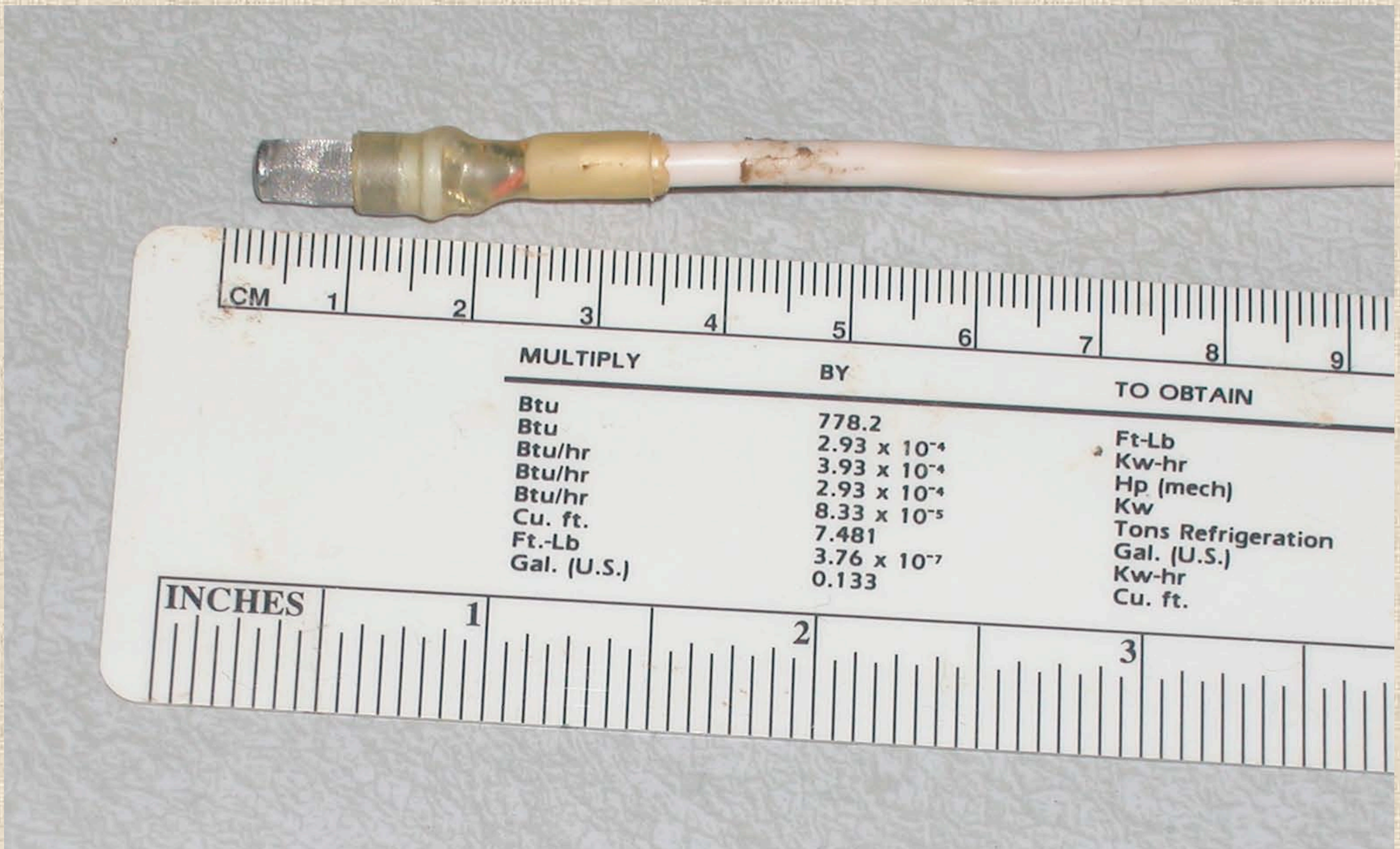
APPARATUS

- Thermocouple Psychrometers
- Sling Psychrometer
- Temperature Control Unit
- A drill-bit, knife, spatula, tape, sealing material (aluminum foil, plastic wrap, etc.)

Diffusion Test Setup

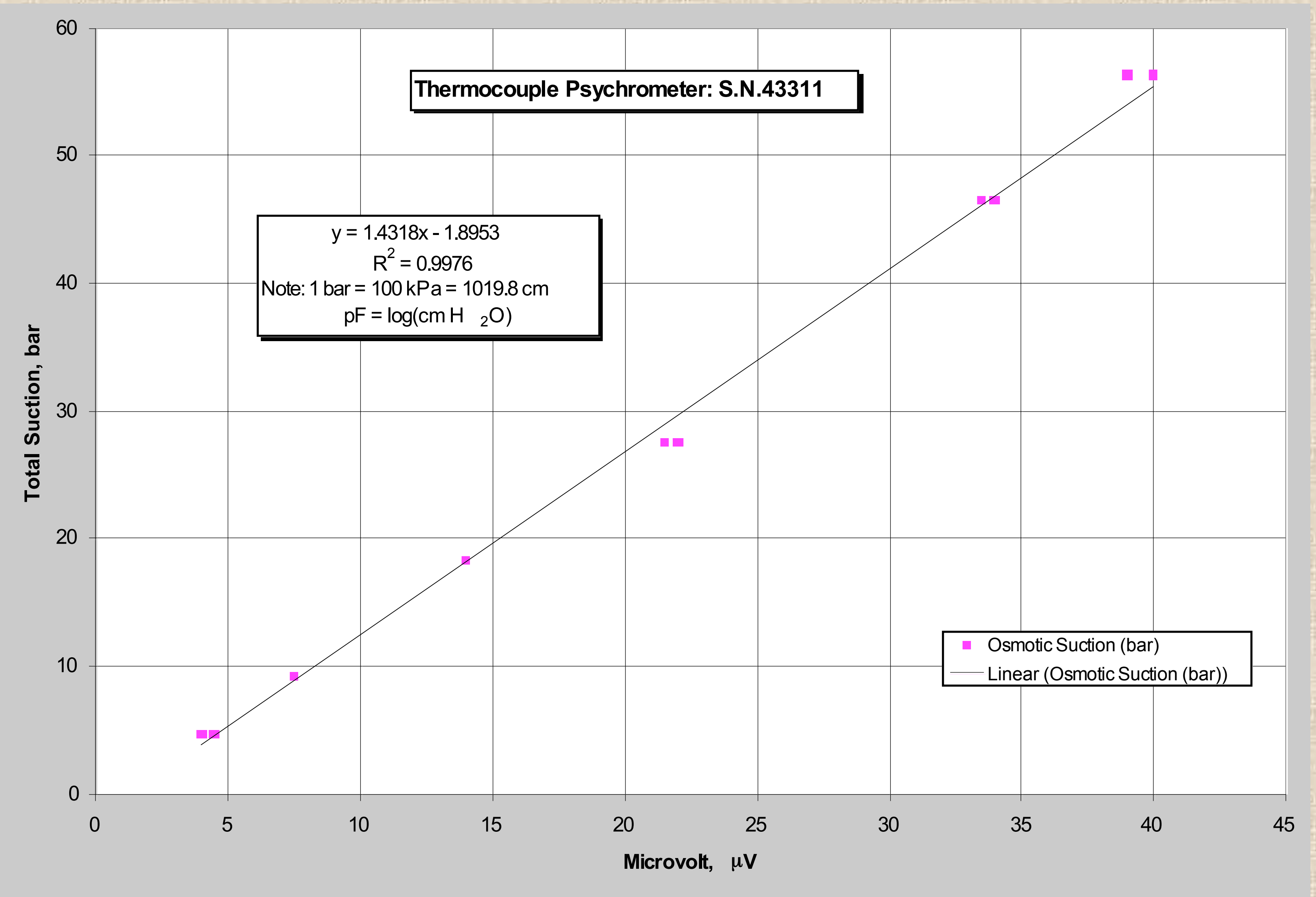
Temperature Control Unit

Thermocouple Psychrometer

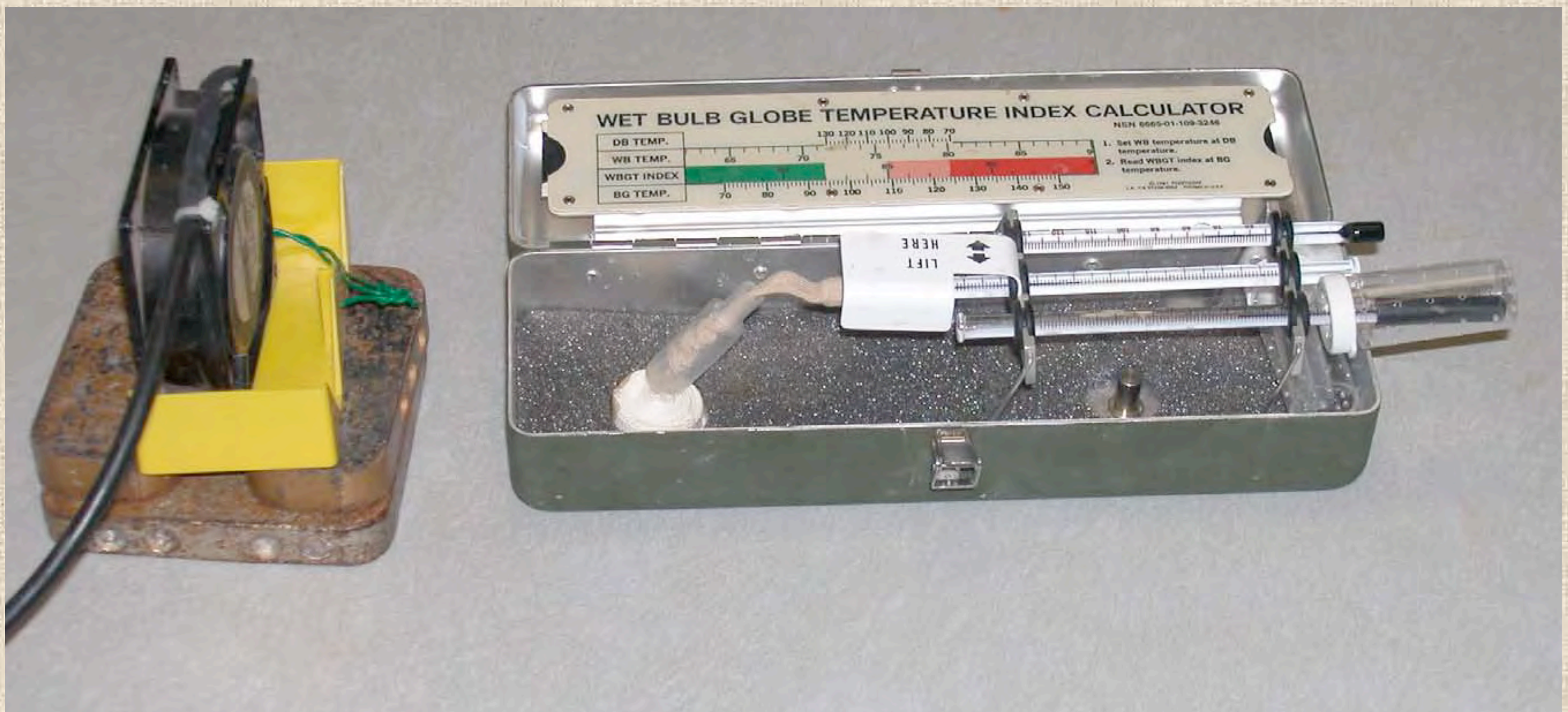


Psychrometer Calibration Salt Solutions

Calibration Curve



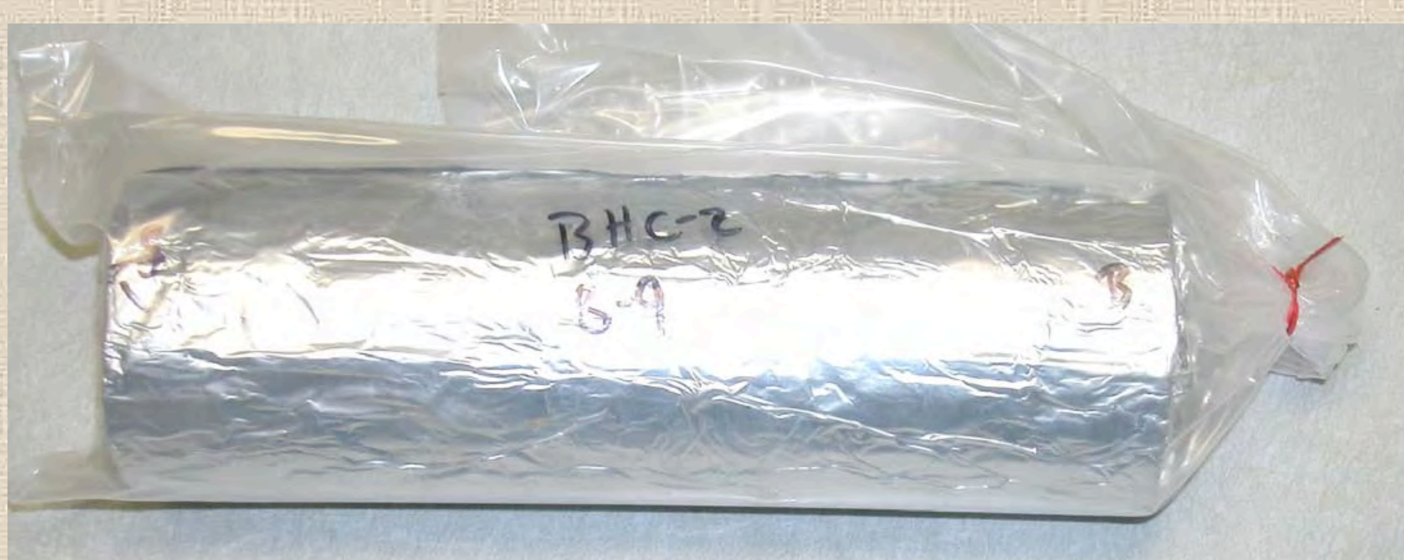
Wet Bulb Thermometer



CR7 40-Channel Datalogger

CR7 40-Channel Datalogger

Shelby Tube Soil Sample



Sample Preparation

Sample Preparation

Sample Preparation

Sample Preparation

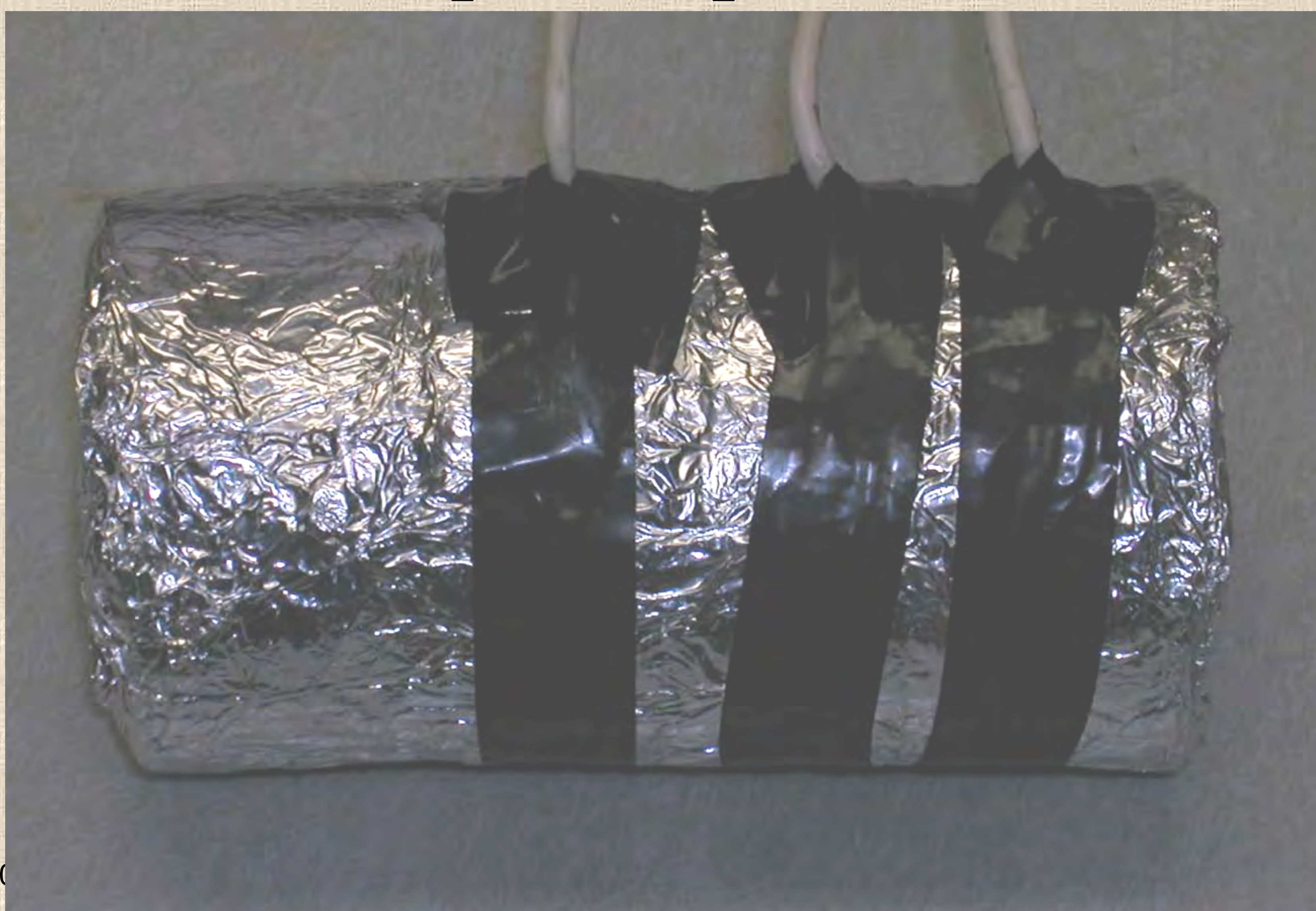
Sample Preparation

Psychrometer Installation

Psychrometer Installation

Sample Preparation

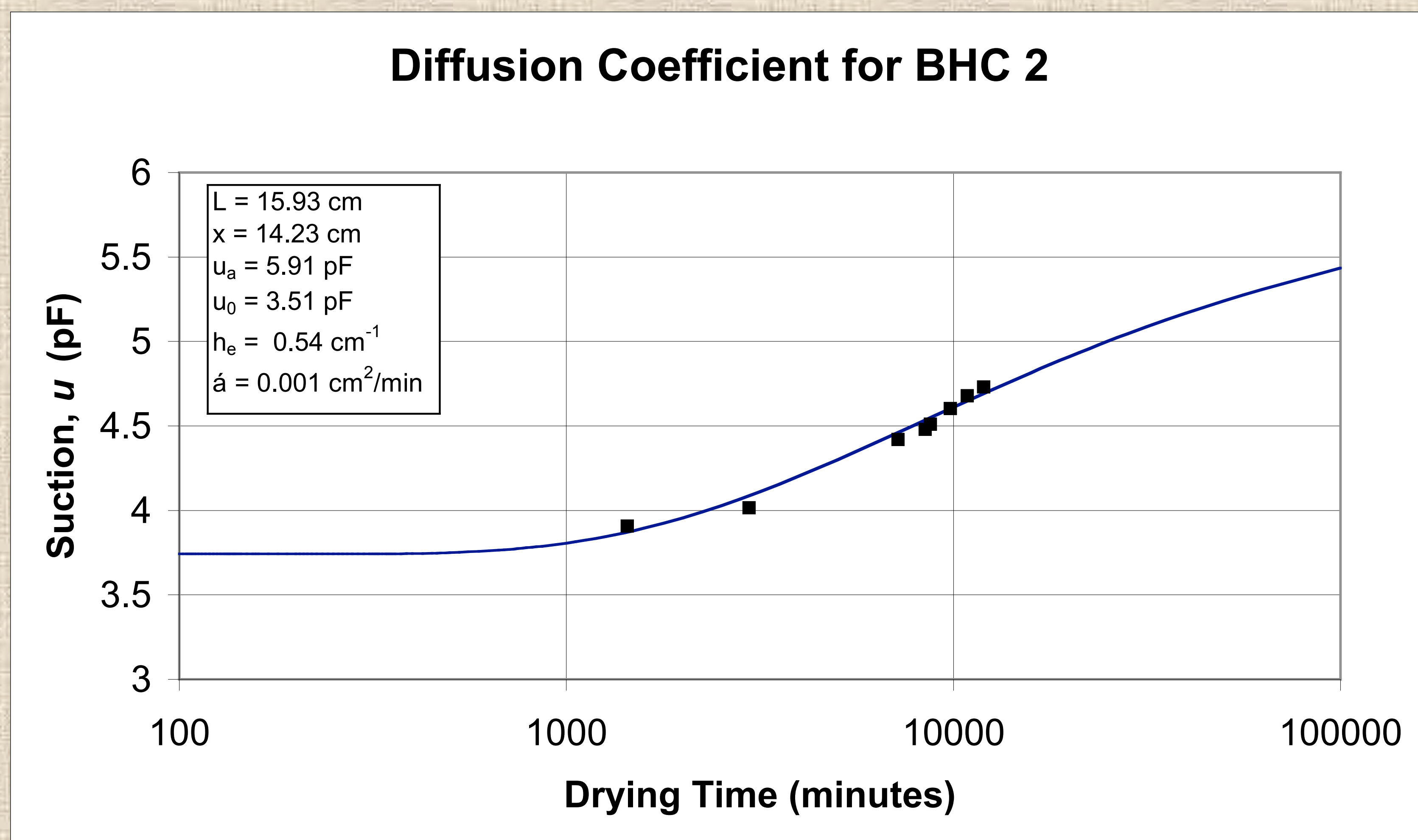
Sample Preparation



Sample Preparation

Testing in Progress

Diffusion Coefficient



Re-Evaluation of Current TxDOT PVR Procedure with A New Suction-Based Approach

Rifat Bulut, Ph.D.
Texas Transportation Institute
Texas A&M University System

Foundation Performance Association
Houston, Texas
August 10, 2005

TxDOT Project Background (2002-2004)

TTI Project Name: Design Procedure for Pavements on Expansive Soils (3 Volumes)

- *Volume I – Theoretical Background*
- *Volume II – Experimental Protocols, Case Studies Site Descriptions*
- *Volume III – Computer Programs Manuals*

PI: Dr. Robert L. Lytton

Co-PI: Dr. Charles P. Aubeny

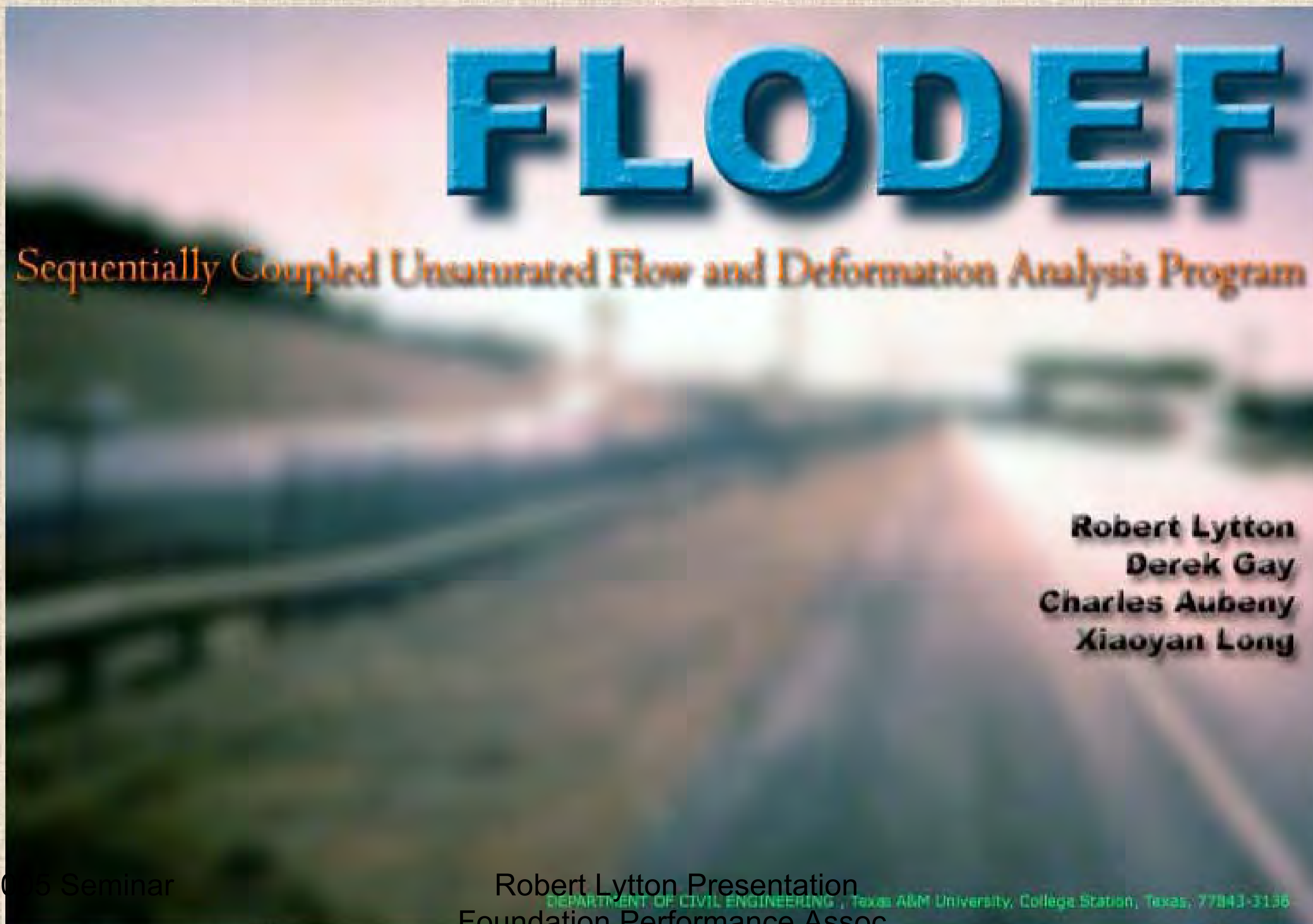
Outline

- *TxDOT PVR Assumptions*
- *Analysis Program (Flodef)*
- *Design Program (Winpres)*
- *Laboratory Testing (Diffusion Coefficient)*
- *TxDOT Case Studies*
- *PVR Comparison*
- *Implementation*

TxDOT PVR Tex-124-E Assumptions

- *Soil at all depths has access to water in capillary moisture conditions*
- *Vertical swelling strain is one-third of the volume change at all depths*
- *Remolded and compacted soils adequately represent soils in the field*
- *PVR of 0.5 inch produces unsatisfactory riding quality*
- *Volume change can be predicted by use of the plasticity index alone*

Analysis Program - Flodef



Analysis Program - Flodef

Two-Dimensional Transient Analysis For the Effects of:

- ***Vertical Moisture Barrier***
- ***Subgrade Material (Lime Stabilized / Inert Soil)***
- ***Median Condition (Paved / Non-Paved)***
- ***Shoulder Condition (Paved / Bare)***

Analysis Program - Flodef

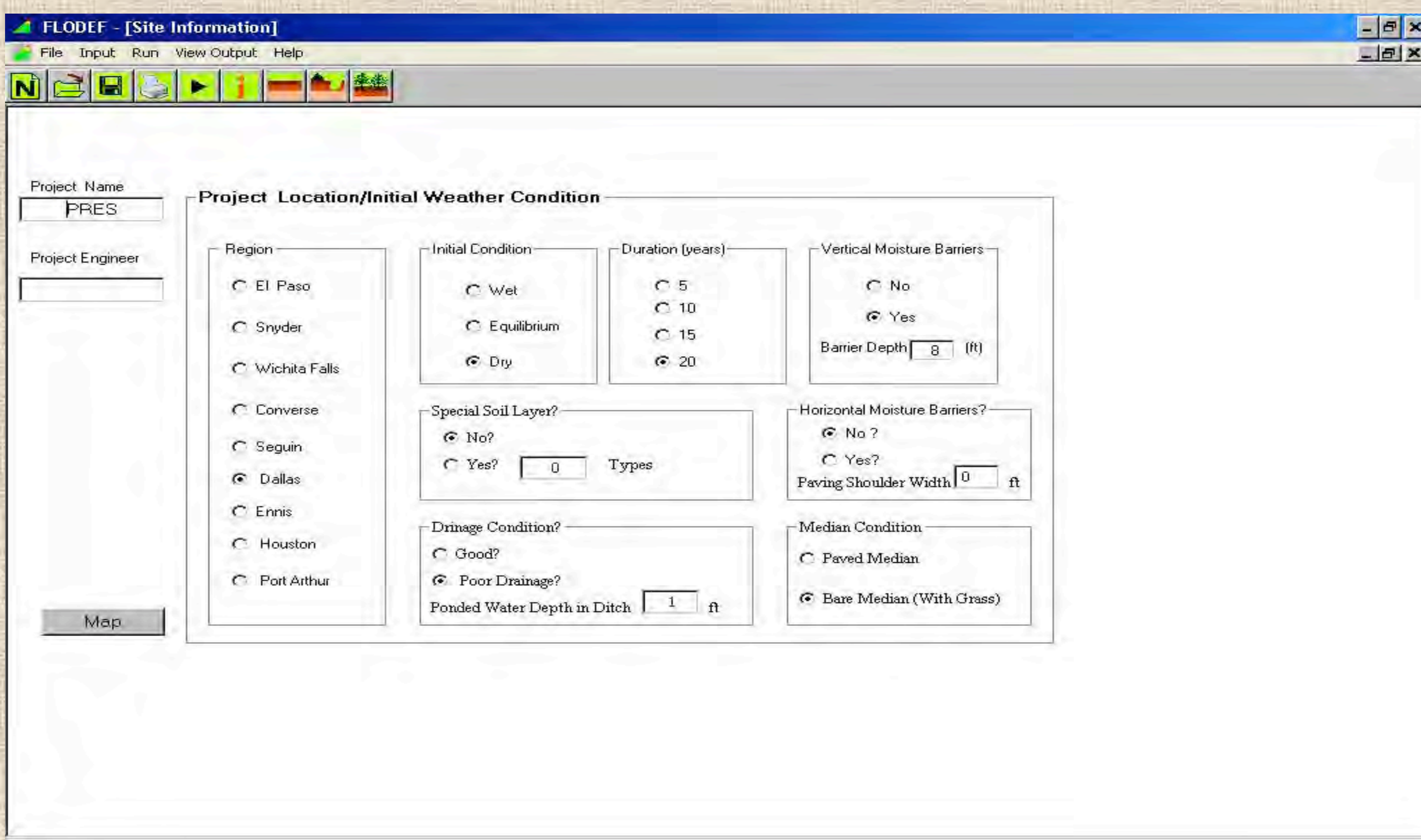
Input

- *2D Cross Section*
- *Soil Index Properties*
- *Geographic Location*
- *Vegetation*
- *Moisture Controls*
- *Drainage Conditions*

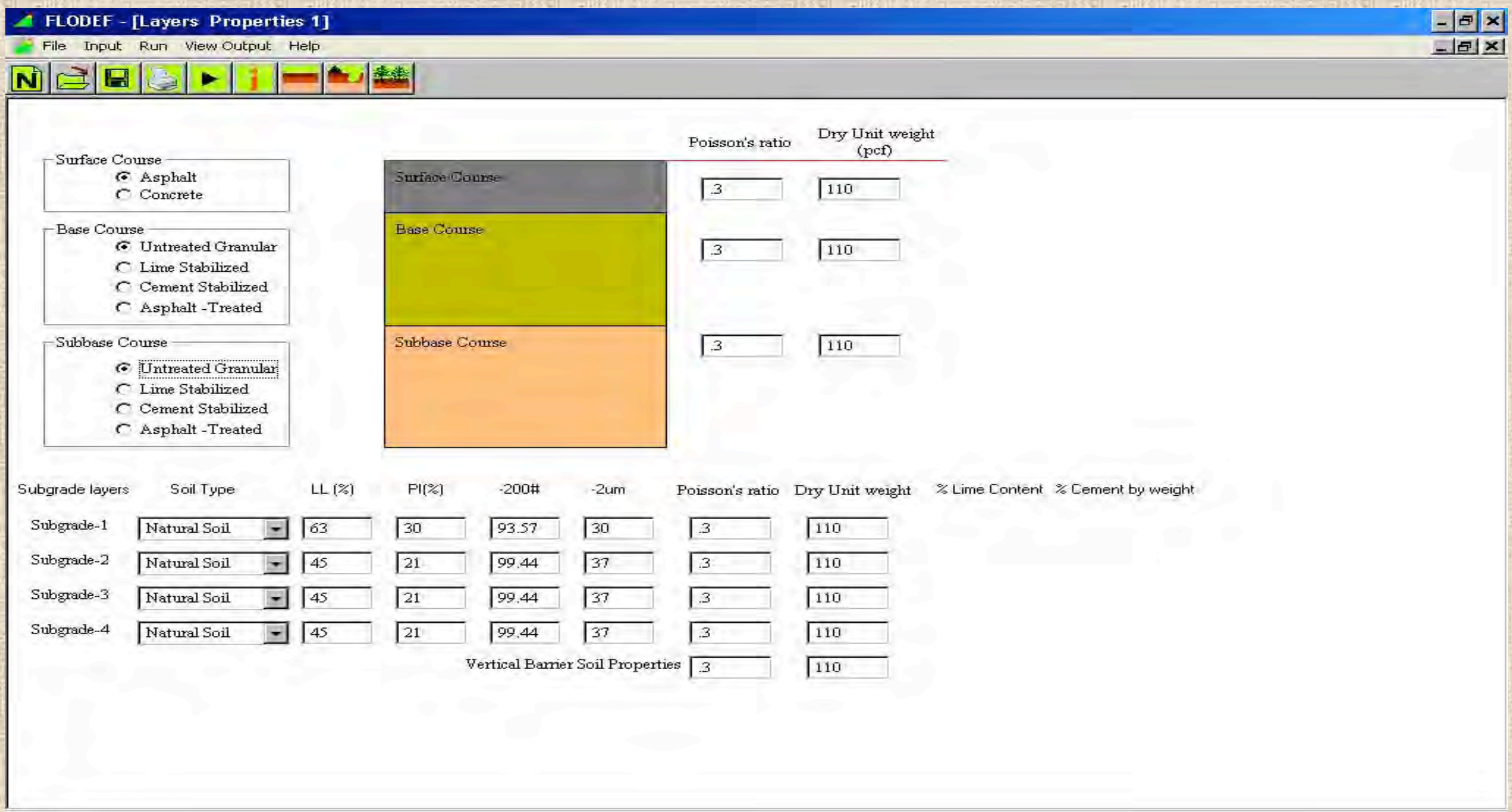
Output

- *Shrink-Swell versus Time*
- *Suction versus Time*

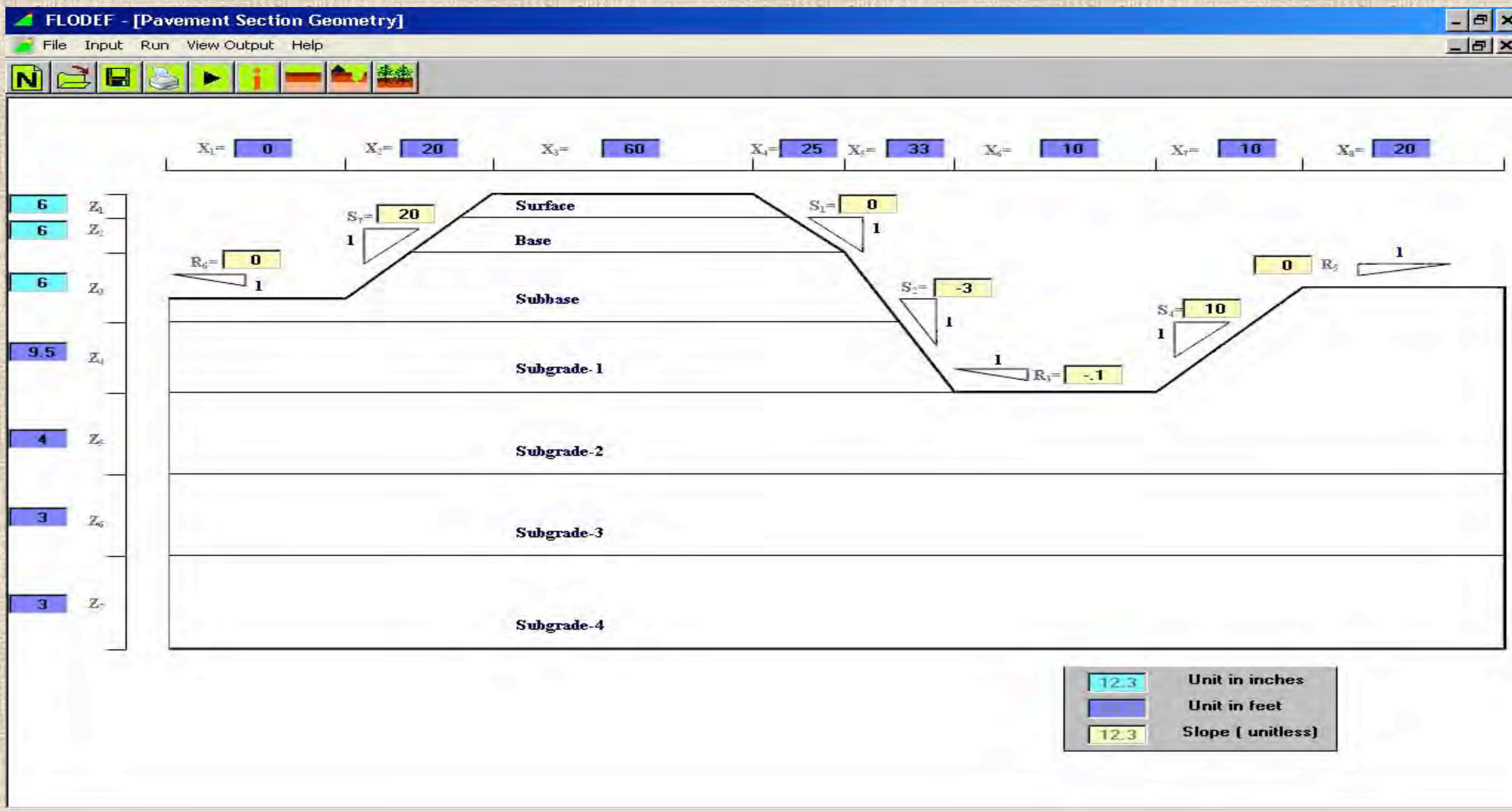
Analysis Program - Flodef



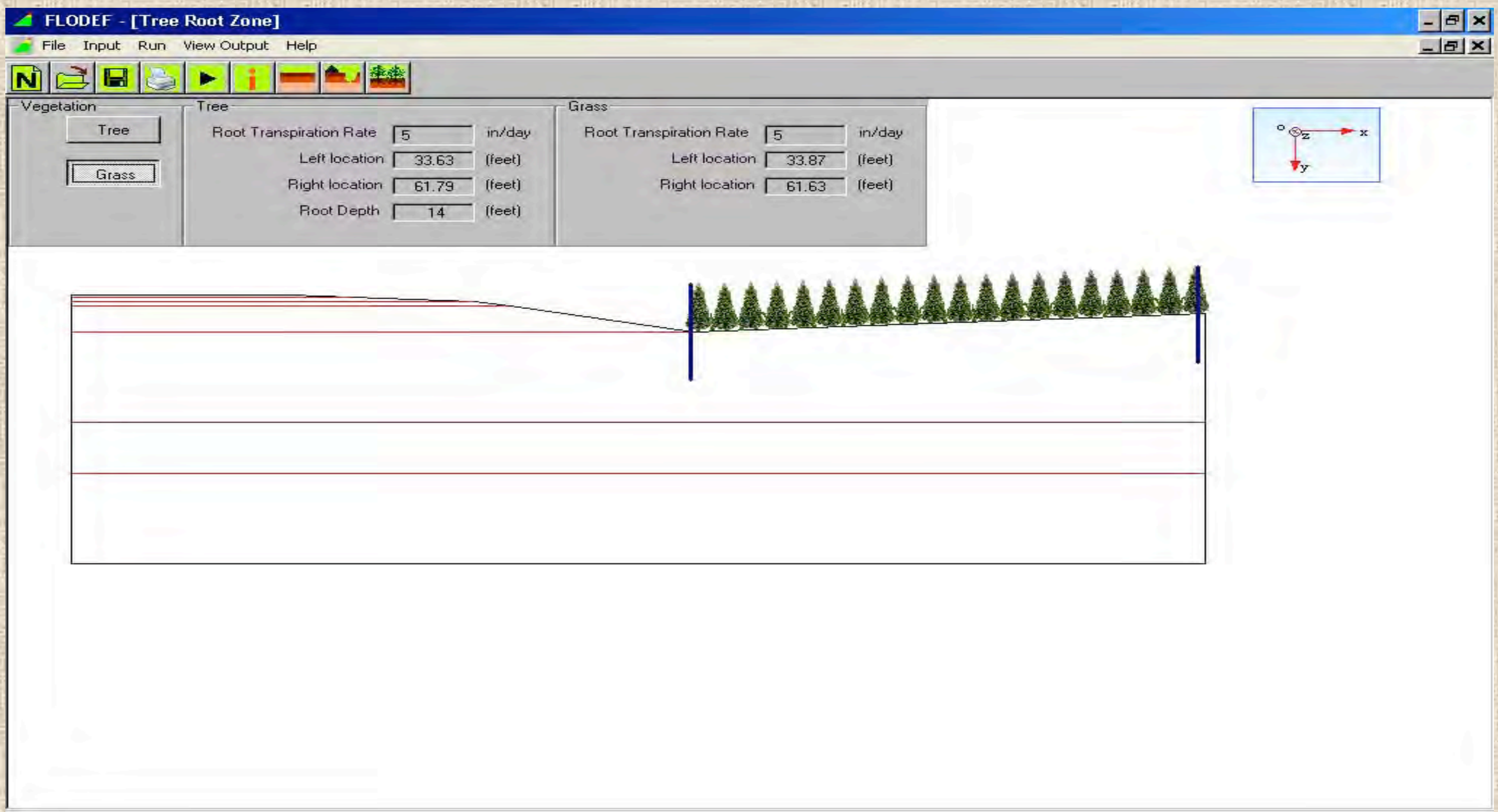
Analysis Program - Flodef



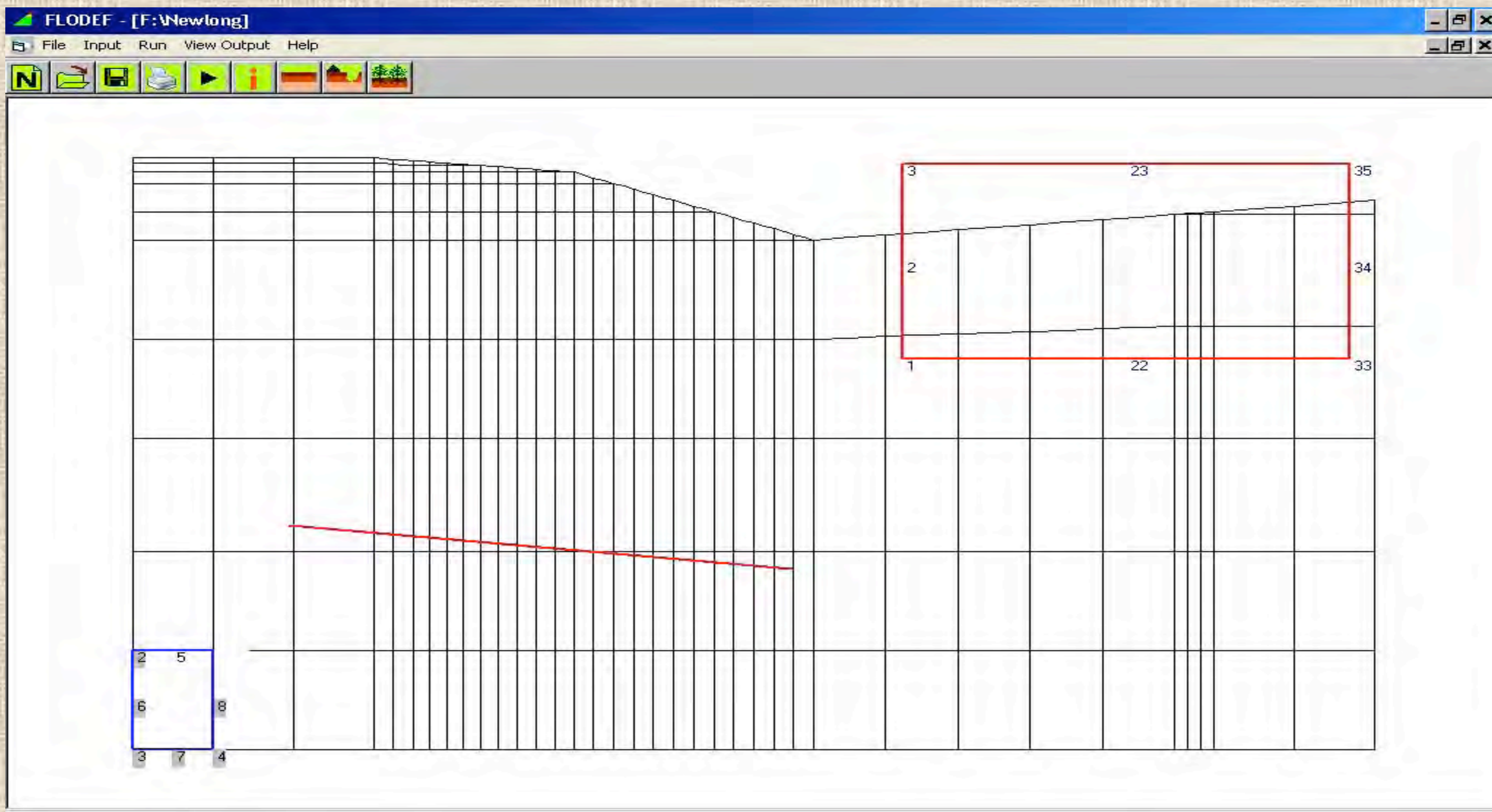
Analysis Program - Flodef



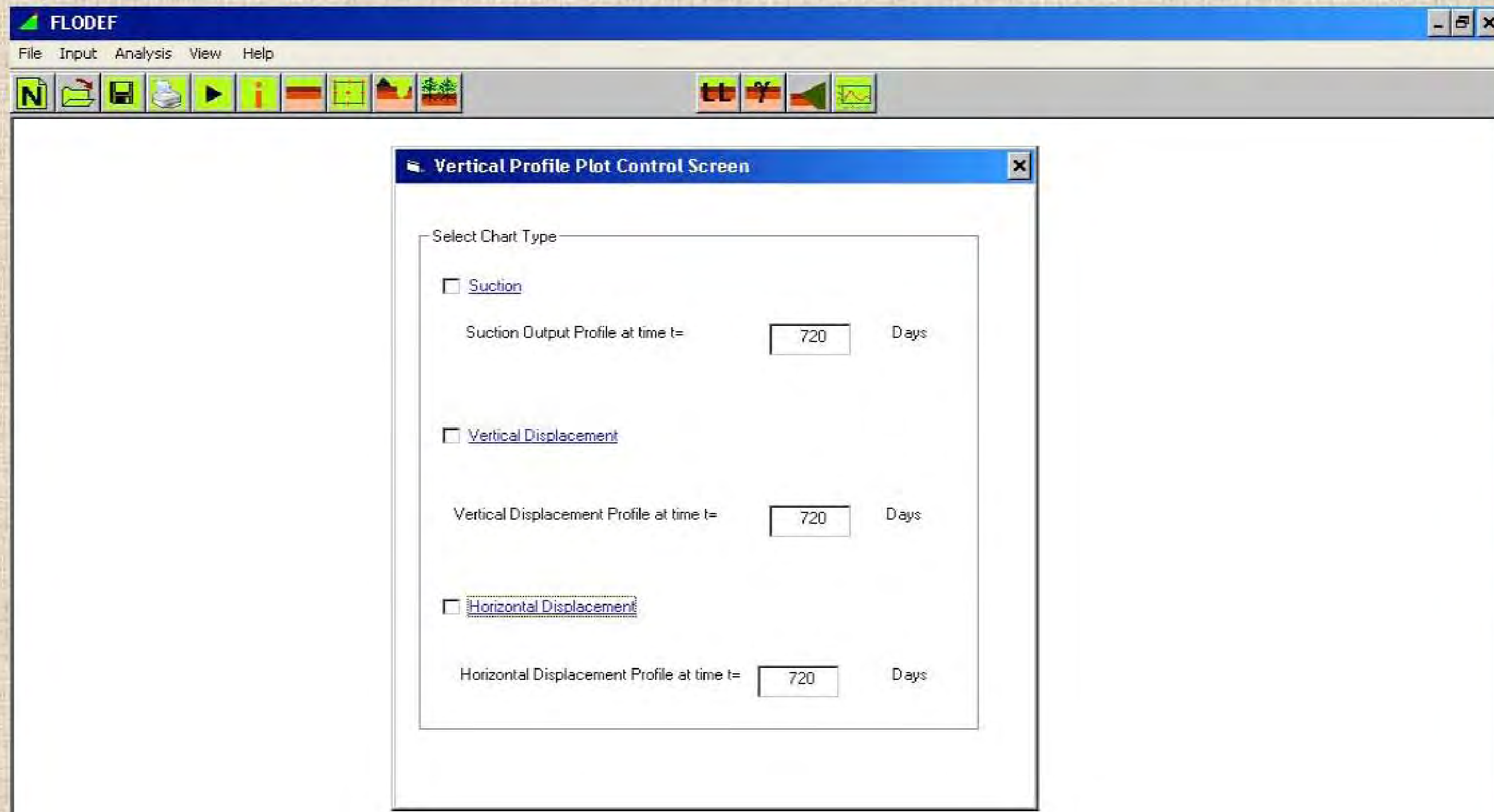
Analysis Program - Flodef



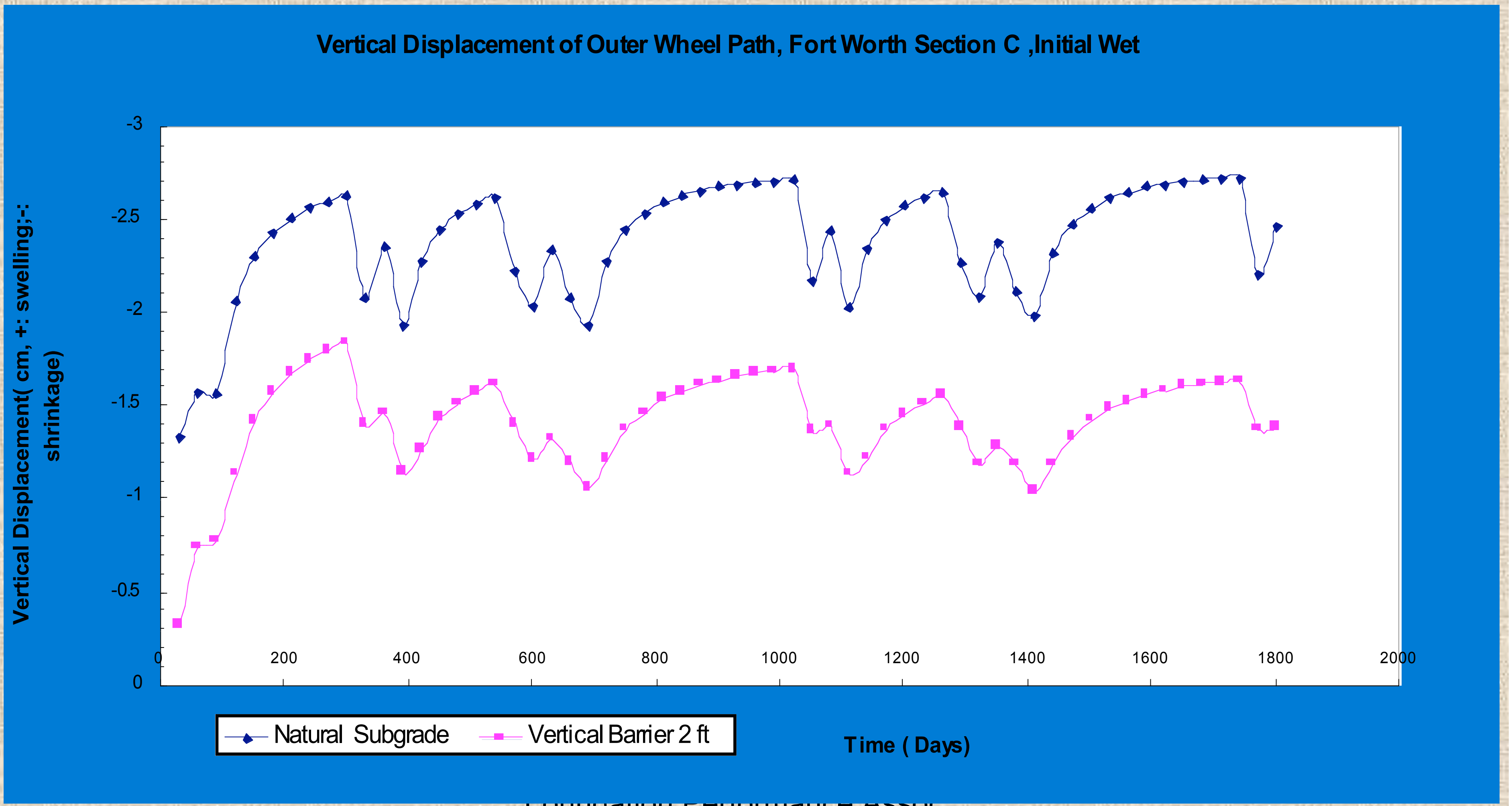
Analysis Program - Flodef



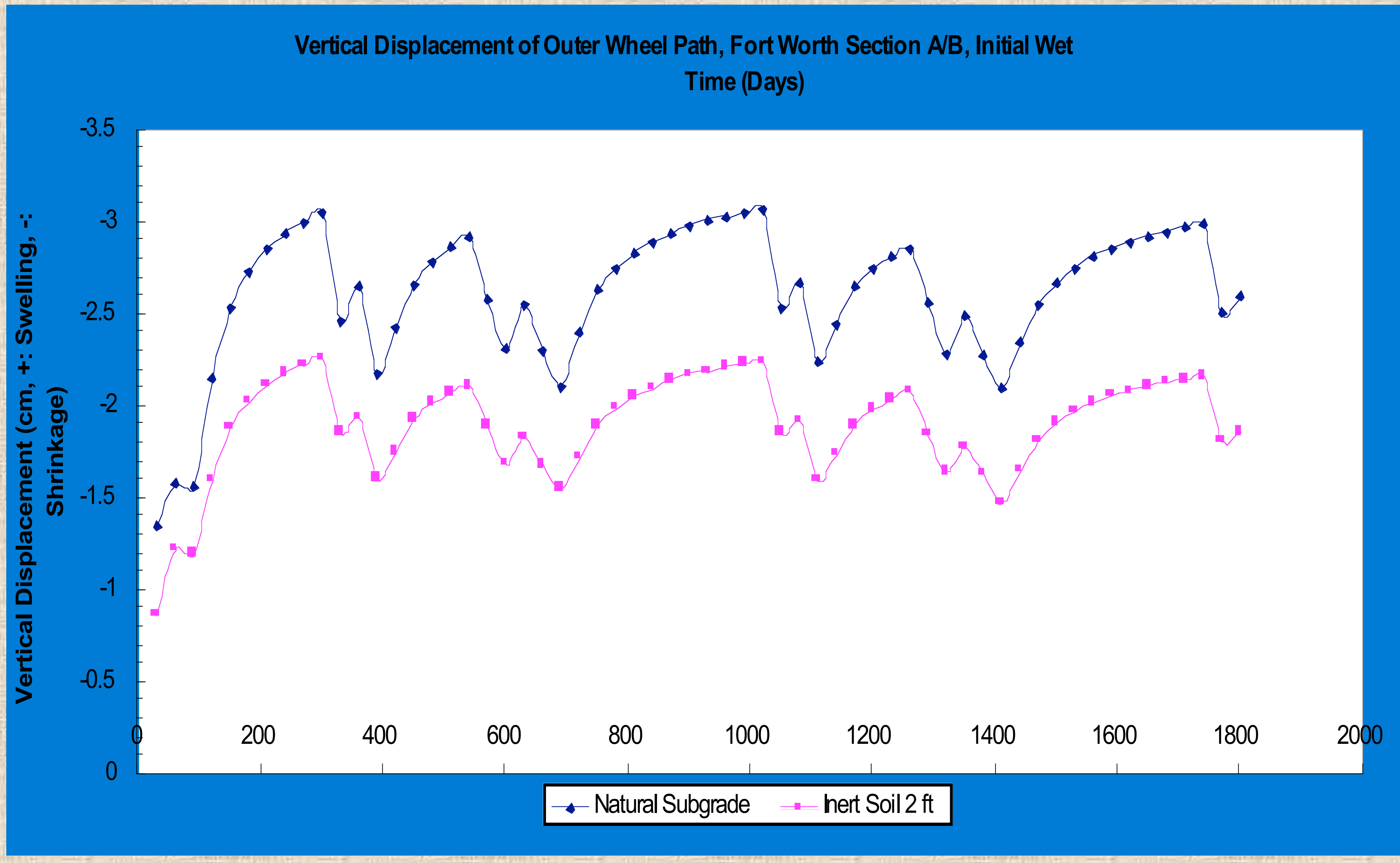
Analysis Program - Flodef



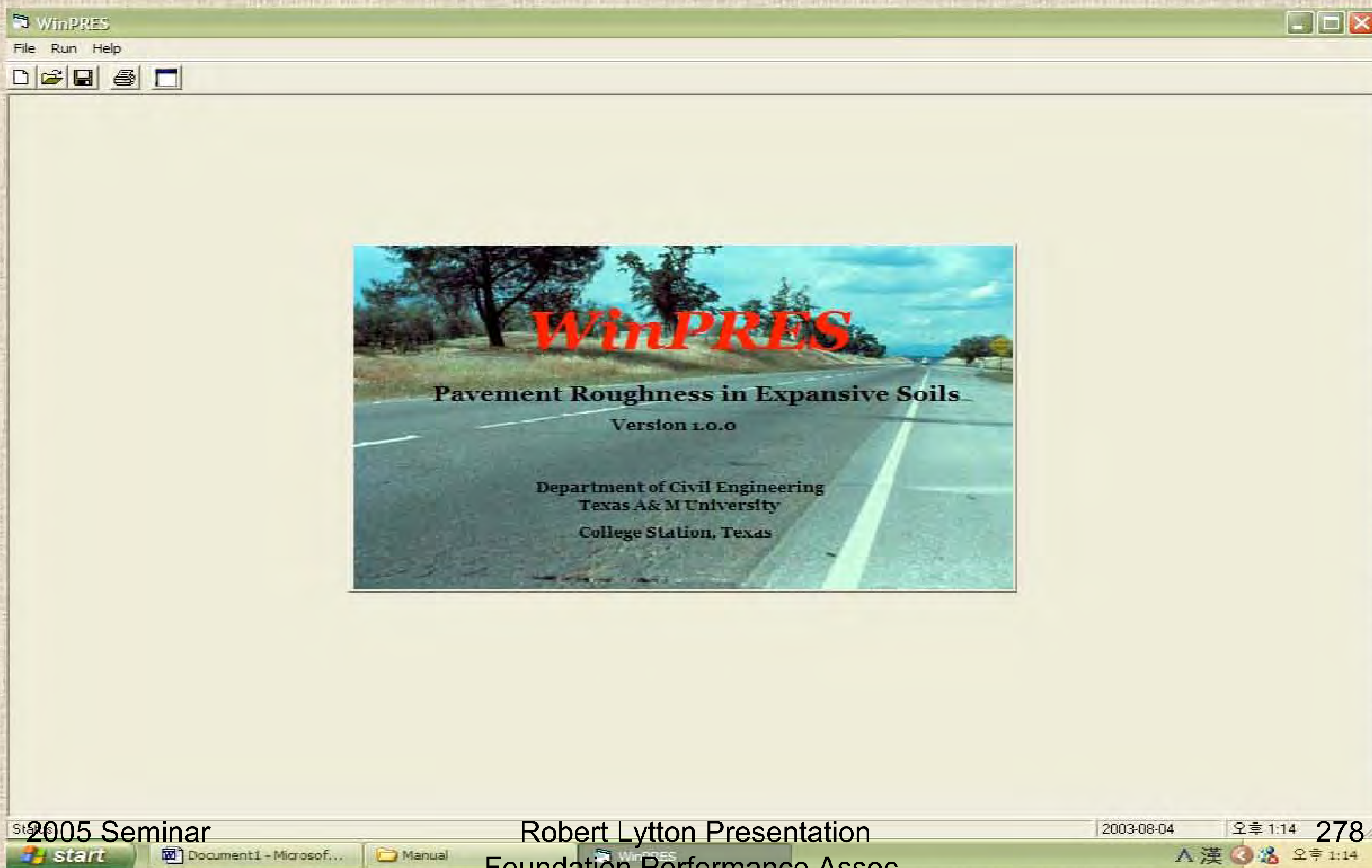
Analysis Program - Flodef



Analysis Program - Flodef



Design Program - Winpres



Design Program - Winpres

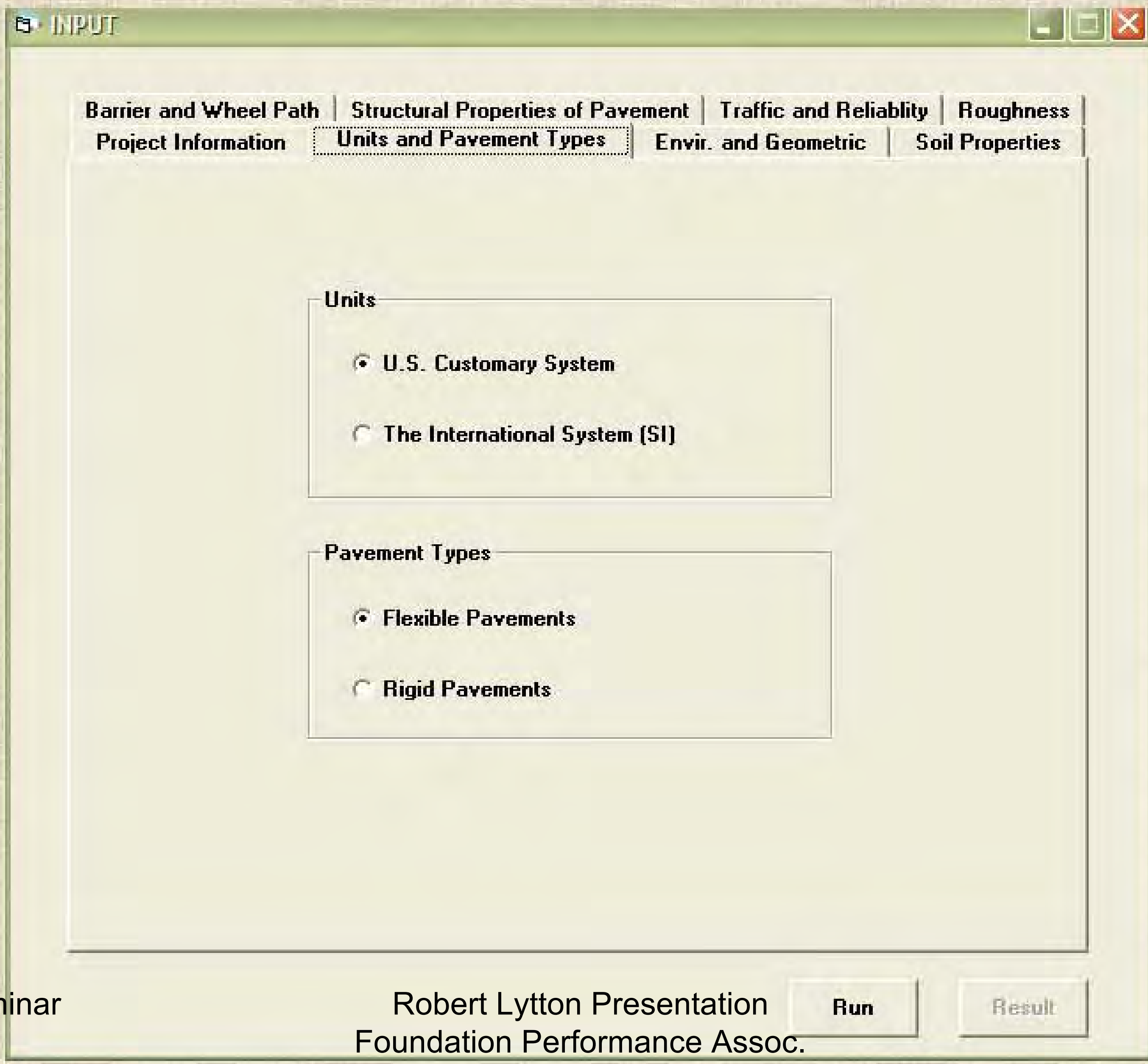
Input

- *Soil Index Properties*
- *Geographic Location*
- *Site Drainage and Vegetation*
- *Pavement Data*
- *Moisture Controls*
- *Traffic Data*
- *Reliability Level*

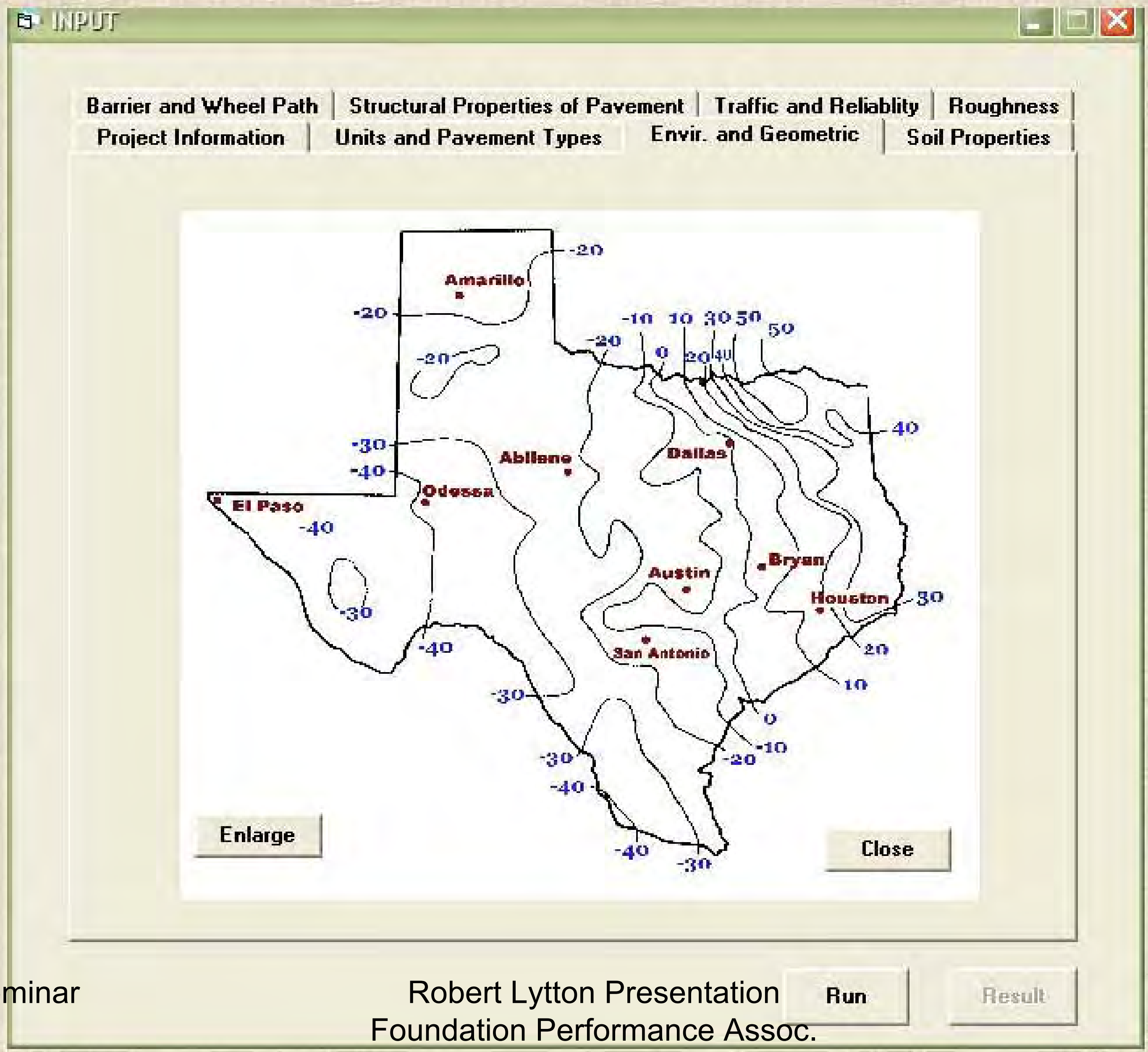
Output

- *Shrink-Swell versus Time*
- *PSI versus Time*
- *IRI versus Time*

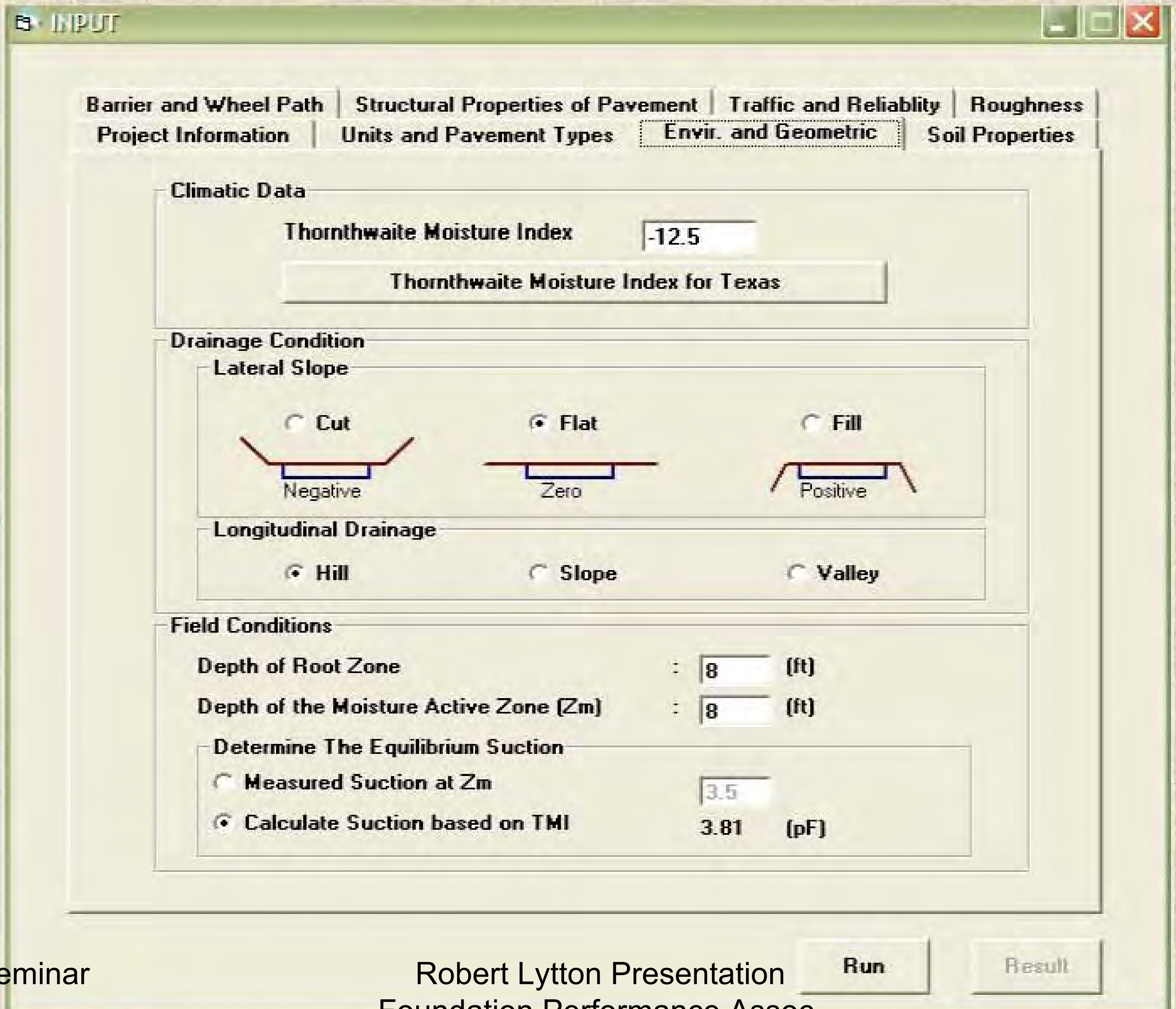
Design Program - Winpres



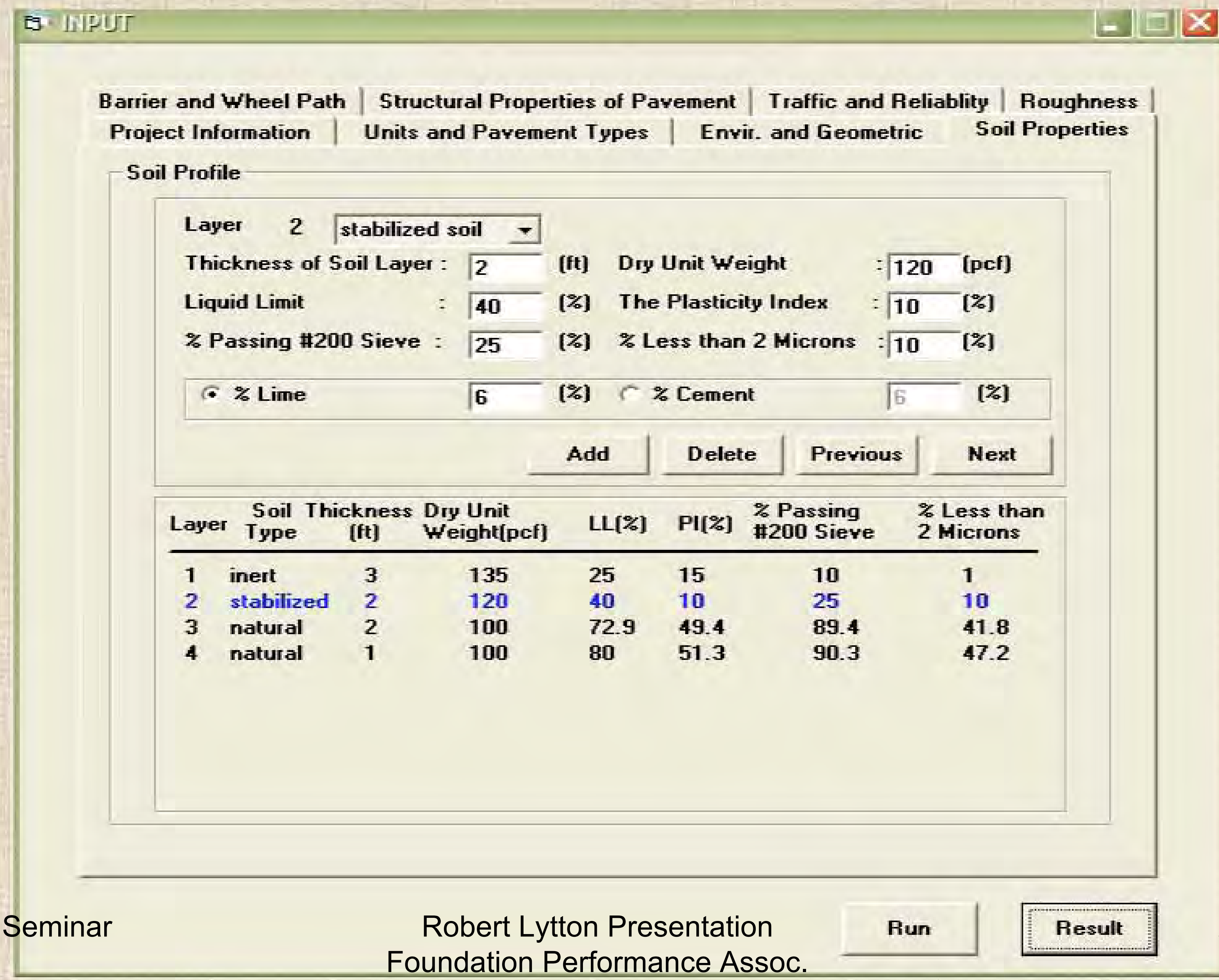
Design Program - Winpres



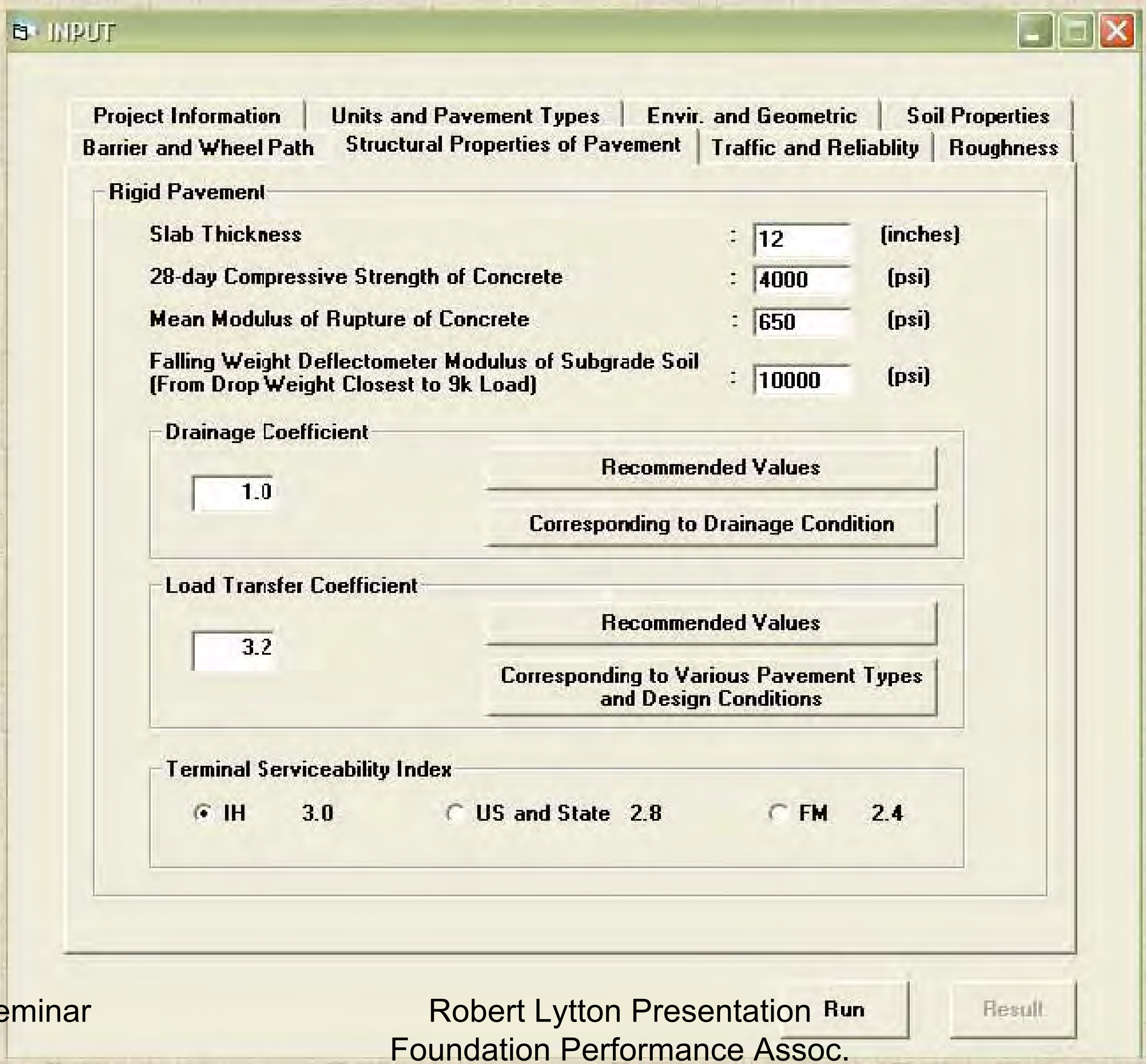
Design Program - Winpres



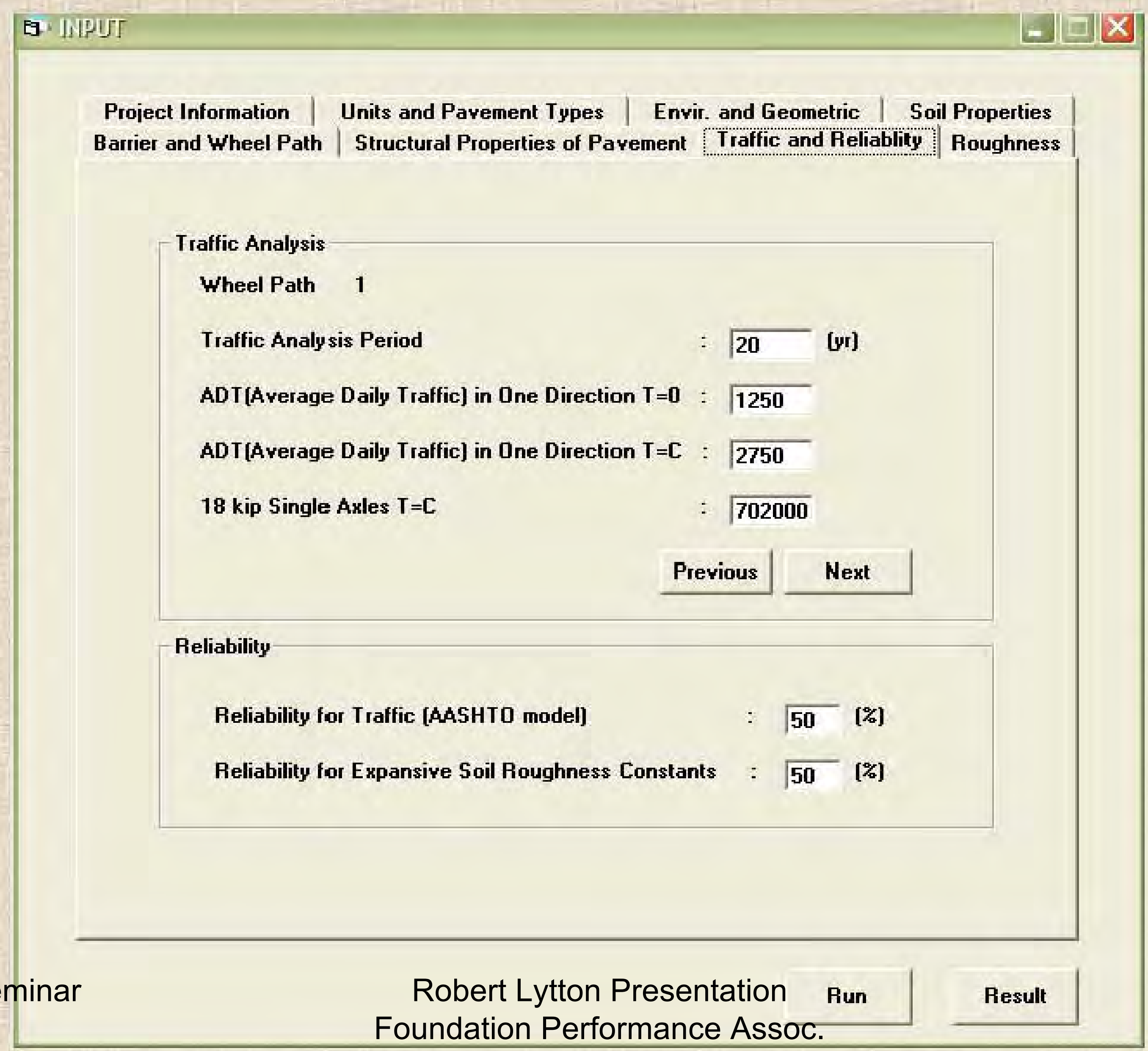
Design Program - Winpres



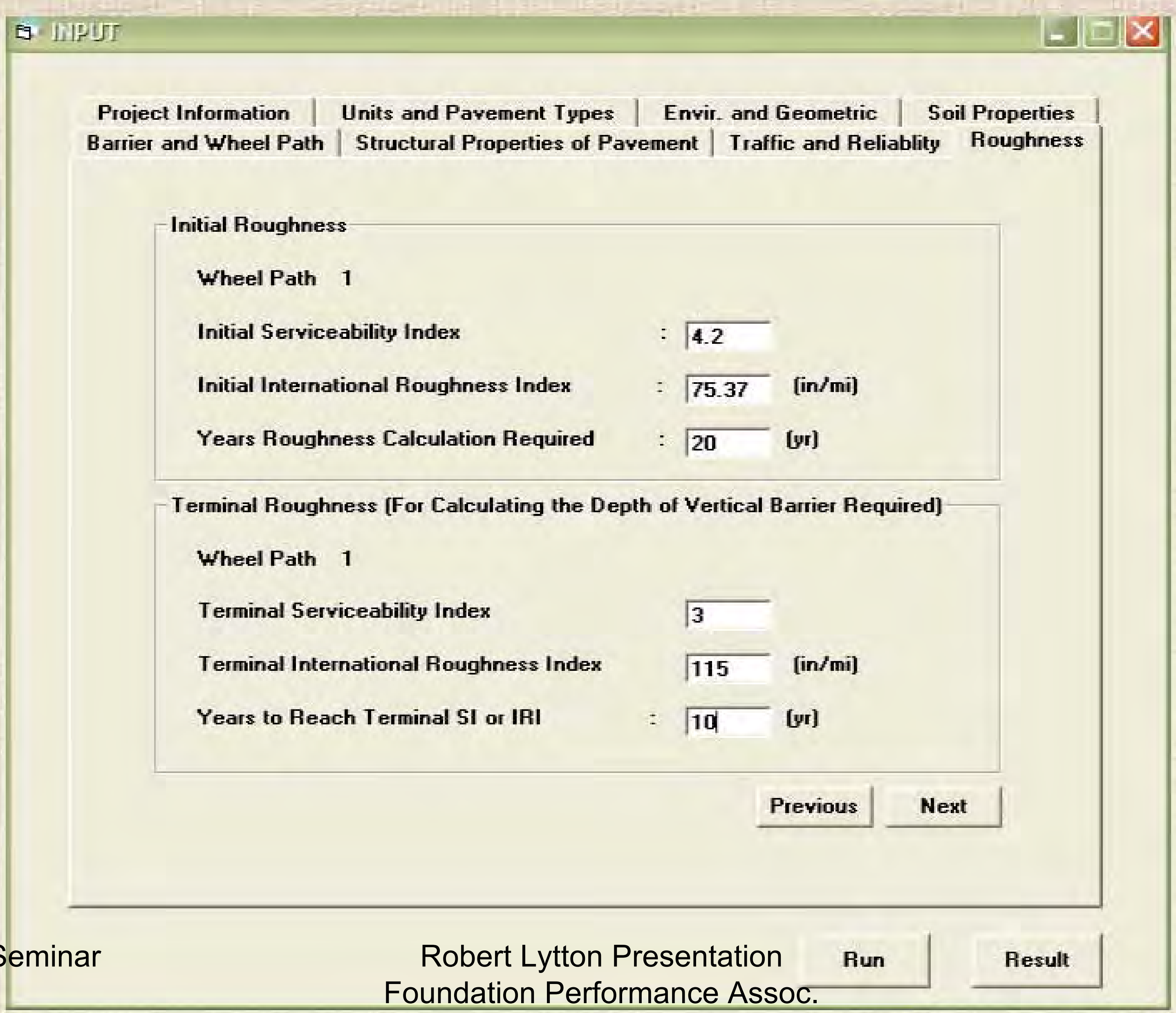
Design Program - Winpres



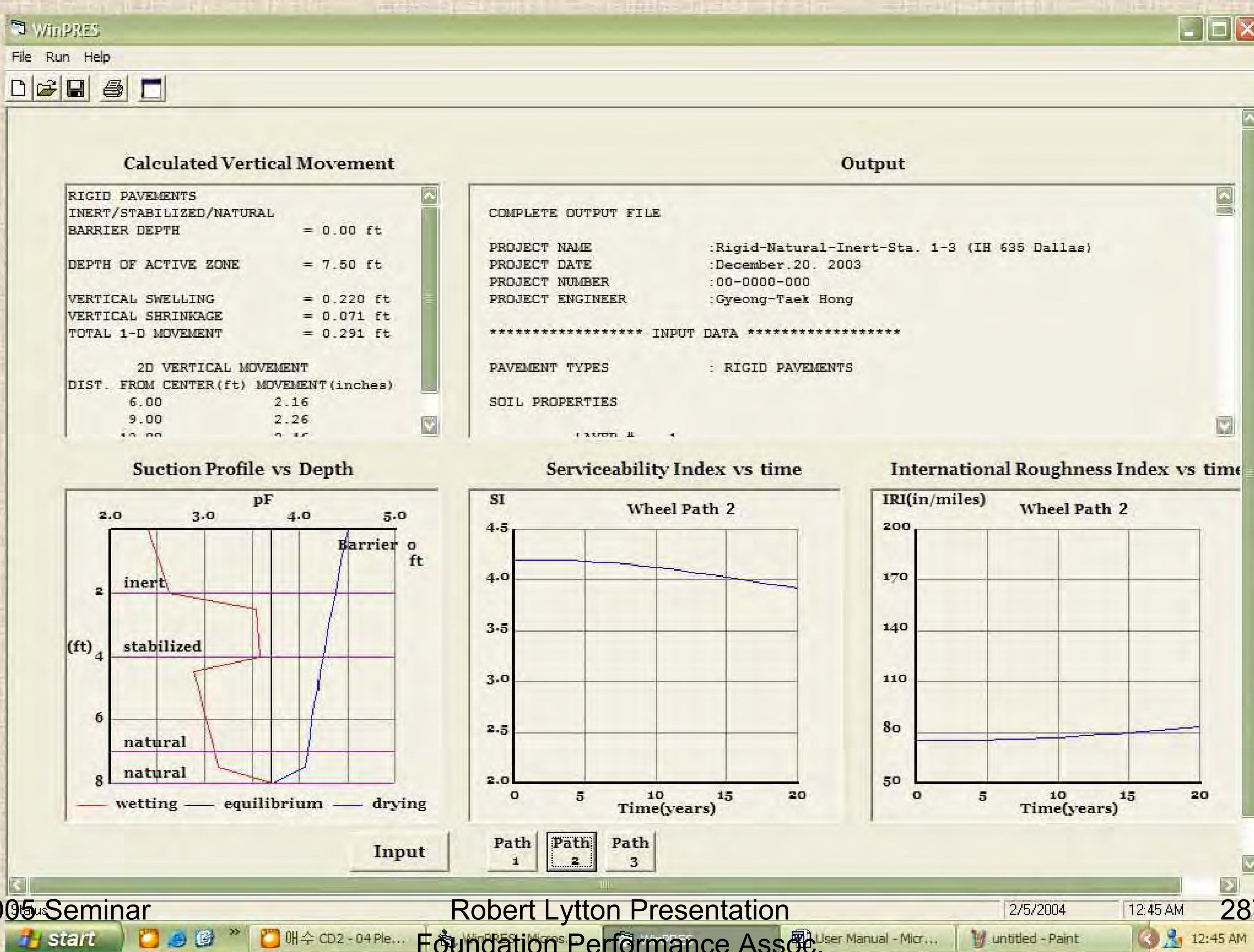
Design Program - Winpres



Design Program - Winpres

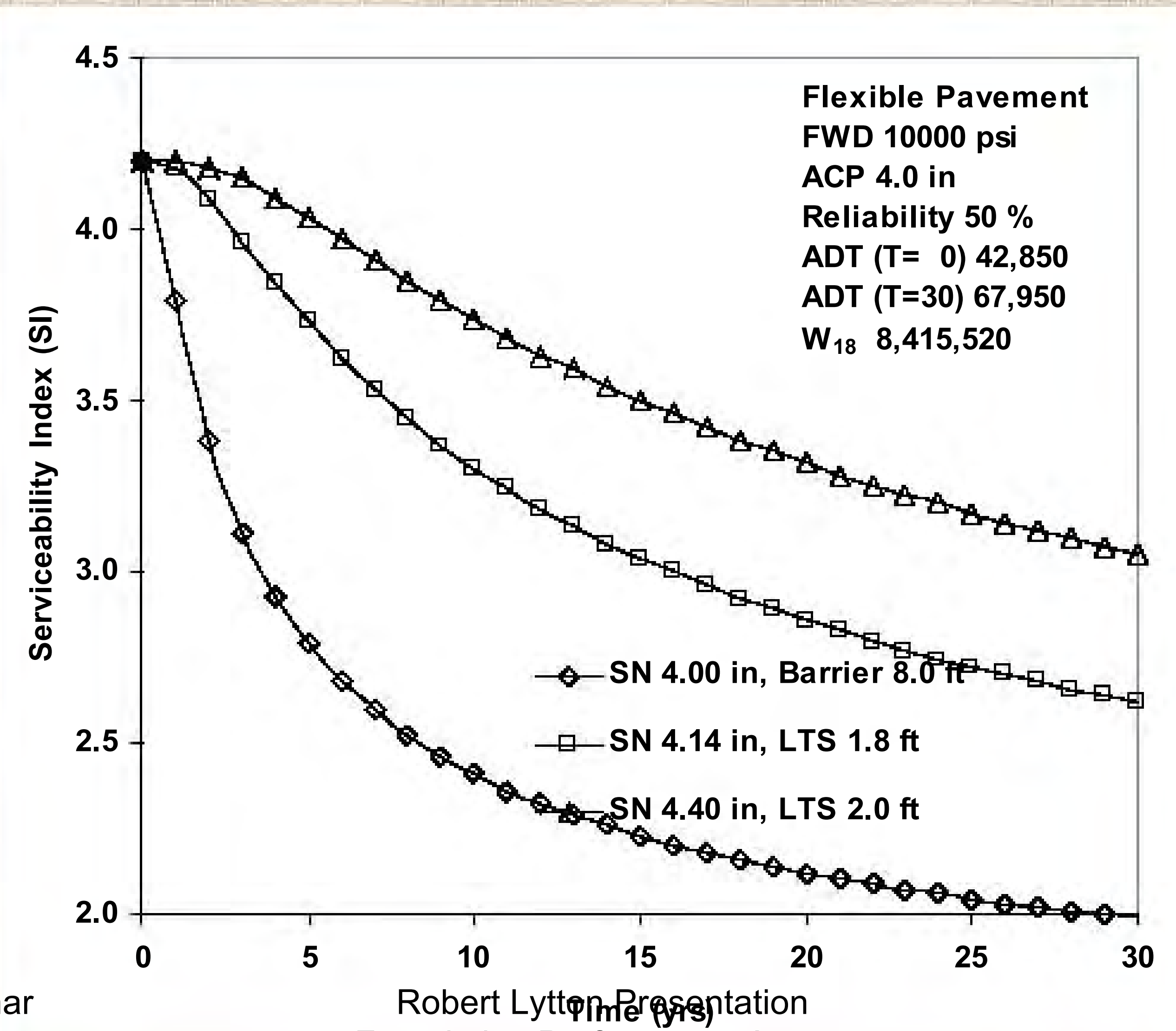


Design Program - Winpres



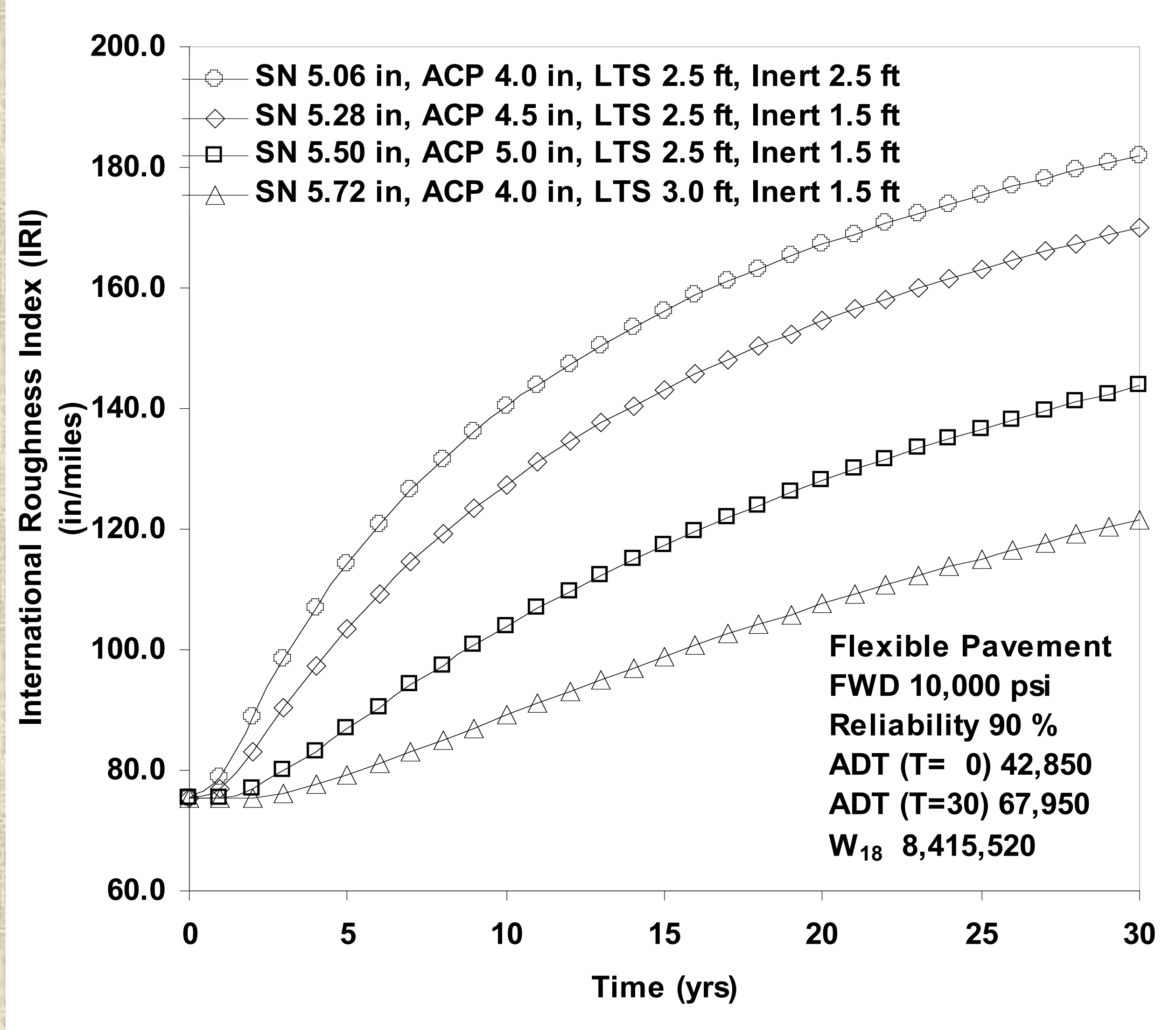
Design Program - Winpres

PSI versus Time



Design Program - Winpres

IRI versus Time

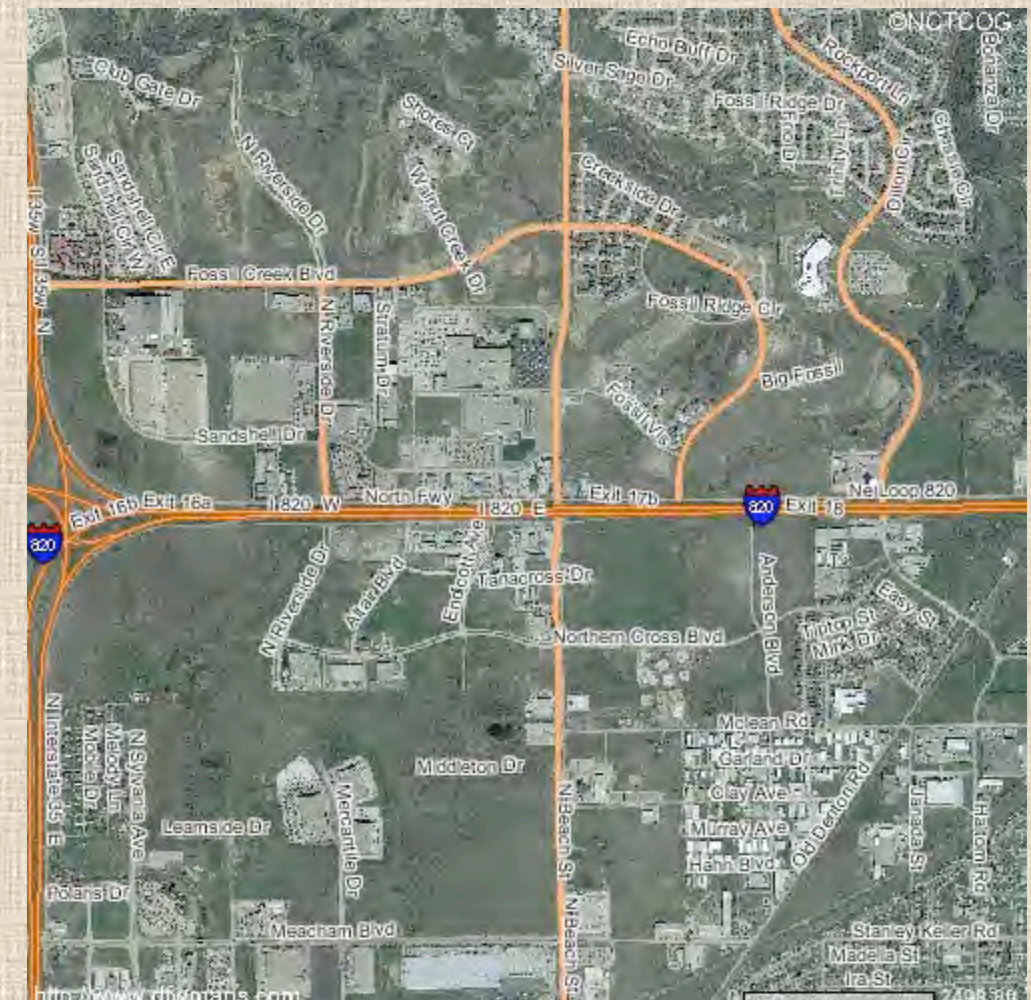


TxDOT Case Studies

- ***Fort Worth District***
- ***Atlanta District***
- ***Austin District***

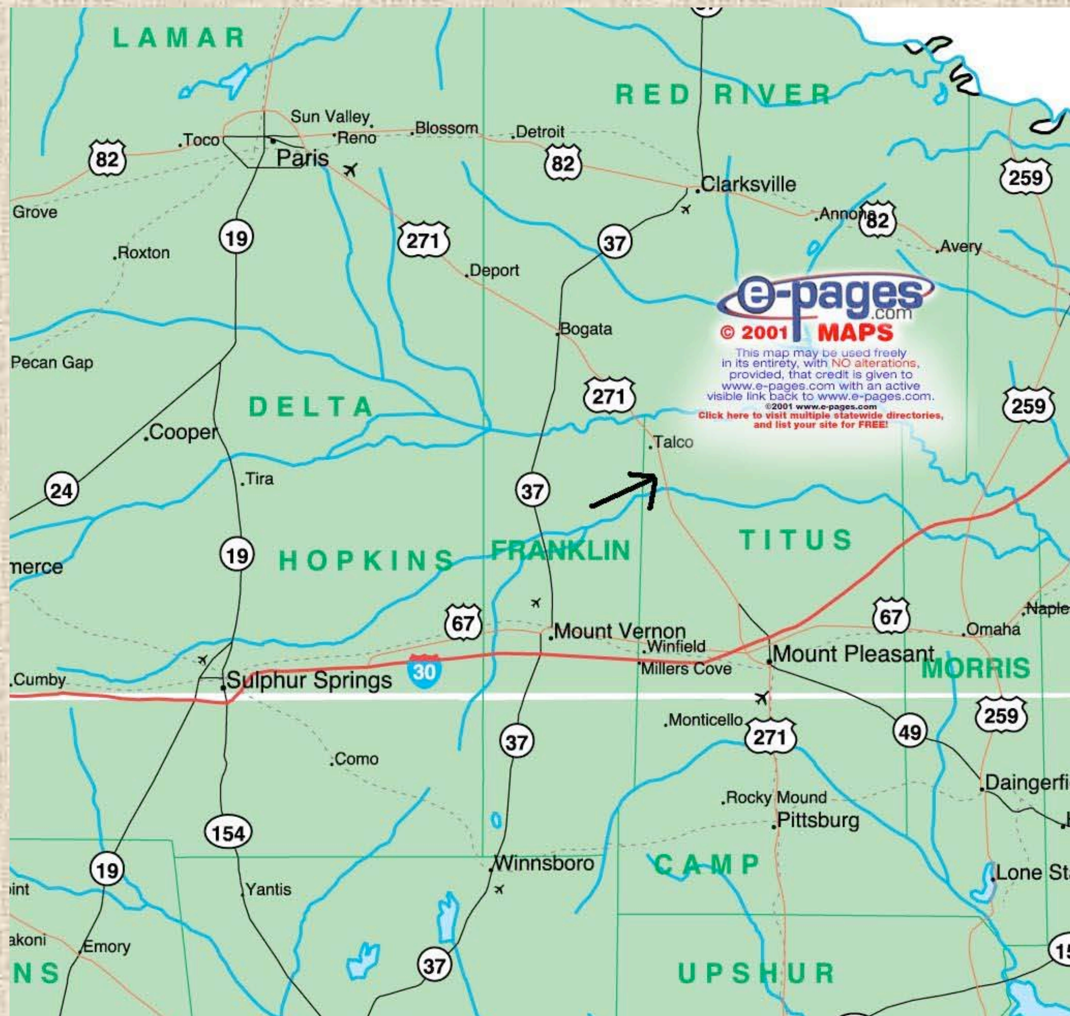
TxDOT Case Studies

Fort Worth District - North Loop 820



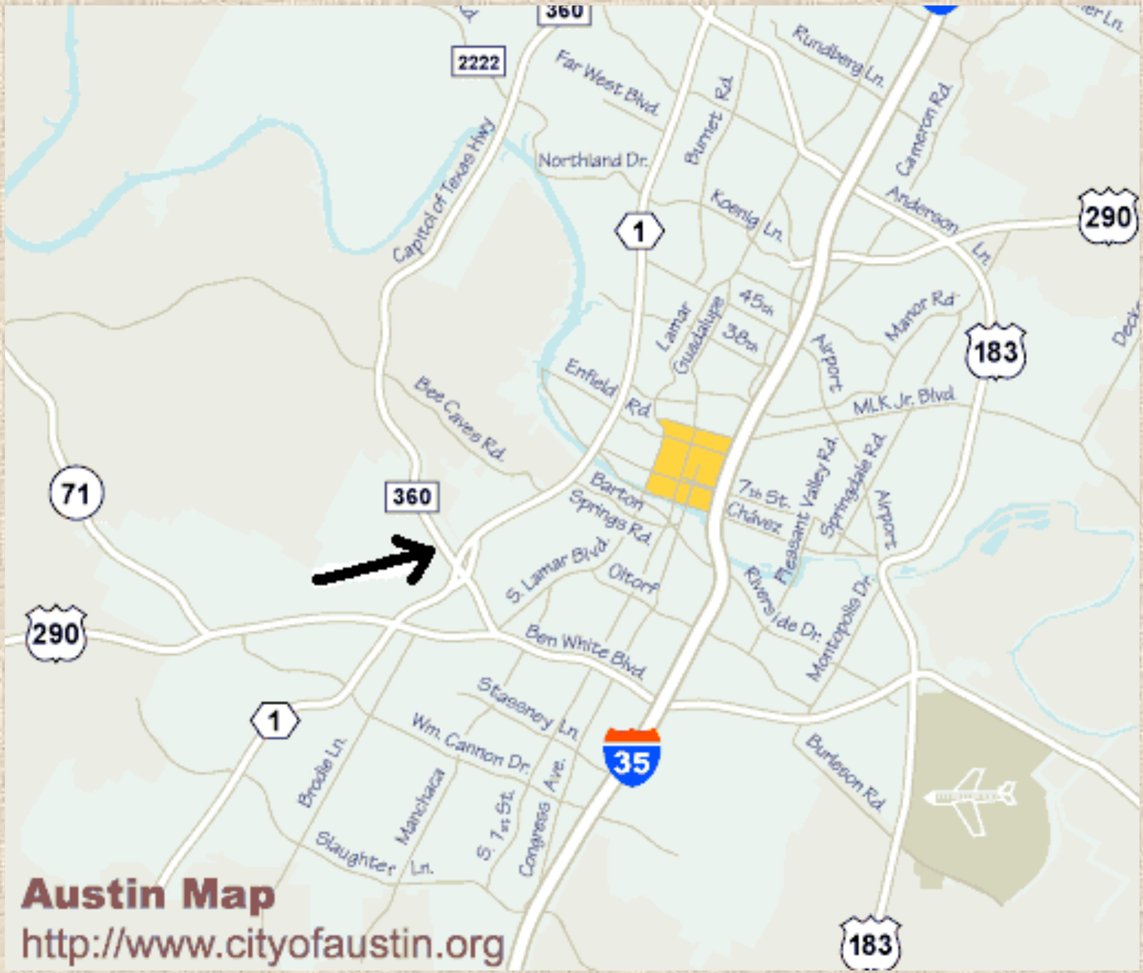
TxDOT Case Studies

Atlanta District - US 271



TxDOT Case Studies

Austin - Loop 360



TxDOT Case Studies

Index Properties

- **Atterberg Limits**
- **Clay Fraction (**Hydrometer analysis**)**
- **Fines Fraction (**Wet Sieve**)**

Suction

- **Initial** } From filter paper test
- **Matric** } From filter paper test
- **Water Content-Suction Curve**
  (From filter paper test and pressure plate apparatus)

Moisture Diffusion Coefficient

- **Diffusion Test**

TxDOT Case Studies

Sample No.	Sample Depth (m)	Liquid Limit (%)	Plasticity Index (%)	Percent Fines (%)	Initial Total Suction (log kPa)	Atmospheric Total Suction (log kPa)	Laboratory Measur ements α_{intact} (cm^2/sec)
1(A1)	3.35 -3.66	45	22	84.2	2.38	5.06	5.90E -05
2(A5)	0.91 -1.22	49	30	-	2.02	5.21	7.86E -05
3(B2)	3.35 -3.66	53	32	-	2.30	4.93	9.66E -05
4(A1)	1.52 -1.68	37	17	83.5	1.84	5.06	4.83E -05
5(C2)	2.74 -3.35	37	15	89.9	2.43	4.76	13.1E -05
6(B1)	0.61 -1.07	33	19	76.5	2.45	4.84	10.6E -05
7(B3)	2.89 -3.35	50	29	95.9	2.77	4.76	4.66E -05

TxDOT Case Studies

Sample No.	Sample Depth (m)	Liquid Limit (%)	Plasticity Index (%)	Percent Fines (%)	Initial Total Suction (log kPa)	Laboratory Measurements α_{intact} (cm^2/sec)	Field Estimates α_{field} (cm^2/sec)
1(A3)	2.74-3.04	63	43	93.6	2.25	5.05E-05	3.67E-03
2(B4)	3.96-4.26	45	21	99.4	2.56	1.08E-05	3.90E-03
3(C1)	0.61-0.91	62	36	99.7	2.28	3.73E-05	3.49E-03
4(C5)	2.13-2.43	42	19	98.2	2.81	1.73E-05	4.01E-03
5(B1)	1.07-1.52	47	29	75.3	2.53	5.65E-05	4.11E-03
6(B2)	1.98-2.43	68	48	91.8	2.39	6.30E-05	3.69E-03
7(B2)	2.89-3.26	68	48	90.6	2.21	1.07E-04	3.82E-03
8(B3)	1.07-1.52	49	29	84.9	2.46	3.21E-05	4.05E-03

Subgrade Movements for the Pavement Design with Minimum Acceptable Predicted Performance, Austin, Loop 1

Case Study Location	Type of Pavement	Acceptable Pavement Design*	Movements at the Edge of Pavement (in)			Movements in outer Wheel Path (in)	PVR (in)	
			Swell	Shrink	Tot	Total	Edge	Outer+
Main Lanes	Flexible	ACP 4.0 in LTS 2.8 ft	0.78	0.66	1.44	0.93	2.40	1.93
	Rigid	CRCP 12.0 in LTS 2.0 ft	1.03	0.76	1.79	1.19	2.54	2.10
Frontage Road	Flexible	ACP 4.0 in LTS 2.0 ft Inert 2.0 ft	0.71	0.54	1.25	0.93	2.08	1.76
	Rigid	CRCP 11.0 in	2.03	1.00	3.03	2.28	2.97	2.37

Summary

- ***Total movement controls the rate of increase in roughness***
- ***Shrink prediction alerts the designer to longitudinal cracking***

Summary of Comparisons

PVR:

- **Over-predicts swell**
- **Neglects shrink**
- **Overly conservative designs**

IMPLEMENTATION

Three TxDOT Laboratories:

- **Dallas-Fort Worth**
- **Austin**
- **Bryan**

IMPLEMENTATION

✓ Laboratory Testing Equipment
  - Filter Paper Method
  - Thermocouple Psychrometer
  - Transistor Psychrometer

✓ Training Courses
  - Computer Programs
  - Analysis and Design

THANK YOU!

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Texas A&M University System
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PREDICTION OF MOVEMENT IN EXPANSIVE CLAYS

Robert L. Lytton, Fellow, ASCE *

Abstract

The movement of expansive soils is usually due to a change of suction near the soil surface. The properties of the soil that govern the amount and rate of movement are the suction compression index, and the unsaturated permeability and diffusivity. Methods of using these to determine suction and heave (or shrinkage) profiles with depth are outlined. Methods of estimating these properties using simple laboratory tests, namely Atterberg limits, water content, dry density, porosity, sieve analysis, and hydrometer analysis are presented. Differential movement governs the design of slabs-on-ground, highway and airport pavements and canal linings, which are themselves controlled by the edge moisture variation distance as it changes with the unsaturated diffusivity and the Thornthwaite Moisture Index are presented for both the center lift and edge lift distortion mode. The values were computed using a coupled unsaturated moisture flow and elasticity finite element program which had been calibrated to match reasonably well the measured suctions in an extensive field study involving several pavement sites in a number of different climatic zones in Texas.

Introduction

The prediction of movement in expansive soils is important principally for the purpose of designing foundations or other ground supported structural elements. In design, the principal interest is in making an accurate estimate of the range of movement that must be sustained by the foundation. It is for that reason that envelopes of maximum heave and shrinkage are important for design purposes. For slab-on-ground design, differential movements are important. For highway and airport pavements, canals, and pipelines, the wave spectrum of differential movements versus wave lengths are the

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* A.P. and Florence Wiley Professor of Civil Engineering, Texas A&M University, College Station, Texas 77843

desirable design characteristic. Structural floors suspended above expansive clays must be provided with a gap that exceed the total expected heave. Drilled piers (or shafts) must be designed to resist simultaneously a vertical movement profile and a horizontal pressure profile, both of which change with wetting and drying conditions. Retaining structures, basement walls, rip rap, and canal linings must be designed to withstand lateral movements. Finally, all foundations must be designed against the time-dependent vertical and horizontal curvature that is generated by down hill creep.

Each of these types of movement is of sufficient importance and complexity to warrant a separate paper of its own. Differential movement is selected as the topic of this paper principally because it involves the prediction of the total movement at two different locations which are separated by a characteristic distance. This distance depends upon how pervious the soil is. Understanding differential movement and how to predict heave and shrinkage envelopes of it provides much of the information needed for most types of foundation design.

This paper provides results of a multiple year study of differential movements of pavements on expansive soils as they are affected by vertical moisture barriers, and of a computer study of the horizontal zone of influence that is affected by changes of moisture. The first section presents a summary of the theoretical relationships between volume change, suction change, and total stress changes. The second section summarizes material property relationships that were developed during the vertical barrier study. The material properties that can be predicted are the volume change coefficients, unsaturated permeability and diffusivity, and characteristics of the suction-versus-water content relation. The third section presents the results of the computer study of the size of the moisture influence zone for edge lift and center lift conditions. The concluding section comments upon the significance of these results for the prediction of differential movements.

Expansive Clay Volume Change

Movements in expansive soils are generated by changes of suction which is brought about by the entry or loss of moisture. The volume change that accompanies the change of suction (and water content) depends upon the total stress states that surround the soil. Within a soil mass, a decrease of the magnitude of suction results in an increase of water content. The volume of the soil also increases unless the surrounding pressure is sufficient to restrain the swelling.

Suction is defined by the Kelvin equation:

$$h = \frac{RT}{mg} \ln \frac{H}{100} \tag{1}$$

where:

- h = the total suction in gm-cm/gm, a negative number.
- R = the universal gas constant, 8.314×10^7 ergs-K/mole.
- T = absolute temperature, degrees K.
- m = gram-molecular weight of water, 18.02 gm/mole.

g = 981, conversion from grams mass to grams force.
H = relative humidity, in percent.

A complete discussion of suction is found in a book by Fredlund and Rahardjo (1), and will not be explained in more detail here.

A common measure of suction is the pF-scale, in which pF is defined as:

$$pF = \log_{10}|h| \tag{2}$$

where: $|h|$ = the magnitude of suction, a positive value.

Figure 1 illustrates the suction-vs-water content curve for a natural soil under wetting and drying conditions. Hysteresis is commonly observed between these two conditions with the water content upon wetting being lower than that upon drying at the same level of suction. The relation between the soil volume and water content rises from the dry volume to its maximum value around field capacity as long as it is not constrained from doing so by external pressure. When the water content is above the shrinkage limit, the volume change-vs-water content line is roughly parallel to the zero air voids line, gaining one cubic centimeter of volume for each cubic centimeter of water increased. Various suction levels corresponding to the field capacity (pF =2.0); plastic limit (pF = 3.5 for clays); wilting point for plants (pF = 4.5); tensile strength of water (pF = 5.3); air dry at 50% relative humidity (pF - 6.0); and oven dry (pF - 7.0) are marked on the suction-vs-water content curve.

A graph of suction-versus-volume can be drawn using the relations of each to water content. This is illustrated in Figure 2 on the plane corresponding to zero pressure. A similar graph can be drawn relating pressure (total stress) - versus-volume on the plane corresponding to zero suction. The simultaneous change of the magnitude of suction (decrease) and pressure (increase) results in a small change of volume, following the path from Point A to Point C on the pressure-suction-volume surface. The magnitude of suction decreases from Point A′ to Point B′ while the pressure increases from Point B′ to Point C′. The volume change process can be viewed as the net result of two processes:

a. Increase of volume from A to B at constant pressure.
b. Decrease of volume from B to C at constant suction.

For small increments of volume change on this surface, the volume strain, $\Delta V/V$, is linearly related to the logarithms of both pressure and | suction | . The general relation between these, and a change of osmotic suction, π, is:

$$\frac{\Delta V}{V} = \gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_\pi \log_{10}\left(\frac{\pi_f}{\pi_i}\right) \tag{3}$$

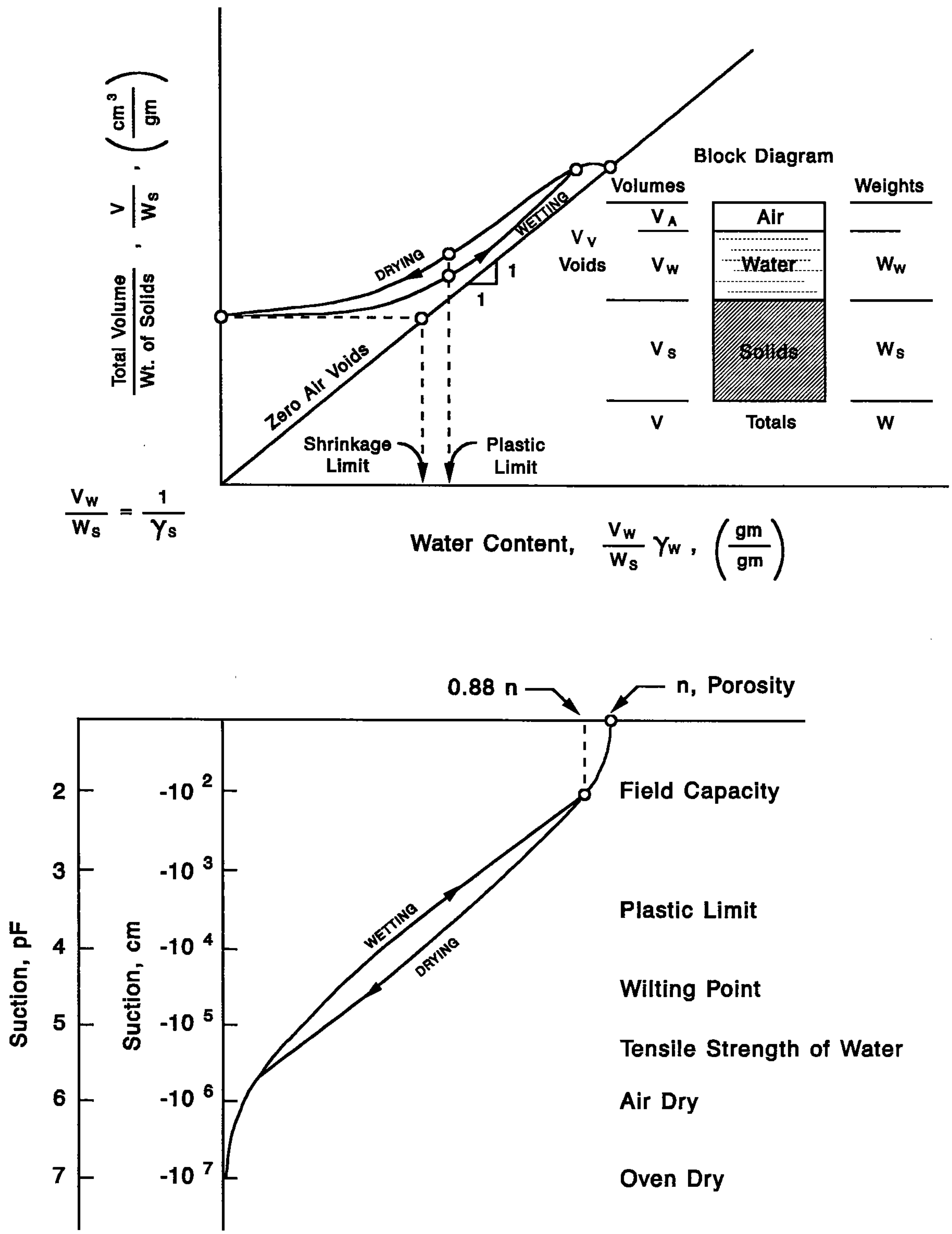

Figure 1. Suction-Water-Content-Volume Relationships in the Absence of Total Stress.

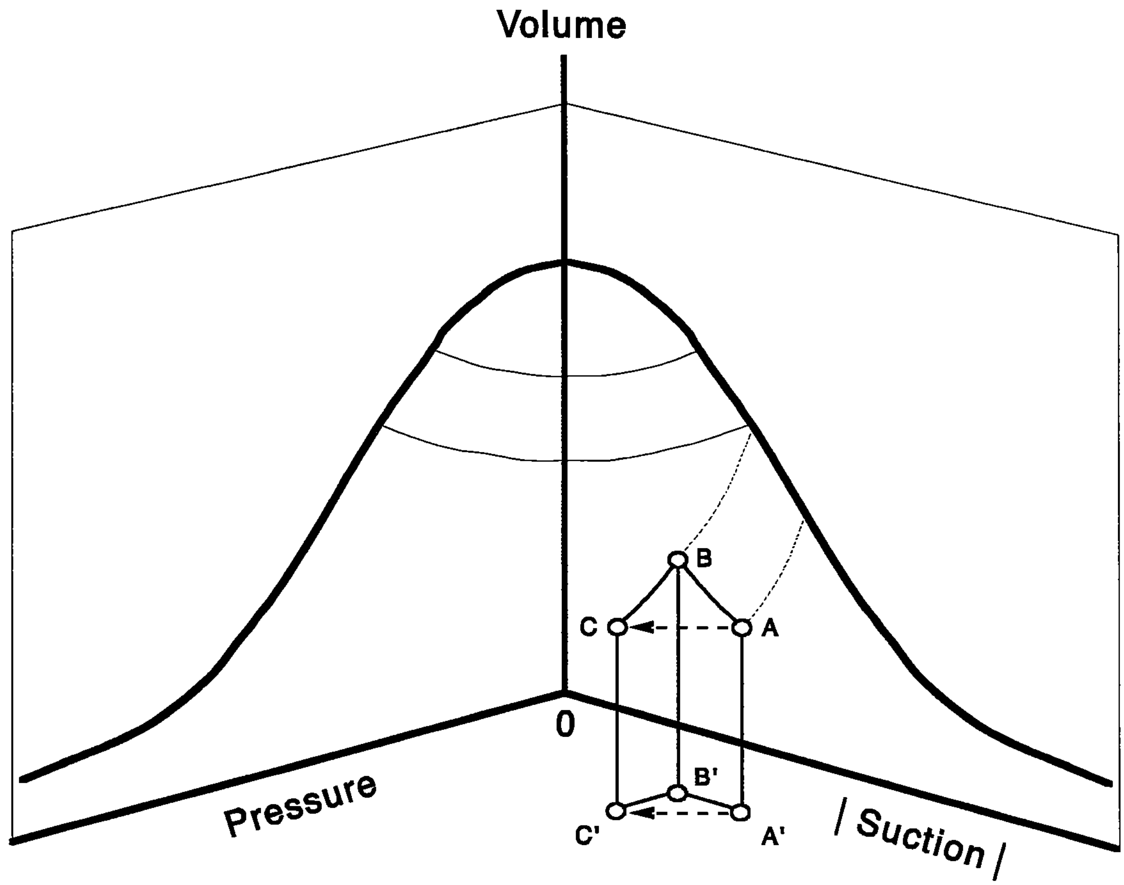

Figure 2. Pressure-Suction-Volume Surface for Expansive Soil.

in which:

$\Delta V/V$ = the volume strain.
h_i, h_f = the initial and final matrix suction.
σ_i, σ_f = the initial and final values of mean principal stress.
π_i, π_f = the initial and final values of osmotic suction.
γ_h = the matrix suction compression index.
γ_σ = the mean principal stress compression index.
γ_π = the osmotic suction compression index.

The mean principal stress compression index is related to the commonly used compression index, C_c, by:

$$\gamma_\sigma = \frac{C_c}{1+e_o} \tag{4}$$

where:

e_o = the void ratio

In order to predict the total movement in a soil mass, initial and final values of matrix suction, osmotic suction, and mean principle stress profiles with depth must be known. It is the change of matrix suction that generates the heave and shrinkage while osmotic suction rarely changes appreciably, and the mean principal stress increases only slightly in the shallow zones where most of the volume change takes place. It is commonly sufficient to compute the final mean principal stress, σ_f, from the overburden, surcharge, and foundation pressure and treat the initial mean principal stress, σ_i, as a constant corresponding to the stress-free suction-vs-volume strain line represented by Equation (3). Because there is no zero on a logarithmic scale, σ_i may be regarded as a material property, i.e., a stress level below which no correction for overburden pressure must be made in order to estimate the volume strain. It has been found to correspond to the mean principal stress at a depth of 40 cm. This is illustrated in Figure 3.

The mean principal stress is estimated by:

$$\sigma = \left(\frac{1+2K_o}{3}\right) \sigma_z \tag{5}$$

where:
σ_z = the vertical stress at a point below the surface in a soil mass.
K = the lateral earth pressure coefficient.

With an active soil which can crack itself in shrinking and generate large confining pressures in swelling, the lateral earth pressure, K_o, can vary

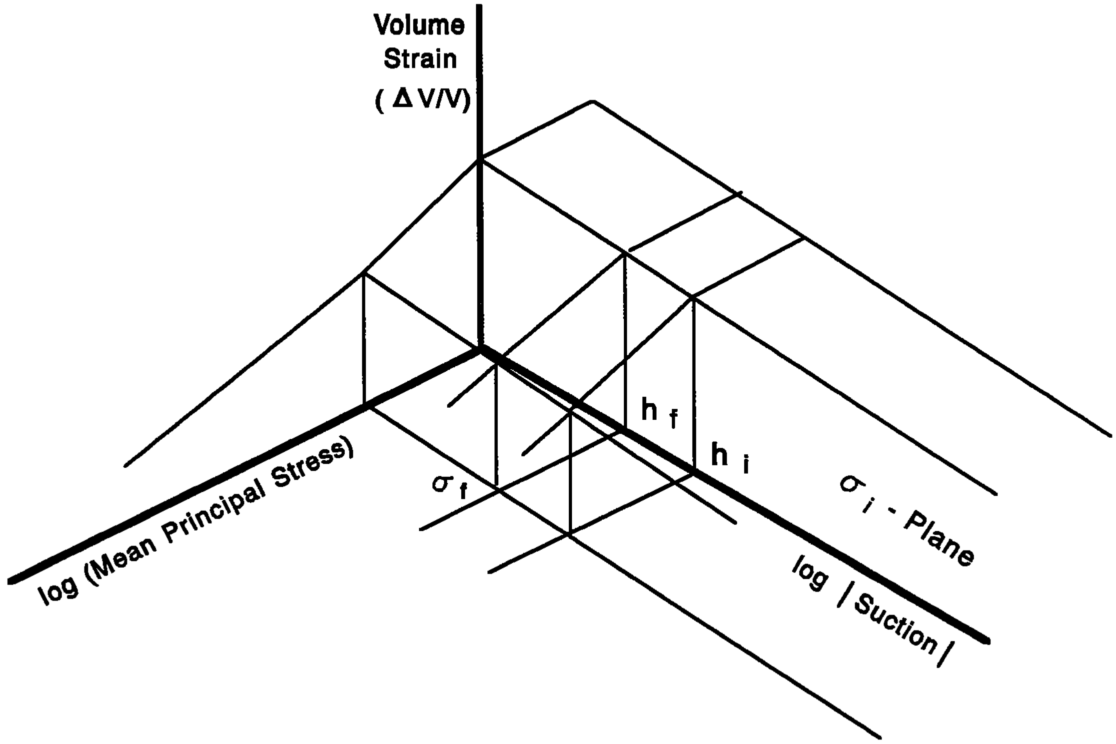

Figure 3. Volume Strain-log | Suction | -log | Mean Principal Stress | Surface.

between 0.0 and passive earth pressure levels. Typical values that have been backcalculated from field observations of heave and shrinkage are as follows:

K_o = 0.00 when the soil is badly cracked.
K_o = 0.33 when the soil is drying.
K_o = 0.67 when the soil is wetting.
K_o = 1.00 when the cracked are closely and the soil is swelling.

The vertical strain is estimated from the volume strain by using a crack fabric factor, f.

$$\frac{\Delta H}{H} = f(\frac{\Delta V}{V}) \tag{6}$$

Backcalculated values of f are 0.5 when the soil is drying and 0.8 when the soil is wetting. The level to which the lateral pressure rises is limited by the Gibbs free energy (suction) released by the water; the level to which it drops on shrinking is limited by the ability of the water phase to store the released strain energy. The total heave or shrinkage in a soil mass is the sum of the products of the vertical strains and the increment of depth to which they apply, Δz_i.

$$\Delta = \sum_{i=i}^{n} f_i(\frac{\Delta V}{V})_i \, \Delta z_i \tag{7}$$

where:

n = the number of depth increments.
Δz_i = the i^{th} depth increment.

$(\frac{\Delta V}{V})_i$ = the volume strain in the i^{th} depth increment.

The principal material property needed to compute the vertical movement is the suction compression index, γ_n. This may be estimated with the chart developed by McKeen (2), shown in Figure 4. The two axes are given by the activity ratio, Ac, and the Cation Exchange Activity ratio, CEAc, which are defined as follows:

$$Ac = \frac{PI\%}{\frac{(\%\text{-}2\ micron)}{(\%\text{-}No.200\ sieve)}} \, x\ 100 \tag{8}$$

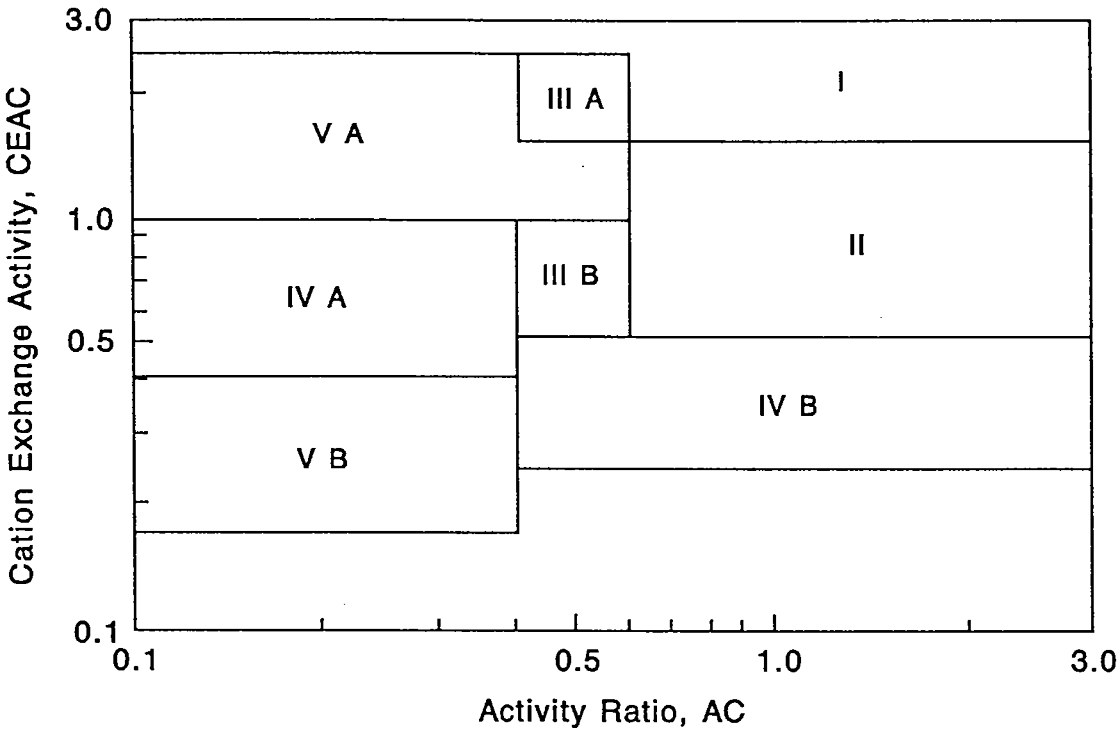

Figure 4. Chart for the Prediction of Suction Compression Index Guide Number.

$$CEAc = \frac{CEC\ \frac{milliequivalents}{100\ gm\ of\ dry\ soil}}{\frac{(\%\text{-}2\ micron)}{(\%\text{-}\ No.\ 200\ sieve)}} x\ 100 \tag{9}$$

where:

PI = the plasticity index in percent.

The denominator of both activity ratios is known as the "percent fine clay" and represents that percent of the portion of the soil which passes the No. 200 sieve which is finer than 2 microns.

The Cation Exchange Capacity (CEC), may be measured with a spectrophotometer (3) or it may be estimated with sufficient accuracy by Equation (10) which was developed by Mojeckwu (3):

$$CEC \cong (PL\%)^{1.17} \tag{10}$$

The regions on the chart each have a volume change guide number corresponding to the suction compression index of a soil with 100 percent fine clay. Values of the guide numbers are given in Table 1. The actual suction compression index is proportional to the actual percent of fine clay in the soil. Thus the actual γ_h is:

$$\gamma_h = \gamma_o\ x\left(\frac{\%\text{-}\ 2\ micron}{\%\text{-}\ No.\ 200}\right) \tag{11}$$

for the soil portion finer than the No. 200 sieve.

The mean principal stress compression index, γ_o, is related to γ_h by the following equation:

$$\gamma_\sigma = \gamma_h \frac{1}{1 + \frac{h}{\theta\ \left(\frac{\partial h}{\partial \theta}\right)}} \tag{12}$$

where: Θ = the volumetric water content.

$\frac{\partial h}{\partial \theta}$ = the slope of the suction-versus-volumetric water content curve.

Suction Profiles

For design purposes, it is desirable to compute the total heave that occurs between two steady state suction profiles, one given by a constant velocity of water entering the profile (low suction levels due to wetting) and the other given by a constant velocity of water leaving the profile (high

Table 1. Values for a Soil with 100% Fine Clay Content.

Region	Volume Change γ_o Guide Number
I	0.220
II	0.163
IIIA	0.096
IIIB	0.096
IVA	0.061
IVB	0.061
VA	0.033
VB	0.033

suction levels due to drying). Steady state conditions are given by Darcy's law:

$$v = -k \left(\frac{\partial H}{\partial Z}\right) \tag{13}$$

The total head, H, is made up of the total suction, h, and the elevation head, Z:

$$H = h + Z \tag{14}$$

The gradient of total head is:

$$\frac{\partial H}{\partial Z} = \frac{\partial h}{\partial Z} + 1 \tag{15}$$

Solving for the change of suction as a function of the change of elevation gives:

$$\partial h = -\partial Z \left(1+\frac{v}{k}\right) \tag{16}$$

Use of Gardner's equation (4) for the unsaturated permeability gives:

$$\Delta h = -\Delta Z \left[1+ \frac{v}{k_o} (1+ a\,|h\,|^n) \right] \tag{17}$$

where a, n = 10^{-9}, 3.0 typically.
k_o = saturated permeability, cm/sec.

The sign of the velocity, v, is positive for water leaving the soil (drying) and negative for water entering the soil. Using Mitchell's equation (5) for the unsaturated permeability gives:

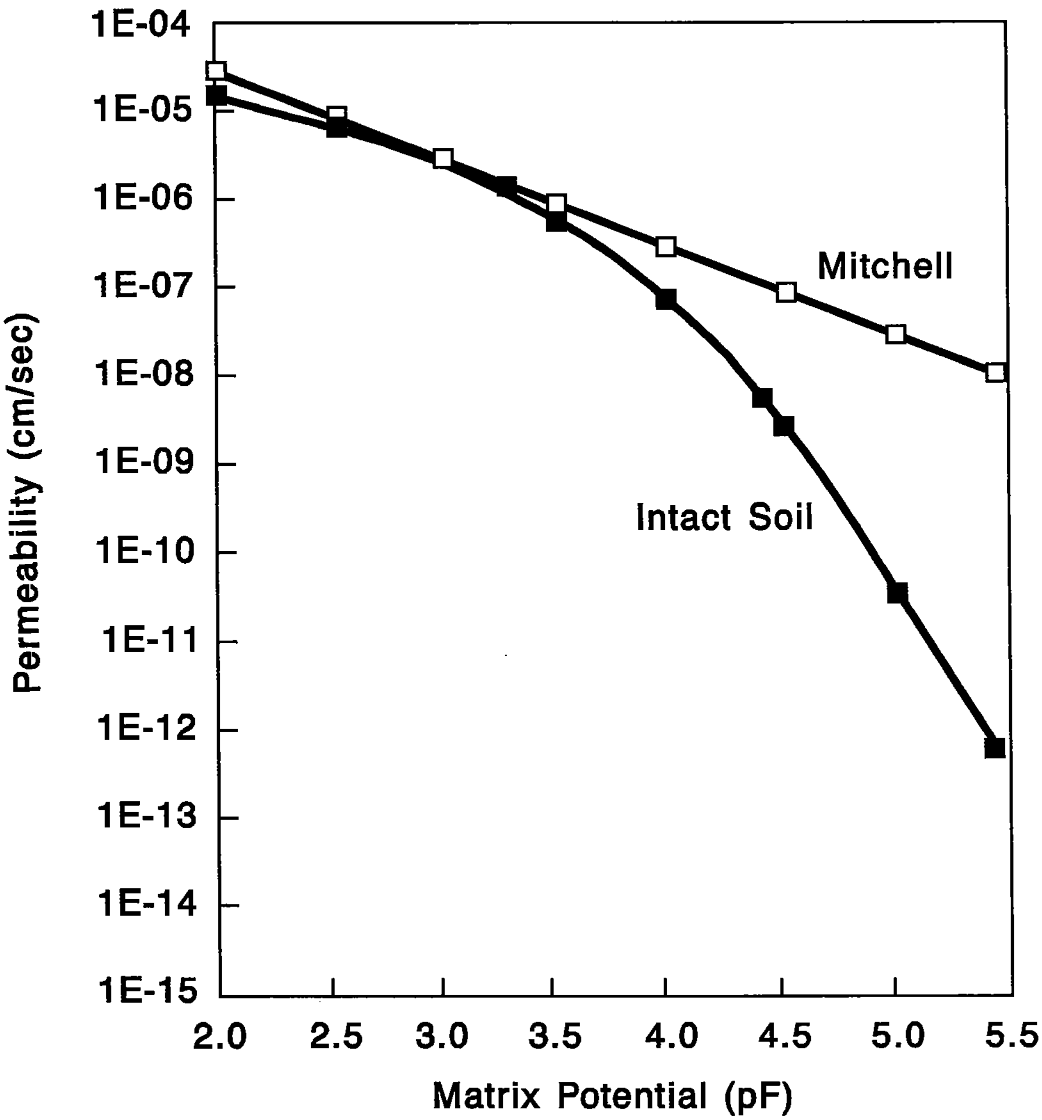

Figure 5. Permeability Relationships for Intact and Cracked Clay Soil.

$$\Delta h = -\Delta Z \left[1 + \frac{v}{k_o} \left(\frac{h}{h_o}\right)\right] \tag{18}$$

where h_o = about -200 cm. in clays.

Mitchell's expression takes into account, to some extent, the increased permeability of the soil mass due to the cracks that become open at high suction levels. This is illustrated in Figure 5 which contrasts the permeability of intact soil with the Mitchell unsaturated permeability formulation. The increased permeability due to cracks begins to develop at approximately a pF of 3.5.

The velocity of water entering or leaving the soil may be estimated from Thornthwaite Moisture Index moisture balance computations (6).

The suction profiles for two transient states can be predicted approximately using:

$$U(Z,t) = U_e + U_o \exp\left(-\sqrt{\frac{n\pi}{\alpha}}Z\right) \cos\left(2\pi nt - \sqrt{\frac{n\pi}{\alpha}}Z\right) \tag{19}$$

where:

U_e = the equilibrium value of suction expressed as pF.
U_o = the amplitude of pF (suction) change at the ground surface.

n = the number of suction cycles per second (1 year = 31.5×10^6 seconds).
α = the soil diffusion coefficient using Mitchell's unsaturated permeability (ranges between 10^{-5} and 10^{-3} cm^2/sec).
t = time in seconds.

Tables of values of U_e and U_o for clay soils with different levels of Mitchell's unsaturated permeability have been found using a trial and error procedure. The dry suction profile has a U_e-value of 4.5 and a U_o-value of 0.0. The wet suction profile has U_e and U_o-values that vary with the soil type and Thornthwaite Moisture Index. Typical values are shown in Table 2.

Table 2. Wet Suction Profile Values

Thornthwaite Moisture Index	Mitchell Unsaturated Permeability cm^2/sec	U_e (PF)	U_o (PF)
-46.5	5×10^{-5}	4.43	0.25
	10^{-3}	4.27	0.09
-11.3	5×10^{-5}	3.84	1.84
	10^{-3}	2.83	0.83
26.8	5×10^{-5}	3.47	1.47
	10^{-3}	2.79	0.79

Values of n are 1 cycle per year for all Thornthwaite Moisture Indexes (TMI) less than -30.0 and 2 cycles per year for all TMI greater than -30.0.

Equation 16 shows that the equilibrium suction profile corresponds to a vertical velocity of zero and that it has a slope of 1 cm more negative suction for every 1 cm higher in elevation.

Use of Mitchell's unsaturated permeability formulation in a finite element simulation of suction changes on each side of a vertical moisture barrier produced reasonable predictions of the measured values except in the vicinity of cracks that were open to the air. The pattern of measured versus predicted suctions are as shown in Figure 6. Actual data for a monitoring site near Seguin, Texas are shown in Figure 7. A crack that is open to the atmosphere gets much wetter and drier with fluctuations of the weather than does the cracked soil in which the cracks are not open to the air. The close correspondence between the predicted and measured values of suction in all other instances lends support to the practical use of Mitchell's unsaturated permeability.

The values of the equilibrium suction U_e that may be used to estimate suction profiles vary with the Mitchell unsaturated permeability, $p(cm^2/sec)$, and the Thornthwaite Moisture Index. Typical values are in Table 3.

Heave (or shrinkage) from a present condition in the soil uses as the initial value of suction, h_i, the value measured from samples taken. The suction can be measured by any of a number of acceptable means. The filter paper method is the simplest.

If the suction profile is not controlled by the evapotranspiration at the soil surface but by a high water table, this fact can be discovered by measuring the suction on a Shelby tube sample. If the magnitude of the suction is lower than that expected when the suction profile is governed by

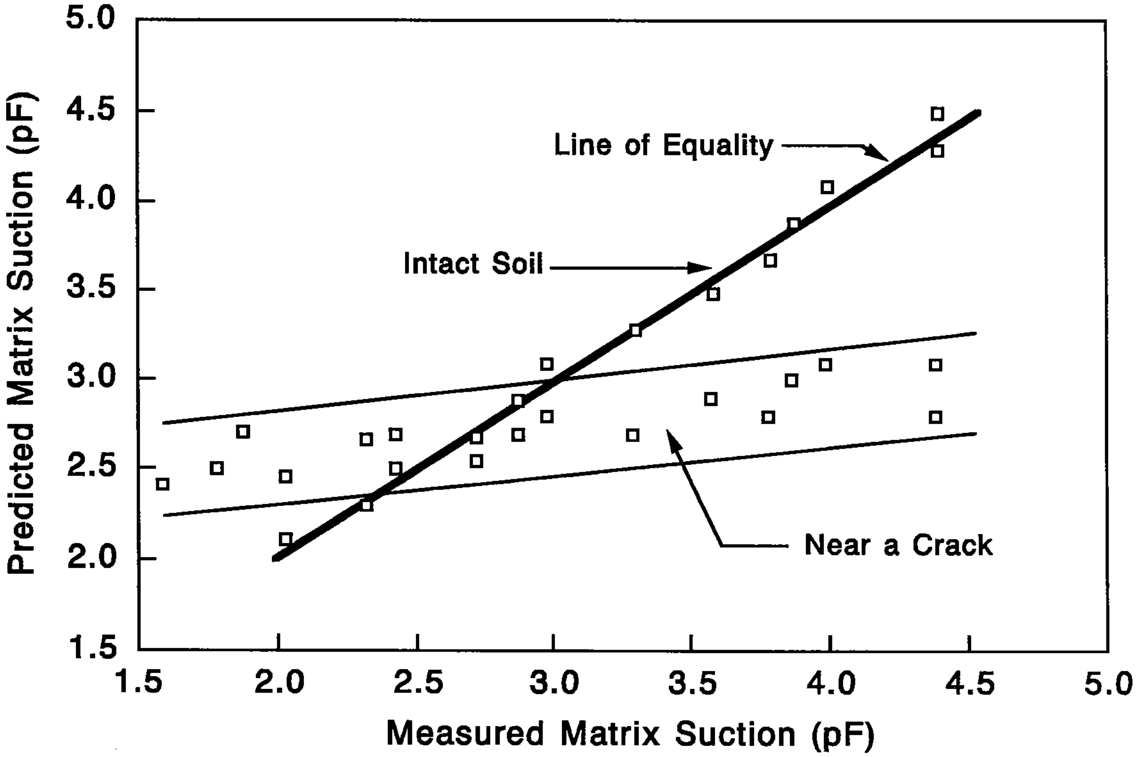

Figure 6. A Typical Pattern of Measured Soil Suction vs. Predicted Soil Suction.

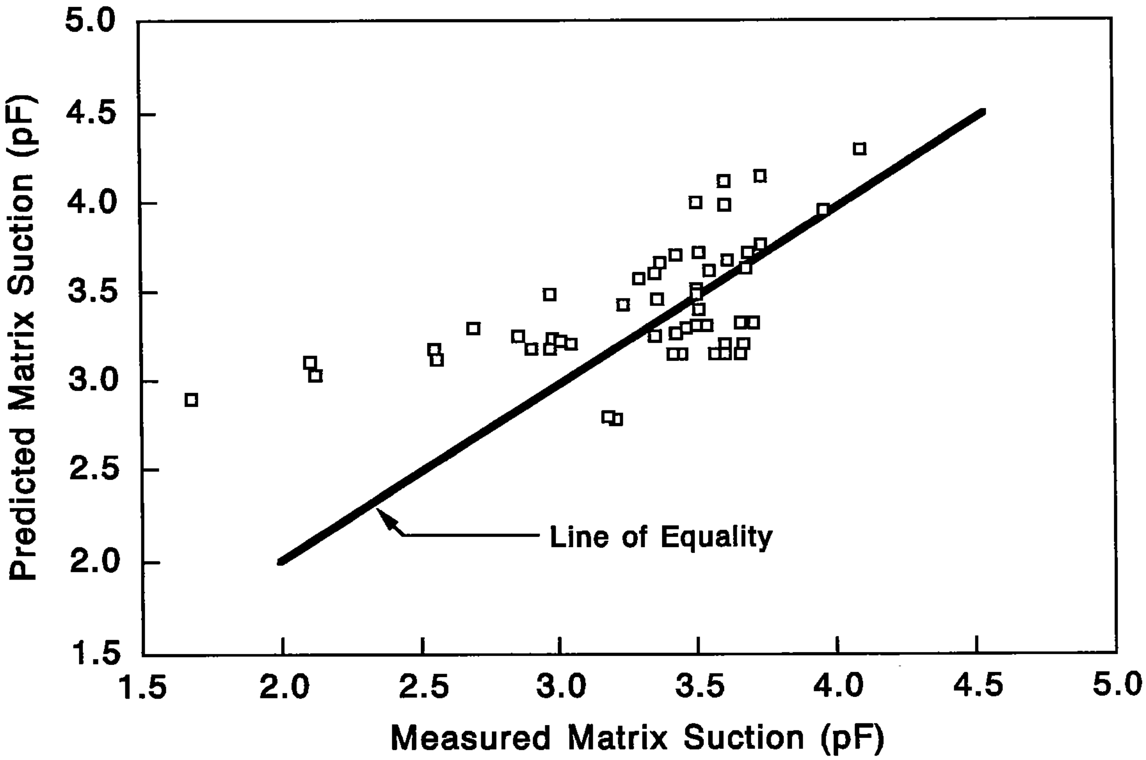

Figure 7. Measured Soil Suction vs. Predicted Soil Suction at Seguin.

Table 3. Equilibrium Suction Values, U_e

	Mitchell Unsaturated Permeability, cm^2/sec		
TMI	~~5 x 10^{-5}~~ 1 x 10^{-3}	2.5 x 10^{-4}	~~1.0 x 10^{-3}~~ 5 x 10^{-5}
-46.5	4.27	4.32	4.43
-30.0	3.80	3.95	4.29
-21.3	3.42	3.64	4.20
-11.3	2.83	3.10	3.84
26.8	2.79	3.05	3.47

surface evapotranspiration, then it is controlled by a high water table. This will usually be within about 10 m (30 feet) of the surface.

If the suction is higher than expected then there is osmotic suction present. Osmotic suction levels may be measured with vacuum desiccators.

Estimates of Unsaturated Soil Properties

The fundamental definition of p is :

$$p = \frac{k_o|h_o|}{0.4343} \tag{20}$$

where:

$|h_o|$ = 200 cm for clays.

The units of k_o, the saturated permeability, (cm/sec), and $|h_o|$, the suction at which the soil desaturates (cm) produce units of (cm^2/sec) for the Mitchell unsaturated permeability.

The Mitchell unsaturated permeability, p, is estimated by:

$$p = \frac{\alpha\gamma_d}{|S|\gamma_w} \left(\frac{cm^2}{\sec}\right)$$

where:

γ_w = the unit weight of water.

α = the Mitchell diffusion coefficient, cm^2/sec., which is used in Equation (19).

$|S|$ = the absolute value of the slope of the pF-vs-gravimetric water content, w line.

γ_d = the dry unit weight of the soil.

The value of α can be estimated from:

$$\alpha = 0.0029 - 0.000162(S) - 0.0122(\gamma_h) \tag{21}$$

The value of S is negative and can be estimated from:

$$S = -20.29 + 0.1555\ (LL\%) - 0.117\ (PI\%) + 0.0684\ (\% - \#200) \tag{22}$$

where:

LL = the liquid limit in percent.
PI = the plasticity index in percent.
-#200 = the percent of the soil passing the #200 sieve.

The slope of the suction-versus-volumetric water content curve is given by:

$$\left(\frac{\partial h}{\partial \theta}\right) = \frac{1}{0.4343}\ \frac{S\ \gamma_w}{\gamma_d}\ h \tag{23}$$

Because both S and h are negative, the slope is inherently positive as illustrated in Figure 1. The correction term in the relation between γ_h and γ_σ given in Equation (12) is found by:

$$\frac{h}{\theta\ \left(\frac{\partial h}{\partial \theta}\right)} = \frac{0.4343}{S\ w} \tag{24}$$

where:

w = the gravimetric water content.

Because S is negative, so is the correction term.

An approximate suction (pF)-versus-volumetric water content curve can be constructed with the empirical relationships given above and the saturated volumetric water contents given in Table 4. The construction is illustrated in Figure 8. First, point A is located at the intersection of the field capacity volumetric water content (= 0.88 Θ_{sat}) and a pF of 2.0. Second, a line with a slope of $S\gamma_w/\gamma_d$ is drawn from point A to its intersection with the vertical axis. Third, point C is located at a volumetric water content of 0.10 Θ_{sat} and the tensile strength of water (pF = 5.3 or 200 atmospheres). Fourth, point D is located at zero water content and a pF of 7.0, corresponding to oven dry. Fifth, a straight line is drawn between points C and D to its intersection with the first line.

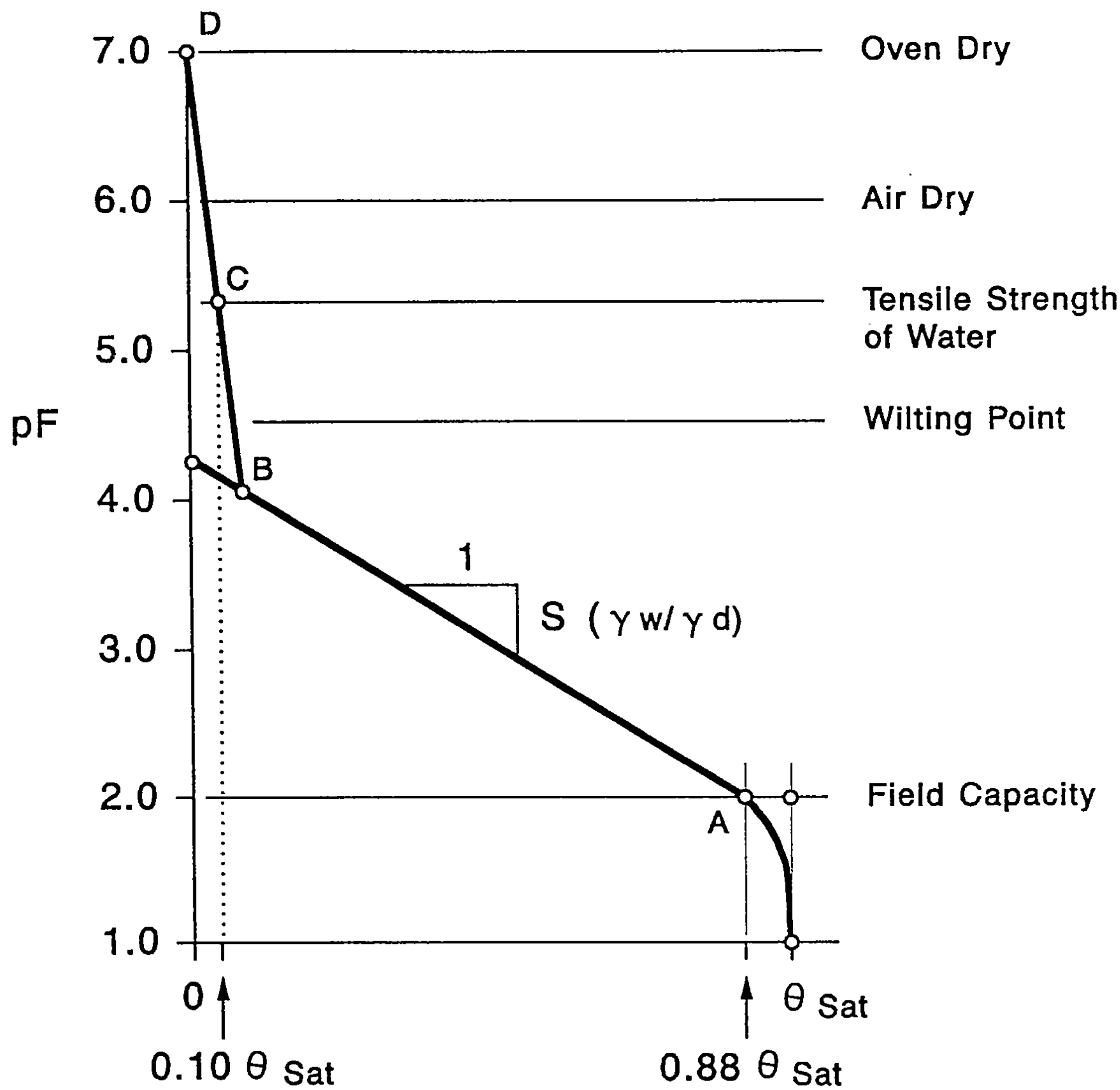

Figure 8. Approximate Construction of a Suction (pF)-versus-Volumetric Water Content Curve.

This construction makes it possible to estimate water contents once the computed suction profiles are known. This allows measured water contents to be compared with the predicted values.

Table 4. Ranges of Saturated Volumetric Water Content (7) by Unified Soil Class.

Unified Class	Ranges of Θ_{sat} *
GW	0.31 - 0.42
GP	0.20
GM	0.21 - 0.38
GM-GC	0.30
SW	0.28 - 0.40
SP	0.37 - 0.45
SM	0.28 - 0.68
SW-SP	0.30
SP-SM	0.37
SM-SC	0.40
ML	0.38 - 0.68
CL	0.29 - 0.54
ML-CL	0.39 - 0.41
ML-OL	0.47 - 0.63
CH	0.50

* Θ_{sat} = n (porosity)

Differential Movement

Differential movement which affects the performance of a ground-supported slab may take numerous shapes but the most important shapes for design purposes are those which generate the maximum values of moment, shear, and differential deflection of the slab. The two shapes that can be generated by water entering or leaving the soil beneath a slab are the edge lift and center lift conditions.

If a slab is cast on dry ground, the entire slab may move upward until an equilibrium suction profile is established, after which the edges will move up and down in response to the seasonal changes. If the same slab were cast on wet ground, the entire slab will move downward until an equilibrium profile is established. Once more, the edges will move up and down in response to the seasonal moisture changes. Thus, a major concern for design is whether these seasonal movements will cause moment, shear, and differential deflections that exceed the capacity of the designed slab cross-section. The distance within which these changes take place has been named the "edge moisture variation distance". An empirical relation between this distance and the Thornthwaite Moisture Index has been used in the Post-Tensioning Institute Manual for the Design and Construction of Post-Tensioned Slabs-on-Ground. Because it is known that the "edge moisture variation distance" depends upon the permeability of the soil as well, it is important to determine that relation.

The calibrated finite element program with coupled transient moisture flow and elasticity that had been used in the study of vertical moisture barriers provided an ideal means to study the edge moisture variation distance. A full range of α and p values were used to determine the relation of the moisture distance and the Thornthwaite Moisture Index and unsaturated soil properties. Both edge lift and center lift conditions were explored using several hundred runs with the program. Center lift conditions were simulated by a one year dry spell following a wet suction profile condition. Edge lift conditions were simulated by a one year wet spell following a dry suction profile condition. The edge moisture variation distance was considered to be that distance between the edge of the foundation and the point beneath the covered area where the suction changed no more than 0.2 pF during the entire period of simulation.

The dry and wet conditions used annual suction variation patterns that were appropriate for each of nine different climatic zones ranging from a Thornthwaite Moisture Index of -46.5 to + 26.8, spanning the range found in Texas. The resulting edge moisture variation distances are shown in Figures 9 and 10. Seven different soils were used in the study. No distance less than 2.0 feet (0.6m) was considered to be adequate for design purposes.

In Figure 9 for the center lift condition, Soils No. 1, 2, and 3 are highly pervious and Soils No. 5, 6, and 7 are practically impervious. Only soils between No. 3 and No. 4 have edge moisture variation distances in the range presently used in the PTI manual.

In Figure 10 for the edge lift condition, Soils No. 5, 6, and 7 are practically impervious while Soils No. 2, 3, and 4 have edge moisture variation distances in the range presently used in the PTI manual. Soil No. 1 is more pervious and outside the range presently used in the PTI manual.

The edge moisture variation distances of soils with unsaturated permeabilities different than these seven soil types can be found by interpolation on these two figures. The edge moisture variation distance in center lift mode, in which the soil around the edge of the slab is drier than the soil supporting it, is more sensitive to changes in the unsaturated permeability than with the edge lift mode.

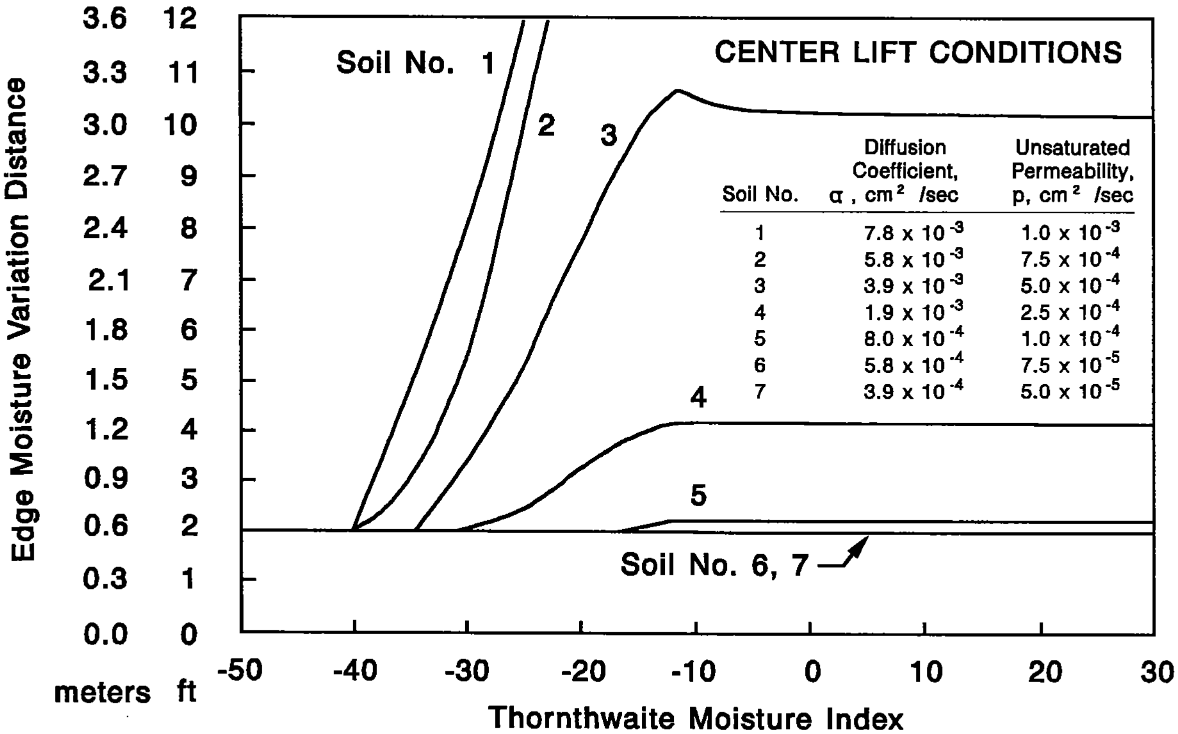

Figure 9. Edge Moisture Variation Distances for the Center Lift Moisture Condition.

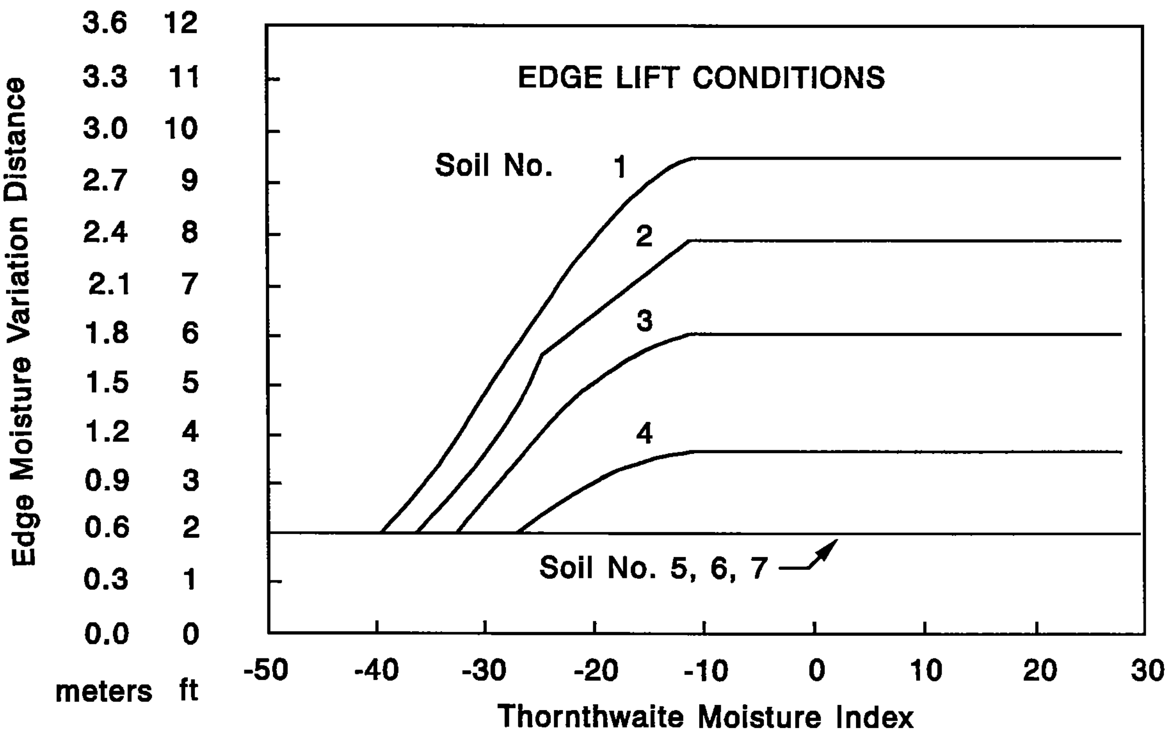

Figure 10. Edge Moisture Variation Distances for the Edge Lift Moisture Condition.

Conclusions

Simple laboratory tests can be used to determine important properties of expansive soils including the compression indices due to matrix suction and mean principal stress, the slope of the suction-versus-water content curve, and the unsaturated permeability and diffusivity. The tests are the Atterberg limits, hydrometer test, water content, dry density, and sieve analysis.

Prediction of differential movement depends strongly upon the edge moisture variation distance which, in turn, depends upon the Thornthwaite Moisture Index and the unsaturated permeability of the soil. Tree roots penetrating beneath the edge of a building will have a zone of moisture influence beyond the edge of the root zone equal to the edge moisture variation distances shown in Figures 9 and 10. This explains the unusually destructive effect that trees have when they grow near enough to the edge of a foundation to have their roots intrude beneath the edge. It also explains the effectiveness of vertical root and moisture barriers around the perimeter of the foundation in reducing the moisture variation distance and the differential movement. A vertical barrier carried to a depth of a 4 feet (1.2 meters) excludes many roots, makes the edge moisture variation distance predictable, and reduces the differential movement that a foundation must be designed to withstand.

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RANGES OF SUCTION

CONVERSION OF UNITS

SIMPLIFIED SAMPLE CALCULATIONS OF

- **VERTICAL FLOW**
- **HORIZONTAL FLOW**
- **HEAVE**
- **SHRINKAGE**

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Notes originally prepared for:

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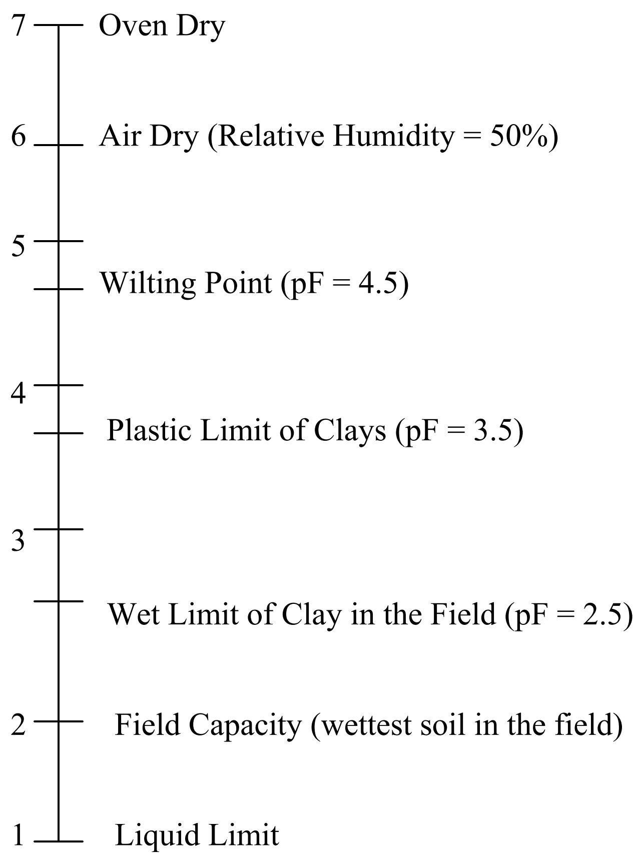
At the University of Houston
At the Invitation of
Dr. Michael O'Neil

RANGES OF SUCTION

The following scale is presented as a guide in determining reasonable levels of suction for estimating differential and total heave and shrinkage.



CONVERSION OF UNITS

Geotechnical laboratories may report suction measurements in a variety of units, especially with metric conversion going on in the United States. The following is to make it easy to convert from any one system of units into the pF – scale that is used in the VOLFLO program.

CONVERSIONS TO pF

$$pF = \log_{10}(kPa) + 1.009$$

$$pF = \log_{10}(Tsf) + 2.990$$

$$pF = \log_{10}(psi) + 1.847$$

$$pF = \log_{10}(psf) + 0.311$$

CONVERSIONS TO *cm* OF SUCTION

$$cm = \frac{lb}{in^2} \times 70.37 \left(\frac{cm}{psi}\right)$$

$$cm = \frac{T}{ft^2} \times 977.36 \left(\frac{cm}{Tsf}\right)$$

$$cm = kPa \times 10.21 \left(\frac{cm}{kPa}\right)$$

$$cm = psf \times 0.4887 \left(\frac{cm}{psf}\right)$$

Typical Suction Levels

Air Dry	- 6.0 pF
Drying in Grass and Tree Root Zones	- 4.5 pF
Plastic Limit in Fat Clays	- 3.5 pF
Natural Water Content in Clays	- 3.2 – 3.7 pF
Clay Wet Limit	- 2.5 pF
Liquid Limit	- 1.0 pF

pF = $\log_{10}$ (suction in cm)

Flow of Water in Clays

Horizontal Flow

$$v = k\frac{\Delta h}{\Delta x} \qquad \Delta h = \Delta x\left(\frac{v}{k}\right)$$

+ velocity = out of the soil

- velocity = into the soil

Δh = change in suction, in cm

Δx = change of horizontal location, in cm

Vertical Flow

$$v = -k\left(\frac{\Delta h}{\Delta z} + 1\right) \qquad \Delta h = \Delta z\left(\frac{v}{k} + 1\right)$$

Same sign convention on velocity flow direction

Δh = change in suction, in cm

Δz = upward change in elevation, in cm

Note: when $v = 0$, $\Delta h/\Delta z = -1$.

Permeability

$$k = 2 \times 10^{-6} \frac{cm}{sec} \left[\frac{-200cm}{-h(cm)} \right]$$

h = value of suction, cm

Volume Change in Clays

Percent Volume Change

1. Swelling — matrix suction swelling term; overburden + surcharge correction; osmotic suction swelling term

$$\left(\frac{\Delta v}{v}\right) = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_h \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_o \log_{10}\left(\frac{\pi_f}{\pi_i}\right)$$

h_f = final matrix suction, cm.

h_i = initial matrix suction, cm.

σ_i = overburden correction constant

$$40^{cm} \times \gamma_t \left({}^{gm}\!/\!_{cm^3} \right)$$

σ_f = mean pressure in $\left(g/cm^2\right)$ at depth z

below 40 cm $\left[= \gamma_t \left(\frac{1 + 2\,K_o}{3} \right) \right]$

π_i, π_f = initial and final osmotic suction, cm.

$\left(\frac{\Delta v}{v}\right)$ = volume change percent (in decimal form)

γ_h = volume change coefficient

Note: Overburden and surcharge correction term is NOT applied above 40 cm or Below where it exceeds the swelling term or when it is the same sign.

2. Shrinking

$$\left(\frac{\Delta v}{v}\right) = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) + \gamma_h \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_o \log_{10}\left(\frac{\pi_f}{\pi_i}\right)$$

shrinking term; overburden and surcharge correction term; osmotic suction shrinkage term

3. Lateral Earth Pressure Coefficient

K_o = 0.0 when there are many cracks in the soil
K_o = 1/3 when the soil is drying out
K_o = 2/3 when the soil is wetting up
K_o = 1.0 when the cracks are closed tightly

4. Vertical Volume Change at Depth, z

$$\left(\frac{\Delta H}{H}\right) = f\left(\frac{\Delta v}{v}\right)$$

f = 0.5 when soil is drying out
f = 0.8 when soil is wetting up

5. Total Heave or Shrinkage

$$y_m = \sum_{i=1}^{n} \underbrace{\left(\frac{\Delta H}{H}\right)}_{\text{vertical volume change}} \underbrace{(\Delta z)}_{\text{vertical increment, cm}}$$

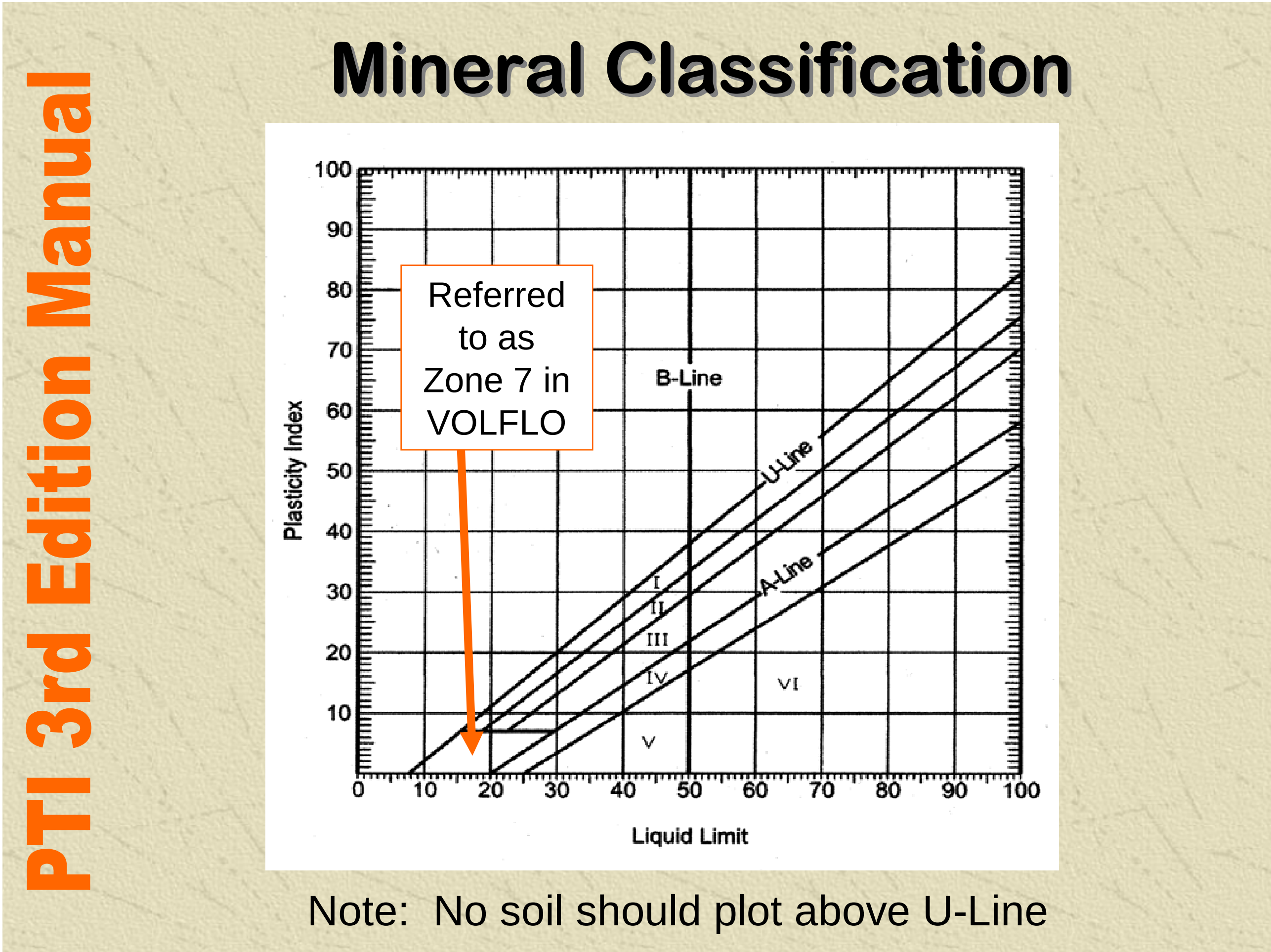
Volume Change Coefficient, γ_h

Need to know: 1. PI%, LL%
2. % Fine Clay

1. PI(%) = Liquid Limit (LL) - Plastic Limit (PL)

2. $$\% \text{ Fine Clay} = \frac{\% \text{ Passing } (2\mu) \text{ size}}{\% \text{ Passing } (\#200) \text{ size}} \times 100$$

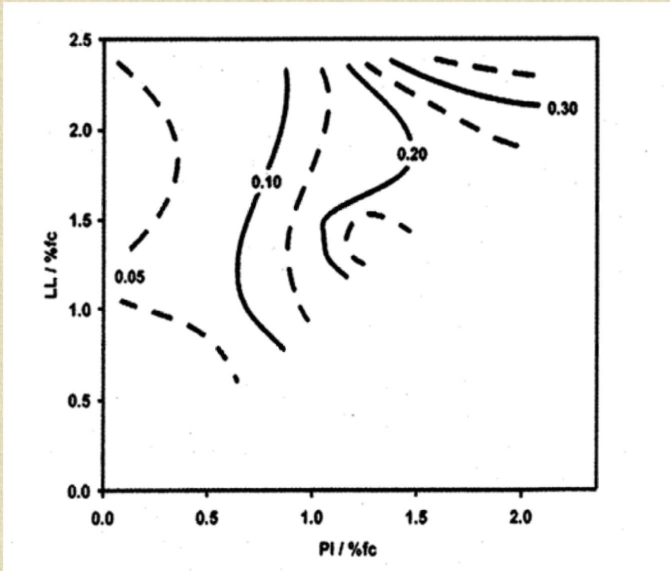
3. $$\text{Activity Ratio, AC} = \frac{\text{PI } (\%)}{\% \text{ Fine Clay}}$$


PTI 3rd Edition Manual
Mineral Classification
Referred to as Zone 7 in VOLFLO
B-Line
U-Line
A-Line
I
II
III
IV
V
VI
Plasticity Index
Liquid Limit
0
10
20
30
40
50
60
70
80
90
100
Note: No soil should plot above U-Line

Suction Compression Index – γ_o

PTI 3rd Edition Manual

Zone II Chart



Note: There is no chart for "Zone 7". PTI recommends γ_o = 0. 01 for this zone.

4. Liquid Limit Activity $= \dfrac{LL(\%)}{\%\ \text{Fine Clay}}$

5. Volume Change Guide Number (From Chart), γ_o

6. Volume Change Coefficient, γ_h

$$= \%\ \text{Fine Clay (decimal)} \times \gamma_o$$

<u>Example Problem</u> - Equilibrium Suction Profile

No vertical flow: v = o

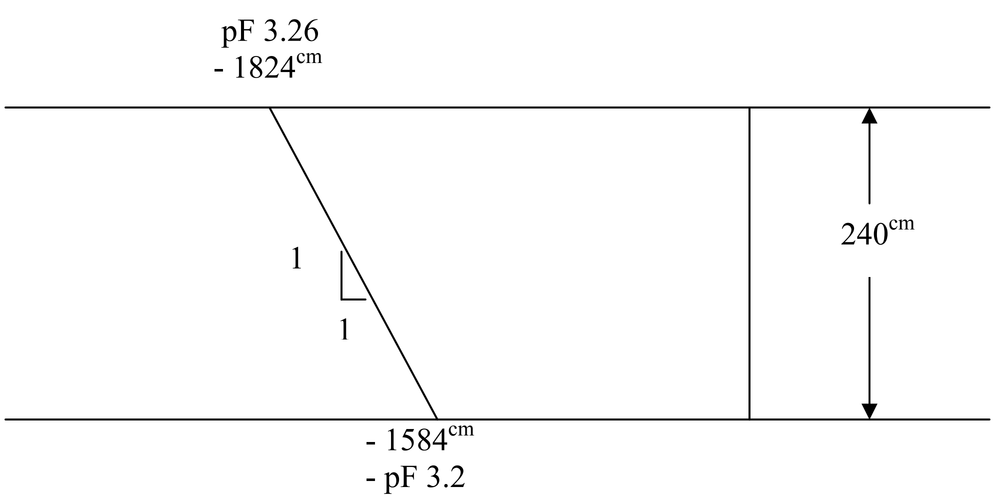
$$\frac{\Delta h}{\Delta z} = -1$$

Suction at 8 ft: pF 3.2 = - 1584 cm

$$\text{Suction at Surface} - 1584 \text{ cm} + \left(\frac{\Delta h}{\Delta z}\right) \times 8' \times 30\ \frac{\text{cm}}{\text{ft}}$$

$$= -1584^{cm} + \left(-1\frac{cm}{cm}\right) \times 240^{cm}$$

$$= \underline{-1824^{cm}}$$



Example Problem - Volume Change

$$\left.\begin{array}{l} LL = 76\% \\ PL = 22\% \\ PI = 54 \end{array}\right\} = \text{Soil Zone II}$$

% Fine Clay = 60%

$$\left.\begin{array}{l} AC = 54/60 = 0.90 \\ LLAC = 76/60 = 1.27 \end{array}\right\} \text{Zone II}$$

$\gamma_o = 0.15$ From Chart No. II

$\gamma_h = 0.15 \text{ x } 0.60 = 0.09$

Assume $\gamma_o = \quad = 0.09$

Dry pF = 4.20 (-15984_{cm})

Wet pF = 2.60 $\left(-398_{cm}\right)$

$$\left(\frac{\Delta v}{v}\right) = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)$$

$$\frac{\Delta H}{H} = f\left(\frac{\Delta v}{v}\right)$$

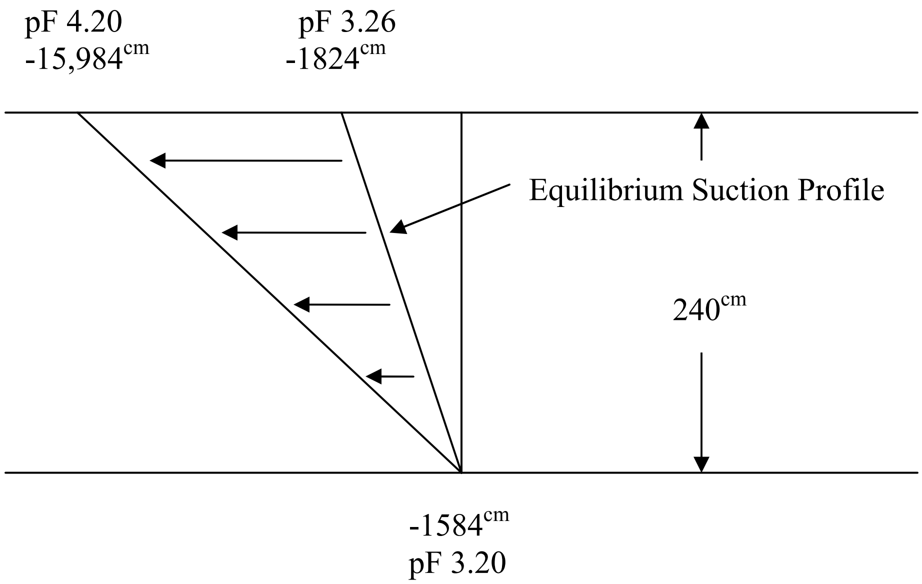
f = 0.5 (Drying) $\sigma_i = 40^{cm} \text{ x } \gamma_t$

f = 0.8 (wetting) $\sigma_f = 2 \text{ x } \gamma_t \text{ x } \left(\frac{1 + 2Ko}{3}\right)$

Ko = 1 (Assume)

$$\left(\frac{\sigma_f}{\sigma_i}\right) = \left(\frac{z^{cm}}{40}\right)$$

Drying

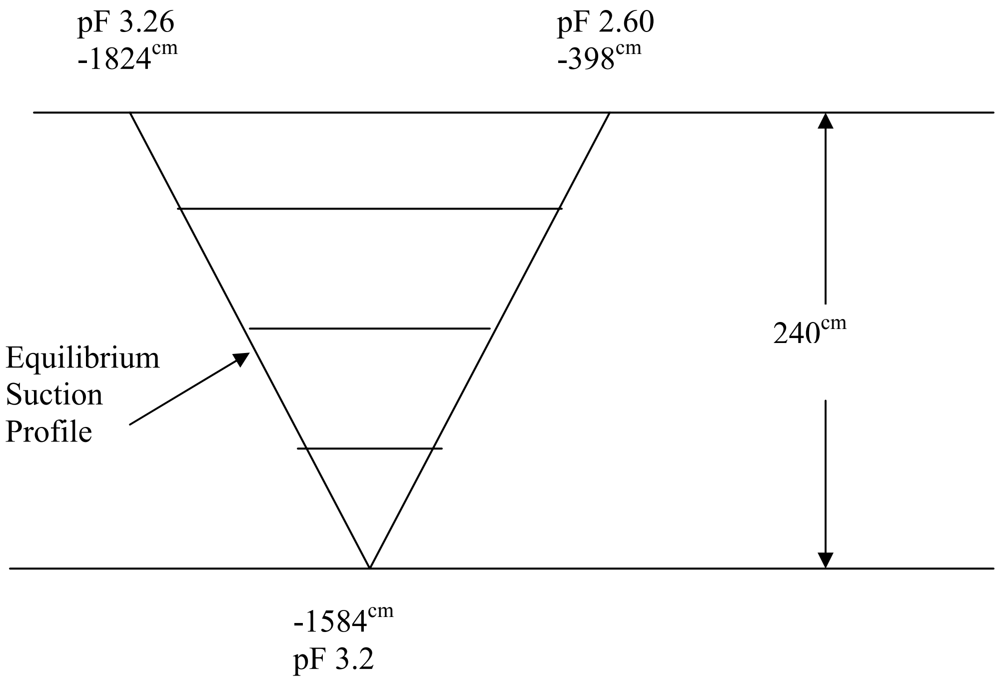


Drying

$\gamma_h = \gamma_\sigma = 0.090$ $f = 0.5$

	Depth, cm	h_i, cm	h_f, cm	$-\gamma_h \log_{10}\left(\frac{hf}{hi}\right)$	$+\gamma\sigma \log_{10}\left(\frac{\sigma f}{\sigma_i}\right)$	$\left(\frac{\Delta v}{v}\right)$	$\left(\frac{\Delta H}{H}\right)$
0’	0	-1824	-15984	-0.0848		-0.0848	-0.0424
	20	-1804	-14784	-0.0822		-0.0822	-0.0411
	40	-1784	-13584	-0.0793		-0.0793	-0.0397
2’	60	-1764	-12384	-0.0762	0.0158	-0.0604	-0.0302
	80	-1744	-11184	-0.0726	0.0271	-0.0455	-0.0228
	100	-1724	-9984	-0.0686	0.0358	-0.0328	-0.0164
4’	120	-1704	-8784	-0.0641	0.0429	-0.0212	-0.0106
	140	-1684	-7584	-0.0588	0.0490	-0.0098	-0.0049
	160	-1664	-6384	-0.0526	0.0542		
6’	180	-1644	-5184	-0.0449	0.0588		
	200	-1624	-3984	-0.0351	0.0629		
	220	-1604	-2784	-0.0216	0.0666		
8’	240	-1584	-1584	-0.0000	0.0700		
					Assume		
					Ko = 1		

Depth, cm	$\left(\frac{\Delta H}{H}\right)$	H, cm	ΔH, cm	y_m	
0	-0.0424	10	-0.424	3.738 cm	1.47in
20	-0.0411	20	-0.822		
40	-0.0397	20	-0.794		
60	-0.0302	20	-0.604		
80	-0.0228	20	-0.456		
100	-0.0164	20	-0.328		
120	-0.0106	20	-0.212		
140	-0.0049	20	-0.098		
160					
180					
200					
220					
240					



Wetting

$\gamma_h = \gamma_\sigma = 0.090$ $f = 0.8$

	Depth, cm	h_i, cm	h_f, cm	$-\gamma_h \log_{10}\left(\frac{hf}{hi}\right)$	$-\gamma\sigma \log_{10}\left(\frac{\sigma f}{\sigma_i}\right)$	$\left(\frac{\Delta v}{v}\right)$	$\left(\frac{\Delta H}{H}\right)$
0’	0	-1824	-398	+0.0595		+0.0595	0.0476
	20	-1804	-497	+0.0504		+0.0504	0.0403
	40	-1784	-596	+0.0429		+0.0429	0.0343
2’	60	-1764	-694	+0.0365	-0.0158	+0.0207	0.0166
	80	-1744	-793	+0.0308	-0.0271	+0.0037	0.0030
	100	-1724	-892	+0.0258	-0.0358		
4’	120	-1704	-991	+0.0212	-0.0429		
	140	-1684	-1090	+0.0170	-0.0490		
	160	-1664	-1189	+0.0131	-0.0542		
6’	180	-1644	-1287	+0.0096	-0.0588		
	200	-1624	-1386	+0.0062	-0.0629		
	220	-1604	-1485	+0.0030	-0.0666		
8’	240	-1584	-1584	0.0000	-0.0700		
					Assume		
					Ko = 1		

Wetting (Continued)

Depth, cm	$\left(\frac{\Delta H}{H}\right)$	H, cm	ΔH, cm	y_m, cm
0	+0.0476	10	0.476	2.360cm = 0.93in
20	+0.0403	20	0.806	
40	+0.0343	20	0.686	
60	+0.0166	20	0.332	
80	+0.0030	20	0.060	
100				
120				
140				
160				
180				
200				
220				
240				

Wetting With Open Cracks

$$\gamma_h = \gamma\sigma = 0.090 \qquad \frac{\text{wetting}}{f = 0.8}$$

	Depth, cm	$-\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right)$	$-\gamma\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right)$	$\left(\frac{\Delta v}{v}\right)$	$\left(\frac{\Delta H}{H}\right)$	H, cm	ΔH	y_m, cm
0’	0	+0.0595		+0.0595	0.0476	10	0.476	4.002cm = 1.58in
	20	+0.0504		+0.0504	0.0403	20	0.806	
	40	+0.0429		+0.0429	0.0343	20	0.686	
2’	60	+0.0365		+0.0365	0.0292	20	0.584	
	80	+0.0308		+0.0308	0.0246	20	0.492	
	100	+0.0258		+0.0258	0.0206	20	0.412	
4’	120	+0.0212		+0.0212	0.0170	20	0.340	
	140	+0.0170	-0.0060	+0.0110	0.0088	20	0.176	
	160	+0.0131	-0.0112	+0.0019	0.0015	20	0.030	
6’	180	+0.0096	-0.0158					
	200	+0.0062	-0.0200					
	220	+0.0030	-0.0237					
8’	240	+0.0000	-0.0271					
			Assume					
			$K_o = 0$					

$$\sigma_f = \frac{Z}{3} \times \gamma_t$$

$$\sigma_i = 40 \times \gamma_t$$

$$\frac{\sigma_f}{\sigma_i} = \frac{Z}{120}$$

Shallow Slides in Compacted High Plasticity Clay Slopes

C.P. Aubeny, Member ASCE[1] and R.L. Lytton, Fellow ASCE[2]

Abstract: Shallow slide failures in embankments constructed of high plasticity clays create costly maintenance problems on highway projects and can threaten the integrity of water-retaining earth structures. This paper investigates the mechanisms of stability degradation that lead to these slope failures. The failure mechanism involves moisture infiltration into the slope surface that leads to decreases in suction and soil shear strength. Both the degree and time rate of strength loss are investigated based on stability and moisture diffusion analyses, respectively. Stability analyses indicate that the failures are associated with destabilizing hydraulic gradients in the pore water, and the suction level at the surface of the slope declines to a limiting suction of about u =2pF when exposed to moisture. Moisture diffusion analyses indicate that the time rate of strength degradation is controlled by the depth and spacing of desiccation cracks that form in the soil mass, and the moisture diffusion properties of the soil. The stability and moisture diffusion models described above were evaluated in light of 34 documented shallow slides in Texas high plasticity clays.

CE Database keywords: Slopes; Shear strength; Moisture diffusion; Suction; Stability

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INTRODUCTION

High plasticity clays occur in many areas of Texas and elsewhere and often offer the most economical material alternative for construction of embankments. While the overall stability of properly designed and constructed plastic clay embankments is generally adequate, experience has shown that the outer layers of these embankments can experience dramatic strength loss over time. The consequent sloughing and shallow slide failures represent a costly maintenance problem in highway embankments. In water-retaining earth structures such as levees, the consequences of failure can go beyond costly maintenance and can threaten the integrity of flood control systems. Effective design and maintenance programs for embankments constructed of plastic clays require an understanding of the mechanisms of strength loss, in particular the site factors which influence the degree and rate of strength loss, and reasonable estimates of the lower limit to which strength can degrade.

This study adopts an effective stress approach to the problem, in which a major portion of the soil strength is derived from negative pore pressures (matric suction). Strength loss occurs as surface moisture during wet periods enters surface cracks and infiltrates into the soil mass thereby weakening the soils. When the moisture infiltration and concomitant reduction in strength advances to a critical depth, the driving stresses in the slope exceed the available resisting stresses and failure occurs. Since the laws of flow through a porous medium govern the changes in suction over time, the prediction of strength loss over time is analyzed through a relatively straightforward solution of the diffusion equation.

To characterize the strength and time rate aspects of shallow slide failures, this paper presents two models, a stability model and a moisture diffusion model, respectively. These models are applied to case studies of slope failures in high plasticity clays documented in a previous study by Kayyal and Wright (1991). All of the slope failures considered in the case studies occurred in slopes that were flatter than the estimated normally consolidated friction angle of the soils. While degradation of the friction angle could conceivably have occurred, this paper postulates a plausible distribution of pore water pressures in the soil mass that can explain the failures without recourse to any such assumed degradation.

Stability Analysis

Given that the slide masses under consideration have small vertical dimensions relative to their lateral extent, the analysis can proceed within the framework of a classical infinite slope analysis (e.g., Lambe and Whitman, 1969). The key considerations in the analysis with regard to the pore water pressure distribution and soil strength are discussed in the following paragraphs.

Formulation

Pore-water pressure.

The analysis assumes the following:

- Due to moisture infiltration into the slope, a condition of full saturation is approached. The pore water pressures in this saturated zone will in general be negative (suction) on the surface of the slope and increase with depth due to hydrostatic effects. At sufficient depths the pore water pressures may become positive. In this case a 'phreatic surface' or line of zero pore water pressure will exist, but this should not be construed as a regional water table as it is associated with localized wetting of the surface of the slope.

- Since all points on the surface of the slope are exposed to the same atmospheric conditions, a uniform pore water pressure (suction) on the surface of the slope, u_{w0}, is a reasonable first approximation (Fig. 1). The magnitude of this suction is unknown, but will be deduced from back-analysis of slope failures that will be presented subsequently.
- Constant pressure head on the surface of the slope implies a variable total head; hence, the water is flowing. Although various conditions of evaporation and moisture infiltration are possible, a neutral case of no moisture entering or exiting the slope will be initially considered. In this case, the water is flowing parallel to the slope. The gradient of total head in a direction parallel to the slope is therefore easily seen as the cosine of the slope angle measured from horizontal, cos β (Fig. 1). It is noted that the osmotic suction π also can contribute to the total head, but this will not influence the gradient of total head h_t for conditions of constant π.

A final consideration in characterizing the pore water pressures in a potential slide mass is shear-induced pore water pressure. In the case of an existing slope that is subjected to gradual moistening and softening of the soil mass, the shear stress τ on any soil element is of course applied long before a failure occurs. However, the development of shear strains capable of generating significant shear-induced pore water pressures is not necessarily a long-term, drained process. In fact, much of the straining, together with the associated generation of shear-induced pore water pressures, may occur a relatively short time before the failure of the slide mass. Hence, it is not unreasonable to consider the possibility of undrained or partially drained conditions of shear during the stage of an impending failure. Since the soil is being subjected to conditions of simple shear, there will be no changes in mean stress during the shearing process and all generated excess pore pressures will be associated with pure shear. In this case, the most appropriate expression describing the generation of excess pore pressures is the Henkel parameter a relating shear induced pore pressures Δp_s to octahedral shear stress $\Delta\tau_{oct}$ (Holtz and Kovacs, 1981) $\Delta u_s = a\ \Delta\tau_{oct}$. An applied simple shear stress, $\Delta\tau$, will therefore induce a shear-induced pore pressure $\Delta u_s = a\sqrt{2/3}\,\Delta\tau = a\ \Delta\tau_{oct}$.

Considering all of the conditions described above - constant suction on the slope surface, flow parallel to the slope, and the generation of shear-induced pore-water pressures – results in the following expression for the pore-water pressure at the base of any potential slide mass of depth H:

$$u_w = u_{w0} + \gamma H \cos^2\beta + \sqrt{2/3}\,a\gamma H \sin\beta\cos\beta \qquad (1)$$

where u_w = pore water pressure at a vertical depth H below slope surface

u_{w0} = pore-water pressure on surface of slope

γ = total unit weight of the soil

β = slope angle measured from horizontal

H = vertical depth

a = Henkel shear-induced pore pressure coefficient

Soil strength.

For the compacted soils considered in this study, any natural cementation has been destroyed during the compaction process and no effective cohesion is assumed; i.e., c'=0. Hence, the shearing resistance within the slide mass will be considered to be solely due to mechanical stress and matric suction. Such resistance can be characterized by a generalized Mohr-Coulomb relationship for an unsaturated soil (Fredlund and Rahardjo, 1993). The foregoing analysis will utilize the premise proposed by Lytton (1995) and Lamborn (1986) that a single internal friction angle can characterize the shearing resistances associated with net mechanical stress and matric suction provided that one accounts for the reduced surface area on which the water phase acts in an unsaturated soil. In this case the generalized Mohr-Coulomb strength equation becomes:

$$\tau_f = (\sigma_n - u_a) \tan \phi' + f\theta \ (u_a - u_w) \tan \phi' \qquad (2a)$$

where τ_f = ultimate shearing resistance

$(\sigma_n - u_a)$ = net mechanical stress normal to failure plane defined as the difference between total overburden stress σ_n and the pore-air pressure u_a

$(u_a - u_w)$ = the difference between pore -air pressure u_a and pore-water pressure u_w

ϕ' = effective friction angle

θ = volumetric water content = volume water/total volume

f = factor ranging from $1/\theta$ to 1 depending on degree of saturation

As full saturation is approached, $f\theta$ tends to unity (Lytton, 1995), in which case Eq. 2a reduces to:

$$\tau_f = (\sigma_n - u_w) \tan \phi' \qquad (2b)$$

Stability Analysis.

Applying the pore-water pressure and strength relations in Eqs. 1 and 2b to an infinite slope analysis leads to the following expression for the factor of safety FS against sliding:

$$FS = \left(\frac{\gamma_b}{\gamma}\right)\frac{\tan\varphi'}{\tan\beta} - \frac{u_{w0}\tan\varphi'}{\gamma H \sin\beta\cos\beta} - \sqrt{2/3}\, a_f \tan\varphi' \qquad (3a)$$

where γ_b is the buoyant unit weight of the soil, γ is the total unit weight of the soil, u_{w0} is the pore-water pressure on the surface of the slope, and a_f is the Henkel coefficient at the failure state.

The first term in Eq. 3a represents the contribution of mechanical stress to the stability of the slope, with a reduction factor, γ_b/γ, for the seepage condition. Noting again that u_{w0} is negative, the second term represents the contribution of soil suction to stability. The third term accounts for the effects of shear-induced pore pressures. Fig. 2 graphically depicts the relative contributions of the mechanical stress, suction and shear-induced pore pressure terms for the case of a 3H:1V slope comprised of a soil with friction angle $\phi' = 25°$. The significance of suction is clearly apparent. However, at low factors of safety, *FS*, the suction and mechanical stress terms contribute to stability in comparable proportions.

Evaporation and Infiltration.

For conditions of uniform soil permeability, Eq. 3a can be readily modified for moisture flowing from or into the slope due to evaporation or infilration, respectively. Unfortunately, a uniform permeability in an unsaturated soil is far from realistic due to the dependence of permeability on the level of soil suction. Nevertheless, the analysis does permit some valuable qualitative insights into the effects of evaporation and infiltration on slope stability. If moisture flow across the face of the slope occurs (i.e., evaporation or infiltration) Eq. 3a becomes:

$$FS = \left(\frac{\gamma_b}{\gamma}\right)\frac{\tan\varphi'}{\tan\beta} - \frac{u_{w0}\tan\varphi'}{\gamma H \sin\beta \text{in}\beta\text{c}} - \sqrt{2/3}\, a_f \tan\varphi' - \left(\frac{\gamma_w}{\gamma}\right)\frac{\sin\delta}{\cos\beta}\tan\varphi' \qquad \text{(Eq. 3}b\text{)}$$

where tan δ is ratio of the hydraulic gradient of water flow normal to the slope to that parallel to the slope (Fig. 1), with a positive δ denoting moisture exiting the slope and vice-versa.

Eq. 3b shows a negative δ (moisture infiltration) to increase the factor of safety *FS*. Hence, while moisture infiltration into a slope will degrade its stability in the long term by decreasing the suction u_w, the favorable hydraulic gradients during infiltration will tend to counteract the effects of this suction loss. In contrast, during an evaporation phase, the $\delta > 0$ condition degrades the stability of the slope. This implies that the most critical condition experienced by a slope is a period of evaporation following a prolonged infiltration period; i.e., when the spatial extent of suction reduction is at its maximum and the direction of the hydraulic gradient is unfavorable for stability.

Case Histories in Texas High Plasticity Clays

The occurrence of shallow slope failures in high plasticity clays is quite common in east Texas. Kayyal and Wright (1991) investigated in detail a number of shallow slides that occurred in embankments constructed of high plasticity Paris and Beaumont clays. Selected data compiled from the Kayyal-Wright study is presented in Table 1. The ages of the embankments at the time of failure ranged from 14-19 years in the Paris clays and 12-31 years in the Beaumont clays. For all slides, Paris and Beaumont, the measured vertical depths of the slides ranged from 0.7 to 3 m, and the slope angles ranged from about 16 to 25 degrees from horizontal.

Material parameters. – Kayyal and Wright (1991) report a liquid limit LL= 80 and a plastic limit PL=22 for the Paris Clays and a liquid limit LL= 73 and a plastic limit PL=21 for the Beaumont Clays. Both soils are classified as fat clays CH by the Unified Soil Classification System. Actual unit weight data for the clays in situ at or near the time of failures are not available. However, after reviewing compaction data on these clays by Kayyal and Wright (1991), and Rogers and Wright (1986), total unit weight values of γ= 18.5kN/m^3 and γ= 19.5kN/m^3 were assumed for the Paris and Beaumont clays, respectively. Internal friction angles of $\phi'=25^{\circ}$ were estimated for the Paris and Beaumont clays based on a correlation between plasticity index PI and constant-volume friction angles proposed by Mitchell (1976).

Since the conditions of drainage and consequently the shear-induced pore pressure response were not known, the failures were back-analyzed for a range of plausible shear-induced pore pressures. While compacted soils can typically be expected to exhibit dilative behavior, it must be recalled that the near-surface soils on the slopes are subjected to wetting. Kayyal and Wright (1991) performed a series of consolidated-undrained (CU) triaxial shear tests on Paris and Beaumont clays for (1) compacted soils subjected to subsequent wetting, and (2) specimens of the same soils that had been normally consolidated from slurries. Their results indicated that the compacted wetted soils behaved essentially the same as the normally consolidated sedimented specimens. Further evidence that wetting a soil tends to erase the memory of previous mechanical stress is provided by Stark and Duncan (1991) who found that soaking specimens of highly over-consolidated natural clays produced specimens that acted essentially like normally consolidated clay. Therefore, an upper estimate of shear-induced excess pore pressures, a_f = 1.4, was selected for the back-calculations, which corresponds to a Skempton A-parameter at failure A_f = 1 typical of a normally consolidated soil in triaxial compression. To account for the possibility that the shearing process is slow enough to permit drainage, the slope failures were also back-analyzed assuming no excess shear-induced pore pressures.

Based on the material parameters and pore pressure assumptions described above, and a known failure condition (FS=1), an apparent matric suction on the surface of the slope u_{w0} was back-analyzed using Eq. 3a. These are tabulated in Table 2 for u_{w0} in units of pressure (kPa) and on a pF scale, where u(pF) = $\log_{10}(-h)$, with h being the matric suction measured in cm of water. The motivation for the use of the pF scale to characterize suction will become apparent in later presentation of the moisture diffusion analyses. The back-analyzed failures indicated apparent matric suctions on the surface of the slope ranging from u_{w0} =1.9-2.3pF in the Paris clays and 1.7-2.1pF in the Beaumont clays.

Discussion of Slope Stability Studies

Evidence of a flow condition.

In all cases analyzed the estimated angle of internal friction of the soil exceeded the slope angle. Hence, in the absence of a destabilizing hydraulic gradient, the slopes should have had factors of safety greater than unity even without the stabilizing effect of negative pore-water pressures. Unless a plausible case can be made for friction angles in the field being lower than laboratory measurements – a topic that will be discussed subsequently – this can be construed as rather compelling evidence that a destabilizing moisture flow condition did in fact exist in these slopes and the contribution of mechanical stress to the factor of safety against sliding is reduced by the factor γ_b/γ. It is again emphasized that a groundwater table near or at the slope surface is not necessary to produce a condition of flow parallel to the slope. A simple condition of constant pore water pressure (or suction) on the slope surface can create this flow pattern irrespective of whether the pore pressures are positive or negative.

Friction angle degradation

A conceivable alternative explanation for the occurrence of slope failures on slopes flatter than the internal friction angle of the soil is that the friction angle of the soil degrades toward a residual value. Making this argument plausible requires that one identify a mechanism for the development of the large cumulative shear strains needed for the development of a residual strength condition, values that typically exceed 100% (Kulhawy and Mayne, 1990). In slopes containing pre-existing slide planes of weakness, e.g., reactivated landslides (Skempton, 1964, 1985), a residual condition could develop at smaller displacements. However, a history of previous sliding was not reported for the cases considered in Table 1. Stark and Duncan (1991) do in fact make a convincing argument that cyclic straining due to reservoir operations led to the development of a residual strength condition in the foundation clays and consequent slide in the upstream slope of San Luis Dam. However, in the case of the shallow slides considered in this paper, no similar mechanism is envisioned for the occurrence of cyclic strains of sufficient magnitude to lead to a residual strength condition.

Regarding the effects of wetting of soils, the laboratory studies of Kayyal and Wright (1991) and Rogers and Wright (1986) indicated that wetting of compacted soils leads to a dramatic reduction in cohesion, but effective friction angles remained consistent with the constant-volume friction angle of the clay in its normally consolidated state. Stark and Duncan (1991) appeared to have a similar experience with natural clays where, after soaking, stiff clays experienced a dramatic loss of cohesion and effective friction angles remained consistent with the constant-volume friction angle of the clay in a normally consolidated state. Hence, there seems to be no compelling evidence at this time indicating that wetting can reduce frictional resistance to residual levels.

In view of the above discussion and the fact that the analysis summarized by Eq. 3a appears to adequately characterize the slope failures, there appears to be little reason to believe that the strength properties of the soils in these slopes degraded to levels below that of the clays in a normally consolidated state.

Wet limit of suction.

The lower limit of suction (wet limit) in a clay exposed to wetting is significant, since it permits an estimate of the lower limit to which soil strength can degrade. For the moisture diffusion analyses presented subsequently in this paper, the wet limit of suction also establishes the appropriate boundary condition for surfaces of the clay mass exposed to moisture. While some scatter exists in the back-calculated matric suction values at the surface of the slide mass u_0 at the time of failure (Table 2), an average value seems to be on the order of u = 2pF. Much of the scatter can be attributed to uncertainties regarding the exact conditions of drainage during shear and moisture flow in the slope at the time of failure. Nevertheless, a clear picture emerges indicating that the suction in an intact soil on a free surface exposed to wetting degrades to a finite non-zero value as full saturation is approached. In general, one would expect that this lower limit of suction will depend on soil type and that it could be substantially lower in lean clays and silts. In fact, the somewhat more plastic Paris clays (LL=80) showed a higher range of u_0 =1.9-2.3pF than the Beaumont clays (LL=73) which showed a range of u_0 = 1.7-2.1pF. The finding of a lower limit of matric suction on the order of u = 2pF is consistent with the findings of Lytton (1997) who, in conducting soil suction profiles at various clay sites in Louisiana and Texas using the filter paper test, encountered no instances of *total* suction measurements less than u = 2.5pF, even in very wet Louisiana swamp soils. Recognizing that the total suction measurements by Lytton included some component of osmotic suction, the apparent field capacity of matric suction of u =2pF back-calculated from the slope failures is consistent with Lytton's measurements.

The wet limit of suction described above may be considered analogous to the air-entry value, or bubbling pressure, in unsaturated soil mechanics literature (Fredlund and Rahardjo, 1993). The chief difference is that reported air-entry values typically correspond to drying portion of the soil-water characteristic curve, while the wet limit discussed above corresponds to wetting of the soil. Due to the hysteresis of the soil-water characteristic curve (Fredlund and Rahardjo, 1993), the wet limit limit can be expected to be somewhat lower than the air-entry value from a drying test. Air-entry values typically exceed 2.4pF for clays (Kovacs, 1981), and Aubertin et al. (1993) report air-entry values from 2.2 to 2.8pF for silt-bentonite mixtures. The range of wet limit values estimated from the slope failures considered in this paper, 1.7 to 2.3pF, are therefore somewhat lower than typical air-entry values for clays. However, the difference is not considered unreasonable in view of the above-mentioned hysteresis in the soil-water characteristic curve.

Apparent phreatic surface.

A matric suction of $u = 2$pF at the surface of the slope implies (see Eq. 1) a depth to a phreatic surface (the line of zero pore pressure) on the order of 1m. This phreatic surface should not be confused with a regional groundwater table, as it is associated with the localized region of wetting near the surface of the slope.

Time to Failure

The stability analyses presented above are based on the assumption that moisture enters the soil mass thereby decreasing the magnitude of the suction and degrading the strength of the soil. If this process progresses to a sufficient depth, a sliding failure occurs. Since a substantial period of time elapses prior to failure, one to three decades (Table 1), the mechanism of moisture infiltration into the slope merits attention. A particular focus of this aspect of the study is to establish a framework for estimating the time interval required for moisture introduced at the boundaries of a soil mass to diffuse into the interior.

In addition to the moisture diffusion characteristics of the soil, the moisture infiltration process will depend on conditions of local drainage, climate, vegetation, and surface cracking. For the slopes considered in this study, information on these conditions is unavailable. Further, in practical design situations, a designer can seldom anticipate these conditions with a high degree of certainty. Given the uncertainties in the boundary conditions, rigorous non-linear analyses cannot be justified and the simplified approach described below was adopted for this research.

Flow through Unsaturated Soils

Simplified formulation for unsaturated flow.

The permeability of an unsaturated soil is dependent upon the degree of saturation or suction level in the soil. A number of equations describing the permeability-suction relationship are presented in the literature; this study employed a relationship proposed by Laliberte and Corey (1966):

$$k = k_0 \, (h_0 / h)^n \qquad \text{(Eq. 4)}$$

where k_0 = reference permeability (saturated)

h_0 = total head at reference state

h = total head

n = material constant

Similarly, provided that the absolute value of suction h is greater than the absolute value of the reference head h_0, the moisture characteristic relationship can be described using:

$$\frac{dw}{dh} = -c \frac{1}{|h|^m} \qquad \text{(Eq.5)}$$

where w is the gravimetric moisture content, and c and m are moisture storage material parameters.

Applying Darcy's law and the conservation of mass principle for an incompressible fluid to Eqs. 4 and 5 produces the following equation for flow in the x-direction:

$$k_0\left(\frac{h_0}{h}\right)^n \frac{\partial^2|h|}{\partial x^2} = c\frac{\gamma_d}{\gamma_w}\frac{1}{|h|^m}\frac{\partial|h|}{\partial t} \qquad \text{(Eq.6)}$$

where γ_d is the dry unit weight of the soil and γ_w is the unit weight of water.

This non-linear partial differential equation in Eq. 6 can be solved using numerical methods. However, if n=m the independent variable h can be transformed to ψ such that (Aubeny et al., 2003):

$$d\Psi = |h|^{-n}\,dh \qquad \text{(Eq. 7a)}$$

$$\Psi = \log_e|h| \qquad n = 1 \qquad \text{(Eq.7}b\text{)}$$

$$\Psi = \frac{|h|^{1-n}}{1-n} \qquad n > 1 \qquad \text{(Eq.7}c\text{)}$$

Substituting Eqs. 7 into Eq. 6 and generalizing to three dimensions leads to the following linear partial differential equation:

$$\nabla^2\Psi = \frac{1}{\alpha}\frac{\partial\Psi}{\partial t} \qquad \text{(Eq.8}a\text{)}$$

$$\alpha = \frac{k_0|h_0|^n}{c}\frac{\gamma_w}{\gamma_d} \qquad \text{(Eq.8}b\text{)}$$

Assuming n=m=1 in Eq. 7b corresponds to an analytical procedure originally proposed by Mitchell (1979) in which unsteady flow through unsaturated soils is analyzed by solution of a linear partial differential equation with suction expressed on a pF scale; i.e., $u = \log_{10}(-h)$.

Several possible limitations to the simplified approach must be noted:

- The exponent n is not necessarily equal to unity as assumed by Mitchell (1979), and Aubeny et al. (2003) show some cases in which n>1 provides better agreement with measurements. Further, published literature on the subject (e.g., Brooks and Corey, 1964) generally indicate n to be unequal to m. Nevertheless, Tang (2003), in a series of moisture diffusion tests on high plasticity clays (to be discussed subsequently), showed that an assumption of n=m=1 in Eq. 8 provided adequate agreement between theory and measurements in a majority of cases.
- Hysteresis in the moisture-characteristic curve is not modeled. However, measurements of the diffusion coefficient α under conditions of wetting and drying (Mitchell, 1979) showed agreement to within about 20%, which was considered adequate by the Authors for the present investigation.
- Eq. 8 is strictly applicable for flow at a constant elevation. While moisture infiltration into a slope does not occur at a constant elevation, as will be discussed subsequently, the changes in pressure head during this process will typically be much larger than the changes in elevation head. Hence, the last limitation noted above is not viewed as particularly severe in the present problem.

- In the final stages of wetting, the magnitude of the matric suction h may decline below the value of h_0 (Eqs. 6 and 8) for which the analytical framework is strictly valid.

In spite of these limitations, the simplified approach presented herein has several notable advantages. In particular, the moisture diffusion coefficient α (Eq. 8b) can be interpreted with little ambiguity from a relatively simple laboratory test described in the next section and measurements show a remarkably good conformity to the simplified theory (e.g., Fig. 4). Further, for cases with simple boundary conditions, analytical solutions are possible with the linearized formulation. Such closed-form solutions can be particularly useful in understanding the basic mechanisms of moisture infiltration.

Measurement of diffusion coefficient, α.

Measurement of the permeability and moisture diffusion characteristics of high plasticity clays is a particularly challenging task for which, due to the minute quantities of water flow involved, one is continually confronted with the possibility of measuring the permeability of the test apparatus rather than that of the soil. The dependence of permeability on suction in partly saturated soils further complicates the issue, since inducing a hydraulic gradient in a soil specimen creates non-uniform conditions of suction, and therefore non-uniform permeability (Eq. 4), in the specimen. Experimental methods that avoid flow volume measurements can provide an effective means of estimating moisture diffusion properties of soils when very small flow volumes are involved. The drying and wetting tests proposed by Mitchell (1979) and the similar 'instantaneous profile' method (Hamilton et al., 1981; Fredlund and Rahardjo, 1993) are examples of this approach. These tests impose suction or flux conditions at the boundaries of the soil specimen while measuring suction in the interior of the specimen. The suction measurements expressed as a function of space and/or time provide a basis for estimating permeability or moisture diffusion properties. The linearization of the formulation (Eqs. 7 and 8), despite the noted limitations, provides an effective means of dealing with non-uniform permeability and moisture storage characteristics in the specimen.

This study utilizes a drying test originally proposed by Mitchell (1979) for estimating the diffusion coefficient α. Based on the relatively minor differences in the wetting and drying test results reported by Mitchell (1979), diffusion coefficients measured in drying tests may reasonably be applied to the conditions of wetting that actually occur in the slope. In the drying test six psychrometers are inserted into an undisturbed soil core sample (Figure 3). The sample is initially sealed on all boundaries. After initial suction measurements are recorded, one end of the sample is exposed to the atmosphere. From Kelvin's equation (Fredlund and Rahardjo, 1993), the suction on the boundary now becomes:

$$h = (\rho_w R\, T_a/M) \log_e (RH) \qquad \text{(Eq. 9)}$$

where ρ_w = the mass density of the water (1,000 kg/m^3 for water)

M = molecular weight of water (0.01802 kg/mole for water)

T_a = absolute temperature, degrees Kelvin

R = universal gas constant, 8.314 N-m/mole-^{o}K

RH = relative humidity

Since the relatively humidity in virtually any soil specimen (typically greater than 99%) will exceed that in the laboratory, moisture will evaporate from the soil with corresponding increases in the magnitude of suction in the soil specimen. A solution to this boundary value problem presented by Mitchell (1979) is included in the Appendix. The diffusion coefficient α can be directly estimated by optimization to obtain a best fit between measured suction and Mitchell's solution (Aubeny et al., 2003). In principle, suction measured as a function of time and space $u_m(x,t)$ can be used as a basis for estimating α. In practice, Aubeny et al. (2003) found that the suction near the sealed end of the specimen (Psychrometers 1, 2, and 3 in Fig. 3) varies very little during the test; hence, it is difficult to distinguish between real changes in suction from noise in the measurements. They therefore recommended estimation of the diffusion coefficient α based on suction versus time measurements for the psychrometers nearest the exposed end, particularly Psychrometer 6 in Fig. 3.

Values of α for high-plasticity clays.

Undisturbed samples were not available from the Paris and Beaumont failure sites listed in Table 1. However, the Texas Department of Transportation provided a number of 7.5-cm diameter, high plasticity clay samples from a highway embankment near Waco, Texas for measurement of moisture diffusion properties. These samples had index properties similar to those of the Paris and Beaumont clay, with liquid limits ranging from LL=60-71 and plasticity indices PI=40-45. The specimens were tested using the procedure described above, with specimen lengths ranging from 19 to 29 cm. Curve fits for estimation of the diffusion coefficient α coefficients were performed for a variety of n values, ranging from n=1 to 3. In most cases, n=1 provided satisfactory fits between measurements and Eq. A.3 (Tang, 2003); therefore, the analyses presented in the remainder of this paper are based on an assumption of n=1. Measurements from a typical test are shown in Fig. 4, and interpreted α from all tests are presented in Table 3. These results indicate an average diffusion coefficient α= 0.085 m^2/yr, with a standard deviation of 0.041 m^2/yr.

Moisture Diffusion Predictions

The analytical framework (Eq. 8) and estimated diffusion coefficients α provide a basis for estimating the time interval required for moisture introduced at the surface of the slope to migrate to a depth sufficient to induce a slope failure. In principle, the calculations should proceed in terms of total head, since the gradient of total energy governs fluid flow. However, since non-uniform condition of osmotic suction are not to be considered in the analyses – in fact measured osmotic suction data are not available for these site - Eq. 8 is equally valid if Ψ is taken as a measure of matric suction.

Due to the linearity of Eq. 8, changes in suction over time can be expressed in term of net normalized suction $U = (u_i - u)/ (u_i - u_b)$, where u_b is the suction at the boundary and u_i is the initial suction after construction of the slope expressed on a pF scale. Based on the discussion earlier regarding the 'wet limit' of suction in a clay, boundaries of the slope exposed to prolonged wetting may be assigned a matric suction u = 2pF. Based on the experience of the Authors, a typical suction in a compacted high plasticity clay prior to wetting is in the range u_i = 3.5-4pF

The analytical predictions presented below will address two slope conditions: an intact soil mass and a soil mass in which surface cracks exist. Cracking will almost inevitably occur in a bare slope or a slope protected by vegetative cover; hence, a cracked condition best represents the cases listed in Table 1. However, analysis of an intact condition provides a useful reference point for evaluating slope performance. Further, an intact condition is actually a realistic approximation for protected slopes; i.e., slopes covered by concrete protective slabs referred to as 'riprap'; the performance of such protected slopes is of considerable interest to practitioners.

Intact slopes.

Removal of slabs and pavements will often show (Odom, 2002) that moisture eventually penetrates through joints in the slab such that the soil directly beneath the slab becomes extremely wet. Further, the presence of the slab tends to inhibit drying during dry climactic periods; hence, the soil directly beneath the slab is typically in a permanently moist condition. While the slab is often ineffective in preventing wetting of the soil, the permanently moist state is likely to inhibit the development of cracks. In view of this experience, a reasonable moisture diffusion model of this condition (Fig. 5a) is as follows: (1) an intact soil mass, and (2) a very wet condition, u = 2pF at the top surface of the soil mass directly beneath the slab. The question then arises as to how long it will take the moisture at the top surface of the soil mass to migrate to a critical depth at which sliding will occur.

The above condition is analogous to one-dimensional heat flow in a semi-infinite solid with a fixed temperature on the free boundary. The solution is published in a number of sources (e.g., Lawton and Klingenberg, 1996) and is conveniently expressed in terms of the complementary error function (erfc), in which suction is substituted for temperature:

$$U = \frac{u - u_i}{u_b - u_i} = \mathrm{erfc}\left(\frac{1}{2\sqrt{T}}\right) \qquad \text{(Eq.10}a\text{)}$$

where u_i is the initial matric suction, u_b is the matric suction at the top wetted boundary (assumed constant over time), and the dimensionless time factor T is related to real time t at any distance of interest from the free surface z by:

$$T = \alpha t / z^2 \qquad \text{(Eq.10}b\text{)}$$

Fig. 6 presents a plot of Eq. 10a from which it is evident that the suction u does not decline to a level approaching that at the wetted boundary until the time T is well above 10. The coordinate z of interest in Eq. 10b is of course the depth at which a slide can occur, which from Table 1 is on the order of 1.5m. Table 3 provides a range diffusion coefficient values, $\alpha = 0.044\text{-}0.13 m^2/yr$. Solving Eq. 10b for real time t implies times to failure for protected slopes on the order of hundreds of years. This is of course well beyond the range of any of the documented slope failures in Table 1, which supports the assertion stated earlier that the effects of cracking must be incorporated into the model for moisture diffusion into unprotected (bare or vegetative cover) slopes.

While slide failures do occasionally occur in 'riprap' protected slopes, they are relatively rare and tend to occur in older slopes (Odom, 2002). It would therefore be reasonable to conclude that, while the riprap protection does not preclude moisture from entering a slope, by inhibiting crack formation it greatly slows the rate at which moisture penetrates to depths capable of creating stability problems.

Effect of Surface Cracking.

As no direct observational data on surface cracking are available for the slope failures in Table 1, the moisture diffusion model for a cracked slope was postulated based on empirical observations found in other studies. The field observations by Knight (1971) shown in Fig. 7 indicate that cracks tend to form in patterns in which the crack spacing equals the crack depth. Hence, a crack pattern develops such as that illustrated in Fig. 5b, with the deepest cracks occurring at the widest spacing and intermediate shallower cracks occurring at more frequent intervals. Noting that cracking occurs in three dimensions, a similar pattern of cracking is assumed to occur in a direction of the strike of the slope.

Based on the above observations on the general nature of crack patterns in clays, surface cracking was assumed to sub-divide the soil mass into a series of square columns with the column heights equaling the crack spacing. As surface water will easily penetrate into the cracks, moisture will diffuse into the soil mass from the crack surfaces thereby considerably reducing the length of the moisture migration path compared to that of an intact slope. While all of the cracks can provide conduits for moisture infiltration, the deepest cracks will be the least affected by drying periods and the most likely to remain permanently wet. Hence, as a first approximation only the deepest cracks are considered in the moisture diffusion analysis.

Neglecting the effects of the shallower cracks, the analytical model for moisture diffusion into the soil mass reduces to two-dimensional flow into a square region. The analytical solution for this boundary value problem can again be found in published solutions for unsteady heat flow, e.g., Powers (1972). For the case of a uniform initial suction, the solution is expressed by the following equation:

$$U = \frac{u - u_b}{u_i - u_b} = \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} A_{mn}\Phi_{mn} \exp\left\{-\pi^2 \ (m^2 + n^2)T\right\} \qquad \text{(Eq.11)}$$

$$A_{mn} = \frac{4}{\pi^2 mn}\left[\cos(m\pi) - 1\right]\left[\cos(n\pi) - 1\right]$$

$$\Phi_{mn} = \sin(m\pi\, x / L)\sin(n\pi\, y / L)$$

The dimensionless time factor T is as defined by Eq. 10b, with L replacing z as the distance scale factor such that $T=\alpha t/L^2$. Evaluating Eq. 11 at the center of the soil mass between the cracks, $x = L/2$, $y=L/2$, produces the solution shown in Fig. 6 for a cracked slope. This solution indicates that the wet condition, u = 2pF, on the surface of a crack migrates to the center or the soil mass at a time factor of about T_f=0.3.

As a means of determining whether the value above is reasonable, the data from Tables 1 and 3 can be applied to Eq. 10b. The assumed soil block dimension L is $H \cos \beta$ (Table 1), and t is the age of the slope (also Table 1). Taking the mean plus and minus the standard deviation of the moisture diffusion coefficient measurements gives upper and lower values of α = 0.044 to 0.13 m^2/yr, respectively. Time factors at failure estimated from these data are tabulated in Table 2. On average, the back-calculated time factors at failure in the Paris clays - back-calculated T_f=0.42-1.2 - are somewhat higher than the estimate from the theoretical model, T_f=0.3. In the case of the Beaumont clays, the underestimate from the theoretical model is more severe, with average back-calculated time factors being in the range T_f=1.2-3.7.

Discussion of Moisture Diffusion Analyses

Back-analyzed time factors

Time factors at failure T_f back-calculated from the postulated moisture diffusion model exceeded the theoretical prediction by factors of about 1 to 4 in the Paris clays and 4 to 10 in the Beaumont clays. Possible causes for the under-predictions could be an overestimate of the moisture diffusion coefficient α or, more likely, the relatively crude estimate of the cracking pattern and suction boundary conditions in the surface of the slope. However, noting that the back-calculated estimates of the time to failure T_f systematically exceed the theoretical prediction, a more fundamental issue is the fact that a significant time period may be required for cracks to develop in the slopes, particularly to the greater depths associated with a slide failure. It is noteworthy that the longer times to failure were in the Beaumont clays in the Gulf of Mexico coast area that, as a moist region, has less frequent and less severe dry periods capable of inducing desiccation cracking in the soils. In contrast, the Paris clay sites are further inland in Texas in a region subjected to more frequent and prolonged dry periods; hence, the rate of development of desiccation cracks would be expected to be considerably higher than in the coastal areas of Texas.

Moisture diffusion process

Investigators of shallow slide failures (e.g., Kayyal and Wright, 1991) have noted that the failures are often preceded by heavy rains. This raises a possible question as to whether the occurrence of slope failures is governed by a single climactic extreme - i.e., an unusually severe rain - rather than the prolonged continuous moisture diffusion process postulated in this paper. In the view of the Authors, heavy rains can *trigger* a slope failure, but only after a relatively long period of crack formation and moisture diffusion has already weakened the slope. This view is supported by the age of the embankments at failure that range from 12 to 31 years. Slopes of this age had undoubtedly been exposed to previous extremes of moisture prior to the final event that triggered the failure.

Significance of cracking

Previous reference has already been made to 'riprap' protected slopes and the observation that the protection usually does not provide a watertight seal (Odom, 2002). This observation has led some designers to question the effectiveness of such protection. While not an effective barrier against moisture, the protection appears to prevent extremes of drying that lead to deep desiccation cracks that can later become conduits for moisture infiltration. Hence, while not necessarily precluding the possibility of failure, the protection can retard the development of cracks thereby providing considerable benefit in prolonging the life of a slope.

Incidence of failures

While slides in high plasticity clay slopes are common, they are not inevitable. Hence, explanations of failures must be consistent with the observed satisfactory performance of a majority of such slopes. This paper postulates that the conditions for failure – at least within a time frame of several decades – are that cracks must form and that climactic and surface runoff conditions must maintain the walls of the cracks in a wetted state for prolonged time periods. Local drainage conditions may easily be such that surface water does not feed the cracks with sufficient frequency to maintain such a condition. Hence, the proposed model is not inconsistent with the observed satisfactory performance of many high plasticity clay slopes.

Conclusions

Shallow slide failures in high plasticity clay slopes involve a number of complex issues including local moisture conditions, the distribution of pore water pressures in the slide mass, cracking of the slope, and the rate of moisture diffusion into the soil mass. Even under favorable circumstances, adequate characterization of these factors is difficult. In cases in which a slide has already occurred the situation is further complicated, since much critical information is typically destroyed. This paper presents simplified stability and moisture diffusion models in an attempt to explain both why and when the slides occur. Given the paucity of site data necessary to fully characterize past failures, the primary intent of these models is to improve our understanding of the mechanisms of shallow slide failures rather than to attempt a detailed simulation all of the processes involved.

The stability and moisture diffusion analyses of 16 slope failures in Paris clays and 18 slope failures in Beaumont clays suggest the following conclusions:

1. The slope failures are consistent with a condition of destabilizing hydraulic gradients. The existence of such a condition provides the simplest plausible explanation as to why failures would occur in slopes in which the angle of internal friction ϕ' of the soil is greater than the slope angle β.
2. Back-calculation of the apparent matric suction near the surface of the slopes at failure indicate a fairly consistent value of about u = 2pF for high plasticity clays. This 'wet limit' of suction represents a lower limit to which the magnitude of the

matric suction will decline when a free surface of soil is exposed to moisture without artificial disturbance of the soil. The magnitude of the wet limit of suction is likely to be dependent on soil type, with higher values associated with higher plasticity soils.

3. The observed failures are consistent with a phreatic surface located about 1m below the surface of the slope. This phreatic surface is associated with a localized region of wetting near the slope surface and is not in general associated with a regional groundwater table.

4. The time-dependent aspects of the slope failures can most likely be explained in terms of (1) cracking on the surface of the slopes, and (2) moisture entering the cracks and diffusing into the soil mass until the magnitude of the suction and strength decline to a critical level.

5. The linearized moisture diffusion analyses for unsaturated soils provide useful first order approximations to the rate of suction change and strength loss in the slope soils.

6. Estimates of the moisture diffusion coefficient α based on the drying test discussed in this paper appeared to be consistent with the time frame of the slope failures when cracking of the soil mass is taken into consideration; however, much remains to be done in predicting crack formation in slopes.

7. The moisture diffusion analyses presented in this paper highlight the importance of surface cracks in the soil and the need for predictive models and field measurements on this topic. In addition to slopes, the performance of a number of other civil structures – pavements, slabs, shallow foundations, retaining walls –

can be affected by moisture infiltration and therefore cracking; hence, research in this area can prove beneficial for a significant portion of the civil infrastructure.

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NOTATION

a_f = Henkel pore pressure coefficient at failure

c = coefficient for moisture characteristic curve

f= factor varying from $1/\theta$ to 1

h = total head

h_e = evaporation coefficient

i = hydraulic gradient

k = hydraulic conductivity

n = exponent in permeability relation

u_a = pore air pressure

u_w = pore water pressure

u_{w0} = pore water pressure on slope free surface

t = time

w = gravimetric water content

x, y, z = coordinates

A_f = Skempton pore pressure coefficient

H = vertical depth of slide mass

L = characteristic dimension in moisture flow analysis

T = dimensionless time factor

U = net normalized suction

α = moisture diffusion coefficient

β = slope angle measured from horizontal

ϕ' = effective angle of internal friction

γ = total unit weight of soil

γ_b = buoyant unit weight of soil

γ_d = dry unit weight of soil

γ_w = unit weight of water

π = osmotic suction

σ = normal stress

τ_{oct} = octahedral shear stress

Ψ = transformed measure of suction

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APPENDIX A– SOLUTION FOR EVAPORATION TEST

For the case of n=m=1 (Eqs. 6 and 7), the variable Ψ can simply be replaced by suction expressed on a pF scale, $u = \log_{10}(-h)$. Hence, Eq. 8 becomes:

$$\nabla^2 u = \frac{1}{\alpha}\frac{\partial u}{\partial t} \tag{A.1}$$

The sealed end is a simple no-flow boundary condition. At the open end, Mitchell (1979) imposes a prescribed flux condition defined by the relation:

$$\left(\frac{\partial u}{\partial x}\right)_S = -h_e\left(u_s - u_{atm}\right) \tag{A.2}$$

where h_e is an evaporation coefficient, u_{atm} is the atmospheric suction defined by Eq. 9, and u_s is the soil suction at the exposed boundary. By measuring gradients of suction near the open boundary, Mitchell (1979) estimated the evaporation coefficient h_e to be 0.54cm^{-1}. This value was adopted for the present study.

Applying these boundary conditions to Eq. A.1 leads to the following solution (Mitchell, 1979) for suction $u(x,t)$ as a function of time and coordinate in the soil sample:

$$u = u_a + \sum_{n=1}^{\infty}\frac{2\left(u_i - u_{atm}\right)\sin\lambda_n}{\lambda_n + \sin\lambda_n\cos\lambda_n}\exp\left[\frac{-\lambda_n^2\alpha\, t}{L^2}\right]\cos\left[\frac{\lambda_n x}{L}\right] \tag{A.3}$$

$$\cot\lambda_n = \frac{\lambda_n}{h_e L}$$

where u_{atm}= atmospheric suction

u_i = initial suction in soil

α = diffusion coefficient

t = time

L = sample length

x = coordinate

h_e = evaporation coefficient

Table 1a. Site Data for Shallow Slides in Paris Clays (after Kayyal and Wright, 1991)

Case	Location	Slope Age (years)	Slope Angle β (degrees)	Vertical Depth H of Slide (m)
1	Loop 286 @ T&P RR SE Quadrant, Lamar County	19	18	1.2
2	Loop 286 @ SH 271 NW Quadrant, Lamar County	14	22	1.2
3	Loop 286 @ Missouri Pacific RR SW Quadrant, Lamar County	18	19	2.4
4	Loop 286 @ Missouri Pacific RR SW Quadrant, Lamar County	18	20	1.8
5	Loop 286 @ Missouri Pacific RR NW Quadrant, Lamar County	18	20	3
6	Loop 286 @ FM 79 SW Quadrant, Lamar County	19	23	1.2
7	SH 271 North, SE of Missouri Pacific RR South Emb, Lamar Co	18	20	1.8
8	Loop 286 & Still House RR Overpass East Abut, Lamar Co.	18	23	1.8
9	Loop 286 & Still House RR Overpass, West Abut, Lamar Co.	18	18	1.5
10	Loop 286 @ SH 271 NW Quadrant, Lamar County	18	20	1.2
11	Loop 286 & SH 71 Overpass (North)East of RR, Lamar County	18	17	0.6
12	SH 271 North, SE of Missouri Pacific RR North Emb, Lamar Co	19	20	1.2
13	SH 271 South, NW of Missouri Pacific RR, Lamar Co	19	23	1.8
14	SH 271 South, SW of Missouri Pacific RR, Lamar Co	19	23	1.8
15	SH 271 East, W of Missouri Pacific RR, Lamar Co	19	18	1.2
16	SH 271 North, NW of Missouri Pacific RR, Lamar Co	19	20	1.2

Table 1b. Site Data for Shallow Slides in Beaumont Clays
(after Kayyal and Wright, 1991)

Case	Location	Slope Age (years)	Slope Angle β (degrees)	Vertical Depth *H* of Slide (m)
1	IH 610 @ Scott St., NE Quad, Harris County	17	21.8	1.1
2	SH 225 @ SH 146, SW Quad, Harris County	31	18.4	1.3
3	SH 225 @ SH 146, NW Quad, Harris County	31	17.9	0.7
4	SH 225 @ SH 146, SE Quad, Harris County	31	16.4	1.1
5	SH 225 @ SPRR Overpass, SE Quad, Harris County	20	21.0	1.2
6	SH 225 @ SPRR Overpass, SE Quad, Harris County	20	17.9	0.9
7	SH 225 @ SPRR Overpass, SE Quad, Harris County	20	22.6	1.5
8	SH 225 @ SPRR Overpass, NW Quad, Harris County	20	17.9	0.8
9	SH 225 @ Scarborough, SE Quad, Harris County	17	25.5	0.9
10	IH 610 @ SH 225, SE Quad, Harris County	19	20.3	0.6
11	IH 610 @ Richmond, SW Quad, Harris County	18	20.3	1.5
12	IH 10 @ Crosby-Lynchburg, NW Quad, Harris County	25	21.0	1.5
13	IH 45 @ SH 146, SE Quad, Harris County	14	18.4	0.9
14	IH 45 @ SH 146, South Side, Harris County	14	17.9	1.1
15	IH 45 @ SH 146, NE Quad, Harris County	12	21.8	0.8
16	IH 610 @ College St., NE Quad, Harris County	18	18.4	0.6
17	US 59 @ FM 525, NE Quad, Harris County	24	22.6	0.9
18	US 59 @ Shepard St., SE Quad, Harris County	22	17.9	1.1

Table 2a. Back-Calculated Suction and Dimensionless Time Factors at Failure for Shallow Slides in Paris Clays

Case	Back-Calculated Surface Suction, a_f=0		Back-Calculated Surface Suction, a_f=0.707		Time Factor at Failure, T_f	
	u_{w0} (kPa)	u_{w0} (pF)	u_{w0} (kPa)	u_{w0} (pF)	α = 0.044 m^2/yr	α = 0.126 m^2/yr
1	-4.9	1.7	-12.7	2.1	0.62	1.85
2	-7.5	1.9	-16.5	2.2	0.48	1.42
3	-10.8	2.0	-26.9	2.4	0.15	0.44
4	-8.9	2.0	-21.3	2.3	0.27	0.79
5	-16.0	2.2	-37.2	2.6	0.10	0.29
6	-8.8	2.0	-18.3	2.3	0.66	1.98
7	-8.9	2.0	-21.3	2.3	0.27	0.79
8	-13.2	2.1	-27.5	2.4	0.28	0.83
9	-6.2	1.8	-15.9	2.2	0.38	1.12
10	-6.4	1.8	-14.9	2.2	0.60	1.79
11	-2.1	1.3	-5.8	1.8	2.32*	6.92*
12	-6.4	1.8	-14.9	2.2	0.62	1.89
13	-13.2	2.1	-27.5	2.4	0.29	0.88
14	-13.2	2.1	-27.5	2.4	0.29	0.88
15	-4.9	1.7	-12.7	2.1	0.62	1.85
16	-6.4	1.8	-14.9	2.2	0.63	1.89
Average	-8.6	1.9	-19.8	2.3	0.42	1.25
Std. Dev.	3.8	0.2	7.9	0.2	0.20	0.59

*Excluded from average.

Table 2b. Back-Calculated Suction and Dimensionless Time Factors at Failure for Shallow Slides in Beaumont Clays

Case	Back-Calculated Surface Suction, a_f=0		Back-Calculated Surface Suction, a_f=0.7		Time Factor at Failure, T_f	
	u_{w0} (kPa)	u_{w0} (pF)	u_{w0} (kPa)	u_{w0} (pF)	α = 0.044 m^2/yr	α = 0.126 m^2/yr
1	-6.5	1.8	-14.7	2.2	0.76	2.25
2	-5.1	1.7	-13.3	2.1	0.88	2.61
3	-2.6	1.4	-7.1	1.9	2.79	8.32
4	-2.6	1.4	-8.7	1.9	1.29	3.85
5	-6.8	1.8	-15.5	2.2	0.67	2.01
6	-3.2	1.5	-8.8	2.0	1.15	3.44
7	-10.1	2.0	-21.7	2.3	0.44	1.31
8	-2.7	1.4	-7.4	1.9	1.66	4.95
9	-7.7	1.9	-15.4	2.2	1.09	3.25
10	-3.1	1.5	-7.3	1.9	2.54	7.56
11	-7.8	1.9	-18.4	2.3	0.38	1.15
12	-8.5	1.9	-19.4	2.3	0.54	1.61
13	-3.5	1.6	-9.3	2.0	0.81	2.42
14	-3.8	1.6	-10.3	2.0	0.59	1.77
15	-4.7	1.7	-10.3	2.0	1.05	3.12
16	-2.4	1.4	-6.2	1.8	2.35	7.00
17	-6.1	1.8	-13.0	2.1	1.47	4.38
18	-3.8	1.6	-10.3	2.0	0.93	2.78
Average	-5.0	1.7	-12.0	2.1	1.23	3.67
Std. Dev.	2.5	0.2	4.8	0.2	0.75	2.25

Table 3. Moisture Diffusion Tests in High Plasticity Waco Clays, LL=60-71, PI=40-45.

Test No.	Initial Suction, u_0 (pF)	Boundary Suction, u_a (pF)	Specimen Length, L (cm)	Psychrometer Location, x* (cm)	Interpreted α (cm^2/min)/ **(m^2/yr)**
1	4.20	5.64	29.21	25.40	.0018 /**.095**
3	3.90	5.80	22.22	19.69	.0012/**.063**
4	3.80	5.91	20.96	18.42	.0022/**.116**
5	3.95	5.74	20.32	17.78	.0010/**.053**
6	4.10	6.00	19.05	16.51	.00076/**.040**
7	3.70	5.62	21.59	19.05	.0028/**.147**
Mean Std Dev	----	----	----	----	0.0016/**.085** 0.00078/**.041**

*Distance from sealed end of specimen.

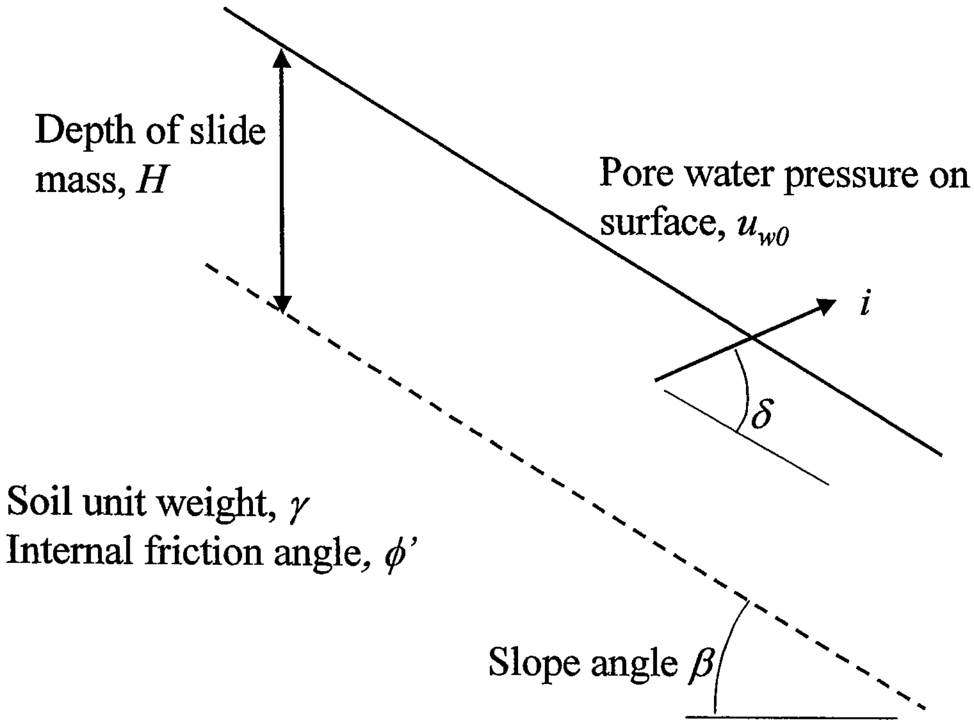


Figure 1. Definition Sketch for Slope Stability Analysis.

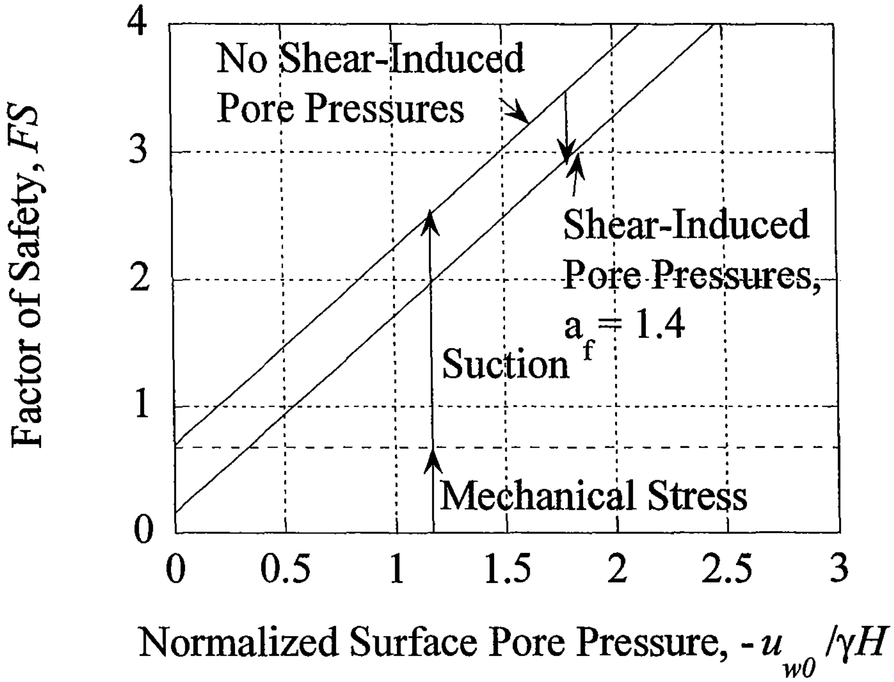


Figure 2. Contributions of Mechanical Stress and Suction to Stability for a 3H:1V Slope with $\phi' = 25°$.

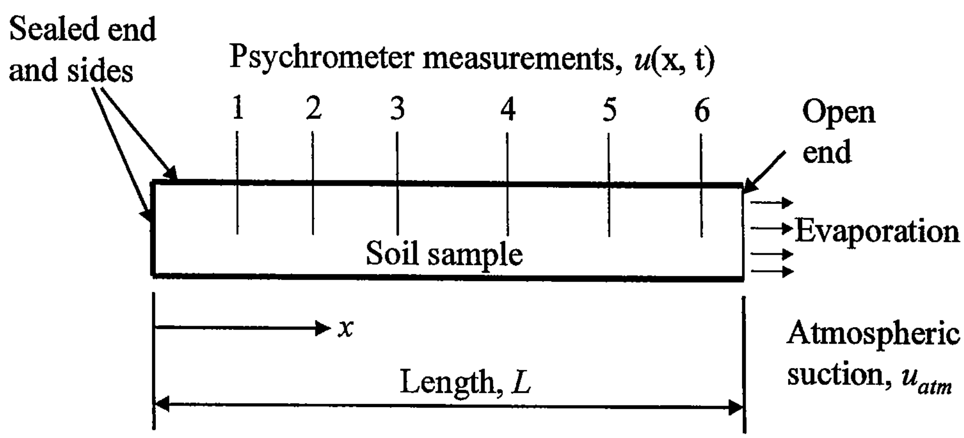


Figure 3. Dry end test for measuring α.

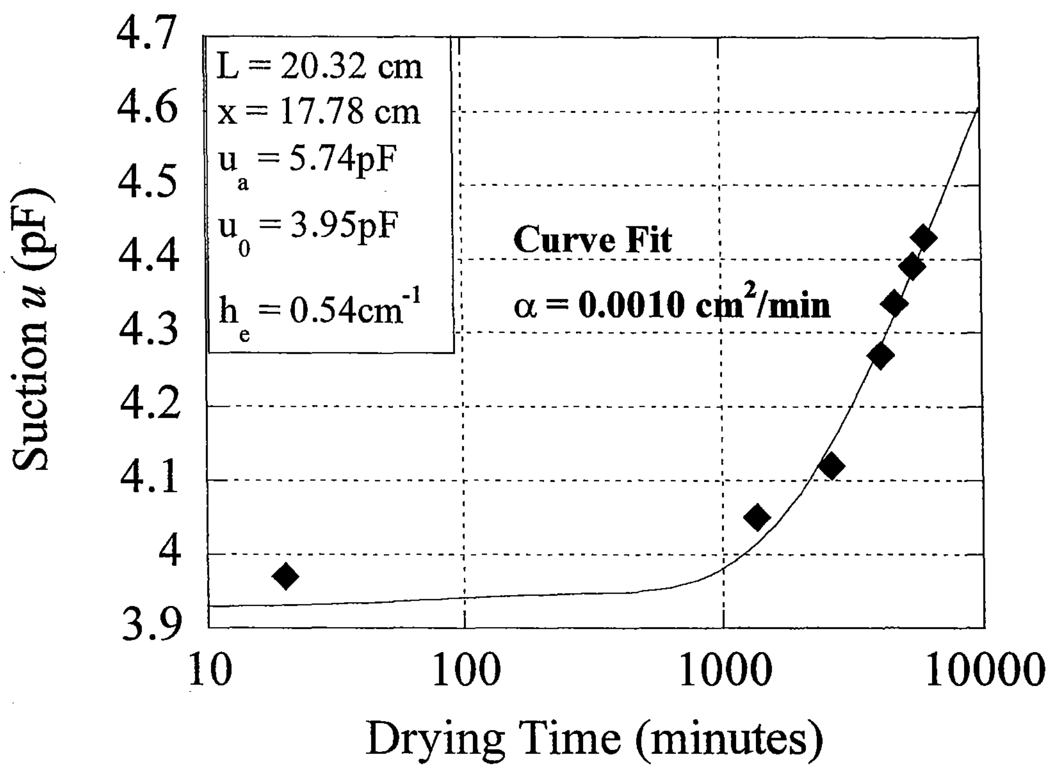
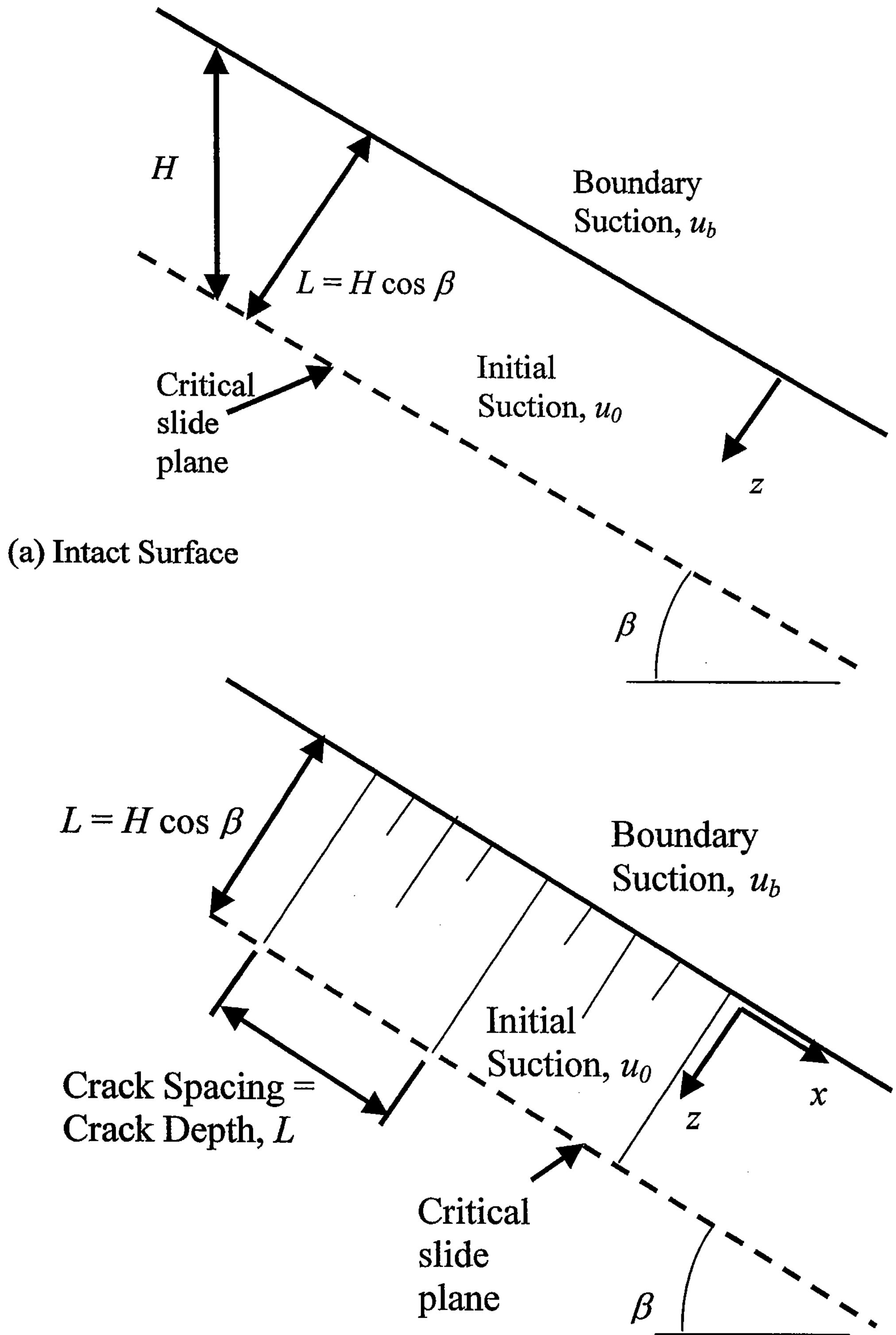


Figure 4. Typical Experimental Measurements for Determination of Diffusion Coefficient α.



(b) Cracked Surface

Figure 5. Definition Sketch for Moisture Diffusion Predictions.

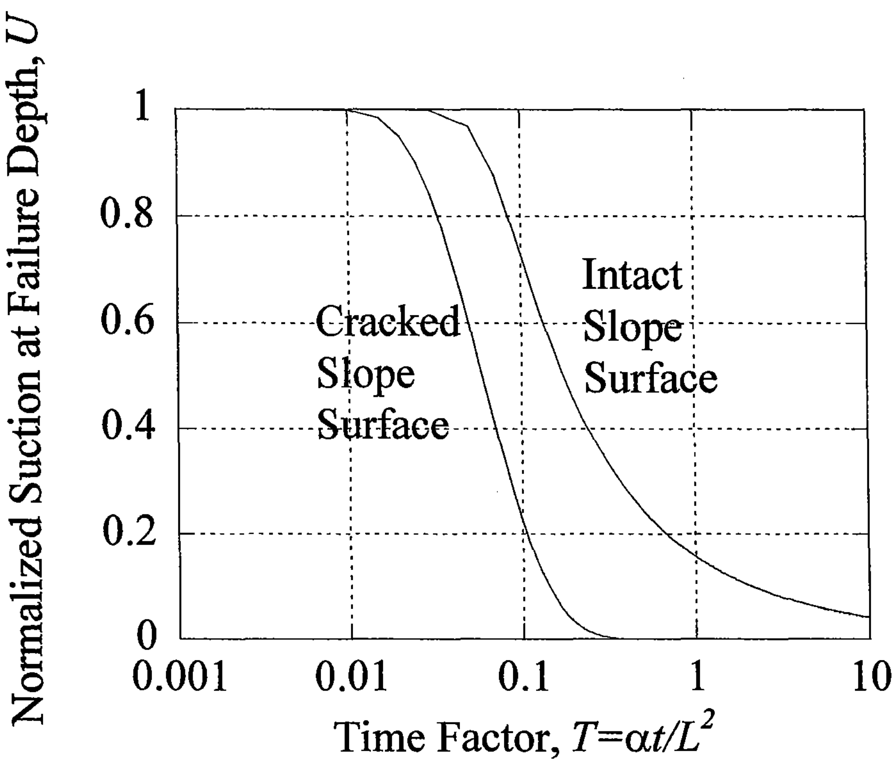


Figure 6. Predicted Suction History in Cracked and Intact Slope Surfaces

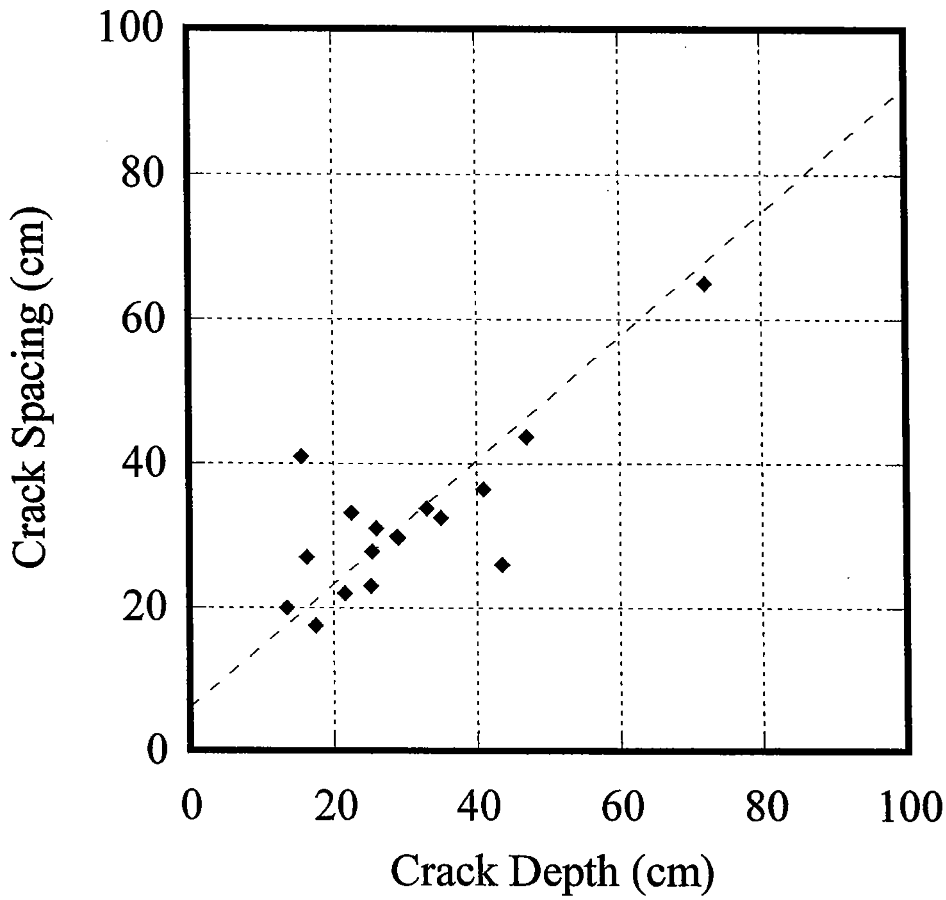


Figure 7. Measured Crack Patterns (after Knight, 1971).

Slab-on-Ground – A Finite Element Method Analysis

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Abstract

A finite element computer program has been developed to analyze slabs on elastic half space expansive as well as compressible soils. Mindlin orthotropic plate theory is adopted for structural analysis of ribbed or constant thickness slabs. The foundation soil is assumed to be an isotropic, homogeneous, and elastic half space. The behavior of an elastic half space is calculated by dividing the surface of the elastic half space into rectangular regions. These regions are represented by stiffness matrices and they are assembled onto the rectangular plate finite elements. The shape of the soil surface underneath the slab is described by the differential soil movement, y_m, and edge moisture variation distance, e_m. The mounded soil surface requires an iterative procedure in the computer program for this soil-structure interaction system. The program calculates displacements, moments in x- and y-directions, twisting moments, and shear forces. The comparisons of the results with the Post Tensioning Institute's (PTI) Design and Construction of Post-Tensioned Slabs-on-Ground manual examples show that the PTI analysis is conservative for the center lift case, but is not conservative for the edge lift case.

Introduction

The soil, which represents a great portion of the earth's surface, is very complicated to deal with in regard to its engineering behavior. The main problem is certainly the variety of its material properties which can make it elastic, plastic, nonhomogeneous, anisotropic, and compressible, expansive or collapsing. It is necessary to understand the properties of the supporting soil and also to describe its behavior mathematically in order to design a foundation properly. A geotechnical engineer dealing with the problematic soils is often faced with the need to calculate displacements of the foundation soil and to analyze the effects of the displacements on the slab.

Foundation design on an expansive soil presents a challenge to a geotechnical engineer because of the shrink and swell properties of these soils. Expansive soils swell when they absorb moisture from the environment and shrink when they lose moisture to the environment. Moisture movement in expansive soils is thus a major cause for volume change and this moisture movement is a result of unbalanced moisture energy (or soil suction) between the expansive soil and its environment. The moisture distribution does not occur uniformly within the soil underlying the foundation and thus results in differential soil movement. It is this differential movement that results in major distresses in the slab foundations. Seasonal changes of soil moisture or soil suction will dictate differential soil movement. Therefore, it is very important

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for a geotechnical engineer dealing with expansive soil to have knowledge of the soil suction distribution within the soil below a slab foundation.

Slab foundations have been analyzed using different approaches such as approximate numerical solutions, finite difference methods, and finite element methods. The finite element method incorporating the foundation soil has recently been widely accepted in analyzing the slab foundations because of its versatility and reliability over other methods. Two plate (or slab) theories are commonly used in finite element applications: Kirchoff plate theory and Mindlin plate theory.

The most current design procedure for the slabs on expansive soils is the method by the Post Tensioning Institute (1996). In the PTI design method, the analysis of the plate structure with the finite element method has some shortcomings. For instance, only rectangular slabs can be analyzed with the PTI method. For non-rectangular geometries, the rectangular slabs are overlapped to match the actual geometry. The PTI slab analysis is based on classical plate theory (Kirchhoff plate theory or thin plate theory), but an improved method should allow for thick plates if needed. Stiffening beams in the PTI method are converted to an equivalent slab thickness for calculating the bending moment, shear, and deflection and the method allows for a uniform distributed load all over the slab as well as line loads along the perimeter. However, an improved method needs to allow a slab cross-section that has stiffening beams and different magnitudes of distributed loads at different locations on the slab. Therefore, the research approach is to develop and conduct a finite element analysis of a slab resting on an expansive soil, which is modeled as an elastic half space, to predict the magnitudes of bending and twisting moments, shear, and deflection under applied design loads.

Foundations on Expansive Soils

If a foundation is placed on an expansive soil, the geotechnical engineer faces a major challenge because the soil can respond with a change in volume (shrinking/swelling). The expansive soil can respond not only to the structural loading but also to a change of soil moisture condition. The unique property of expansive soil is the change in volume when it absorbs moisture from its environment (swelling) or loses moisture to its environment (shrinking). Lightly loaded structures such as houses, apartments, and pavements have been affected by these reactive heaving soils (mainly smectite type clay) in many countries.

To employ the slab analysis procedures, the geotechnical engineer needs to predict the differential soil movement caused by the expansive soils. It is known that the climatic condition of a site is a major factor controlling the magnitude of the differential soil movement. The climatic condition of a site will determine the active zone, the possible maximum seasonal changes of soil moisture condition (wet and dry soil suction profiles, Fig. 1). Once the soil suction profile is obtained, the soil volume change induced by these soil suction changes can be estimated.

When a lightly-loaded structure such as a slab-on-ground foundation is constructed over expansive soils, the climate conditions at the site has a great influence on the type of distress that the foundation will undergo as a result of distortion of the support provided by the foundation soil. In general, there are two major types of expansive soil distortion modes (Lytton 1972): center-lift and edge-lift (Fig. 2). The center-lift case usually occurs when the soil at the perimeter of foundation shrinks. The edge-lift case usually occurs when the soil at the perimeter of foundation swells. Either type of distortion will result in structural damages if the slab is not designed properly. The distortion mechanism should be selected to produce the worst values of design moment, shear force and deflection (Lytton 1972).

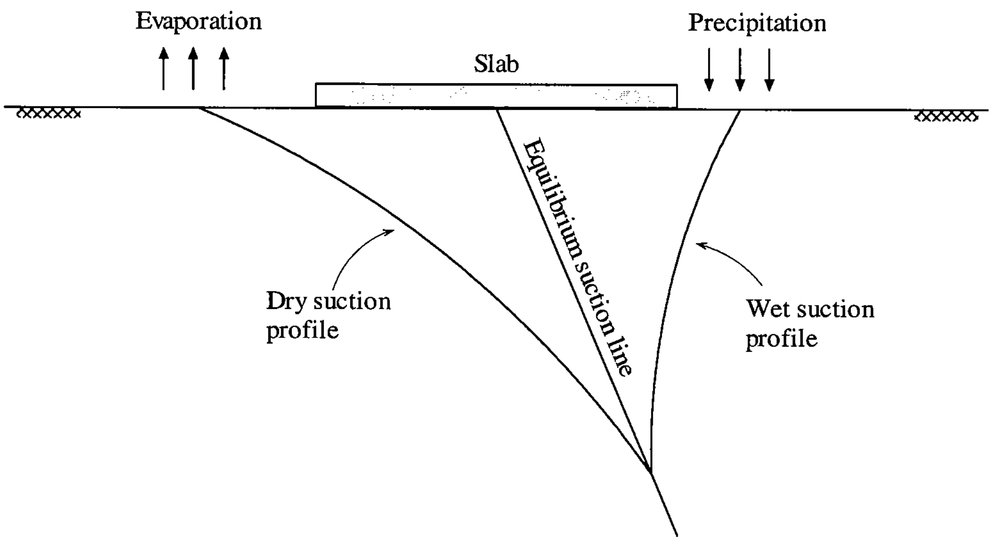


Fig. 1. Soil Suction Profiles.

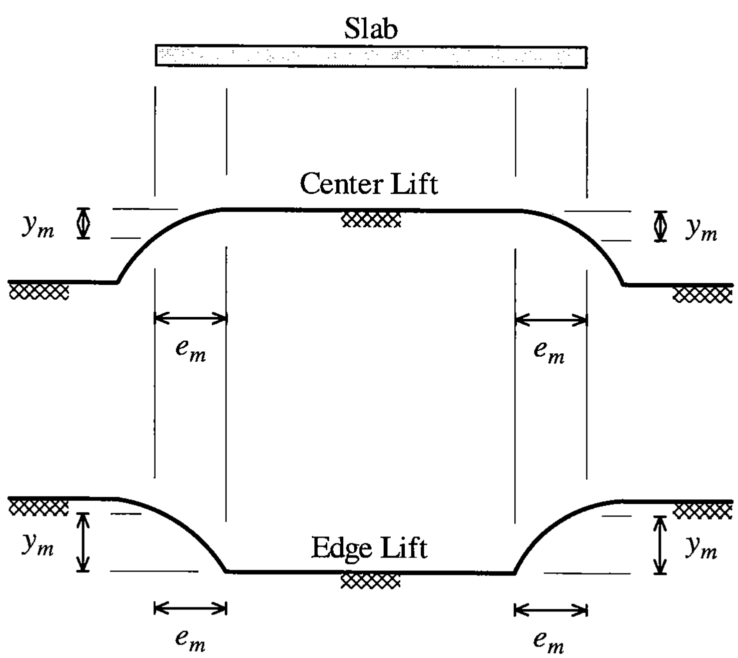


Fig. 2. Slab Distortion Modes.

Volume Change Behavior of Expansive Soils

The volume change behavior of expansive soils may not be predicted satisfactorily using traditional soil mechanics theories as well as elastic or plastic theory due to the large magnitudes of volumetric strains involved (Lytton 1996). However, there are many methods that rely on these theories as well as on some laboratory methods such as consolidometer tests or even on moisture content determinations. Lytton (1973) has shown that the volume change of expansive soils can be predicted satisfactorily with the use of soil suction, which has proven to be a stress state variable for unsaturated soils (Fredlund and Rahardjo 1993). For small increments of volume change, the volume strain is linearly related to the logarithms of both pressure and absolute value of suction (Lytton 1994). The general relation between the volumetric strain and the pressure and suction for a swelling soil is given by

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) \tag{1}$$

where h_i is the initial matric suction, h_f is the final matric suction, σ_i, is the initial mean principal stress, σ_f is the final mean principal stress, γ_h is the volume change coefficient due to shrinking or swelling, γ_σ is the volume change coefficient due to overburden, and $\Delta V/V$ is the percent volume change in decimal form.

This is a more rational approach since suction as a measure of the negative stress in the pore water, which pulls the soil particles together, is dependent on boundary conditions such as vegetation and climate. Suction is a thermodynamic quantity which will retain all the effects of the climate, such as humidity and temperature, within itself and carry them as a stress effect on the soil particles.

In addition, a more rational method has recently been developed for the determination of differential swelling soil profile, y_m, and of edge moisture variation distance, e_m, which are based on moisture diffusion/volume change relationship, by Naiser (1997). This procedure has been put forward as a new computer program known as VOLFLO Win 1.0 in 2002 by Geostructural Tool Kit, Inc. The current version of VOLFLO Win 1.0 is an improved and expanded computer program involving many of the principles of the previous VOLFLO program and at the same time incorporating the work done by Mitchell (1980) and Gay (1994). Mitchell (1980) developed and applied simple mathematical methods for predicting soil suction profiles. Gay (1994) developed a finite element program (FLODEF) for the transient moisture flow in unsaturated soils and a procedure to estimate the mean volumetric water content for soils dependent upon the location and climatic conditions.

The procedures mentioned above are applied to several moisture effect cases, in the new VOLFLO Win 1.0 program, that are common with light commercial and residential structures such as bare soils at the surface, grass at the surface, trees at the surface, and a flowerbed at the surface. Additionally, these procedures include calculating the effects of differential soil movement caused by the introduction of design effects such as vertical and horizontal moisture barriers.

Foundation Model

The analysis of the interaction between the slab foundations and the supporting soil foundation is of fundamental importance to geotechnical engineering. Many of the available interaction models are primarily concerned with elastic analysis. In this research, the slab and foundation soil interaction has been analyzed with the linear finite element method. To determine the vertical soil deformation in excess of the soil's expansion characteristics, the foundation soil needs be properly formulated. There is a spectrum of foundation models ranging from Winkler's type to the semi-infinite, homogeneous and isotropic, elastic continuum. Perhaps the best representation for the most frequently occurring soil materials is the elastic half space, behavior of which is described by Boussinesq's equation (Huang 1993). In this study, the supporting foundation soil for the plate is considered to be an elastic, isotropic, and homogeneous semi-infinite continuum with E_s and ν_s, modulus of elasticity and Poisson's ratio of the soil, respectively. The behavior of an elastic half space is calculated by dividing the surface of the elastic half space into rectangular regions.

For the case of an elastic continuum, since it is the response of the foundation within the contact area and not the stresses or displacements inside the foundation soil which are of particular interest in light commercial and residential structures. The problem reduces to finding a relatively simple mathematical expression which can describe the response of the foundation within the contact area with a reasonable degree of accuracy. Many researchers have attempted to create a convenient model that properly represents the physical behavior of a real foundation. Thus, a whole spectrum of foundation models is known; at one end is the Winkler model consisting of closely spaced, independent linear springs and at the other extreme is an elastic continuum. There is a large class of foundation materials occurring in practice which can be represented neither by a Winkler type foundation nor by an isotropic continuum. To find a physically close and mathematically simple representation of such models for the soil-structure interaction, there are attempts made by Pasternak, Hetenyi, Filonenko-Borodich, and Vlasov (Bulut 2001). Generally, a comparison between Winkler and elastic continuum foundations indicates that elastic continuum foundations are more realistic (Poulos 2000).

A summary of the Boussinesq's solution of the elastic half-space problem is given by Timoshenko and Goodier (1970). In the Boussinesq formulation, the deflection at any point depends not only on the force at that point but also on the forces at all other points (Fig. 3), which is a more realistic approach as compared to the Winkler's model. For the elastic half-space continuum model, the force-deflection relationship can be written as

$$w_{ij} = \frac{1-\nu_s^2}{\pi E_s} \frac{p_j}{r_{ij}} \tag{2}$$

where w_{ij} is the deflection at point i due to a force at point j, p_j is the force at point j, r_{ij} is the distance between points i and j, E_s is the elastic modulus of the foundation soil, and ν_s is the Poisson's ratio of the foundation soil.

The only known approximate solutions to evaluate Eq. 2 for the flexibility coefficients are the ones by Cheung and Zienkiewicz (1965) and Huang (1993). Cheung and Zienkiewicz (1965) considered the foundation consisting of a series of rectangular pressure areas whose centers coincide with the nodal points of the slab. The flexibility coefficients are obtained by integrating the Boussinesq equation over the rectangular element area for the points at which the Boussinesq equation is not defined.

A similar technique to that of Huang (1993) is adopted in this study for calculating the flexibility coefficients using a five-point Gauss quadrature formula in both x- and y- directions. The foundation flexibility matrix is determined in two ways: direct and numerical integration. The flexibility matrix coefficients can be obtained directly if the point at which the deflection is sought and the point at which the vertical unit load is applied is different. In other words, if $i \neq j$ (Eq. 2), then the coefficients are obtained directly. However, if the point of interest for the deflection and the applied vertical unit load coincide, then Eq. 2 becomes singular and thus a numerical integration technique can be employed to overcome the singularity. The stiffness matrix of the foundation soil is then obtained by inverting the flexibility matrix.

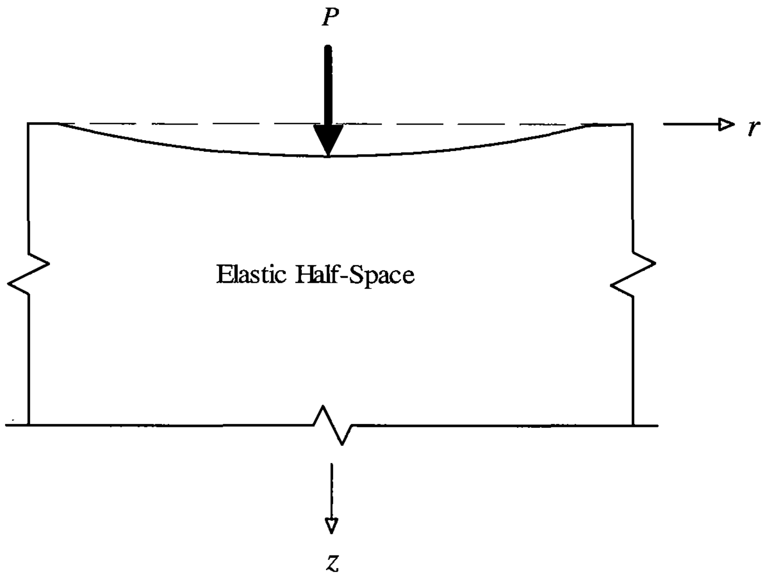


Fig. 3. Elastic Half Space Foundation.

Plate Finite Element Model

The finite element method may be regarded as a generalization of standard structural analysis procedures, in particular the displacement method of analysis, which permits the evaluation of displacements, strains and stresses in a structure. The finite element method is a very powerful method for the solution of differential equations that are in the fields of engineering. In the method, the structural domain is simply divided into regions (finite elements) of appropriate size and shape with all the material properties of the original domain being retained in the individual finite elements. By assuming approximate displacement functions (interpolation functions or shape functions) within an element, it is possible to derive the stiffness matrix of a structure using the principles of energy theorems or virtual work. If conditions of equilibrium are applied at every node of the discretized structure, a set of simultaneous algebraic equations can be formed, and the solution of these equations gives all the nodal displacements. The internal stresses are then obtained using the calculated nodal displacement values. A more complete treatment of the finite element method can be found in numerous books such as Reddy (1993), Zienkiewicz (1971), and Nath (1974).

Considerable research has been done for the development of finite plate elements for the analysis of the bending of plates. Researchers have developed quite a number of elements (i.e., rectangular, triangular, quadrilateral, etc.) with varying number of nodal points along with different types of interpolation functions. The aim of the researcher is to develop an element that has the least number of coefficients and at the same time satisfies the boundary conditions such as continuity of slopes. In this study, the linear finite element model of the shear deformation (or Mindlin or thick) plate theory for the rectangular elements is adopted. A simple four node-rectangular element is chosen because of the restrictions applied by the foundation model formulation. The four-node rectangular element has three degrees of freedom per node; one displacement and two rotations (Fig. 4). A more complete treatment of the plate theories can be found in numerous books such as Timoshenko and Winowsky-Krieger (1968) and Ugural (1981).

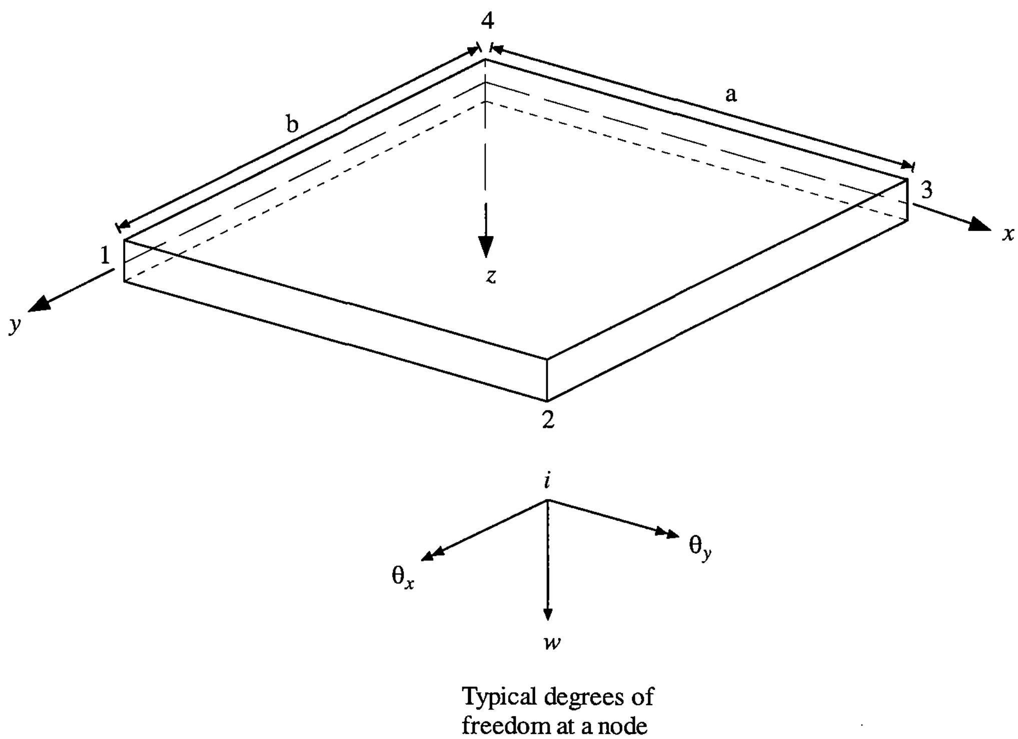


Fig. 4. Rectangular Bending Element.

Mindlin plate theory is very similar to the Kirchhoff theory except that it allows the transverse shear deformations within the plate. Therefore, this theory is very suitable for analysis of thick plates. The plate finite element model used in this research is based on the Mindlin plate theory. In the formulation of the plate element, the assumptions adopted in the shear deformable plate can be summarized as:

1) the plate is linearly elastic,
2) the plate undergoes small lateral deflections,
3) transverse normals do not elongate,
4) straight lines perpendicular to the midsurface before deformation, remain straight after deformation, and
5) the transverse normals to the midsurface before deformation remain straight but not necessarily normal to the midsurface after deformation.

The last assumption results in a constant state of transverse shear strains through the thickness and zero transverse normal strains. The most significant difference between the classical and shear deformation theories is the effect of including transverse shear deformation on the predicted deflections.

Computer Program

The theoretical development of the finite element method formulation of the elastic shear deformable plate theory and the Boussinesq foundation model were implemented into a linear finite element computer code named RSLAB^N (Bulut 2001). The finite element method computer code can be used to analyze ribbed slabs or slabs of constant thickness on expansive as well as compressible soils. The program employs the small-displacement theory and can consider orthotropic plate behavior so that two different Young's modulus values can be assigned to the reinforced concrete slab in two perpendicular directions. The foundation soil is modeled using the Boussinesq elastic continuum formulation. The Boussinesq equation for surface deflection is used for determining the stiffness matrix of the foundation soil. This is different from the more commonly known Winkler (or spring) model where the springs behave independent of each other while in an elastic continuum model there is interaction of neighboring soil elements. The plate is considered to be an assemblage of rectangular finite elements and the behavior of each element is characterized by a stiffness matrix. The element stiffness matrices are assembled into a total structural stiffness matrix by using the conditions of continuity of displacements and equilibrium of nodal forces. Once the plate model has been assembled, it must be connected, in some way, to the supporting soil foundation. This requires the derivation of foundation stiffness coefficients associated with the nodal points corresponding to those in the plate model (Bulut 2001). The assembled matrices (Eq. 3) are then solved to obtain the nodal displacements.

$$[K]\{\Delta\} = \{f\} \tag{3}$$

where $[K]$ is the overall stiffness matrix, $\{\Delta\}$ is the displacement vector, and $\{f\}$ is the load vector.

The code accepts the vertical differential soil movements, y_m, over the range of edge moisture variation distance, e_m, as input data to represent the distortion modes for expansive soils in calculating the displacements and the stresses within the slab. As it has been mentioned in the previous sections, there are mainly two types of critical foundation soil distortion modes due to the soil swelling and shrinking. These modes of distortion create soil surfaces of mound shapes; edge lift and center lift cases. The vertical movements within the edge moisture variation distances are the differential movements the soil would have in the absence of the weight of the slab which, because of its flexural rigidity, will suppress the higher spots of the differential movement (Fig. 5). The problem of soil-slab interaction is solved by superimposing flexible slab on the unloaded differential soil profile.

Gaps occur between the slab and the soil at some points when the slab interacts with these mound shapes. The program has an iterative scheme for checking the contact points between the slab and the pre-deformed mound shapes for the center and edge lift conditions. When gaps are developed at some points, the stiffness coefficients of the soil at those locations are set to zero. The program goes through a number of iterations and checks for contact points between two successive iterations. If the number of contact points between the previous and the current iteration are the same, then the program has converged. It usually takes several iterations to converge to the real solution. These types of problems are considered as non-linear in the geotechnical engineering discipline due to the partial contact conditions and the iteration schemes involved.

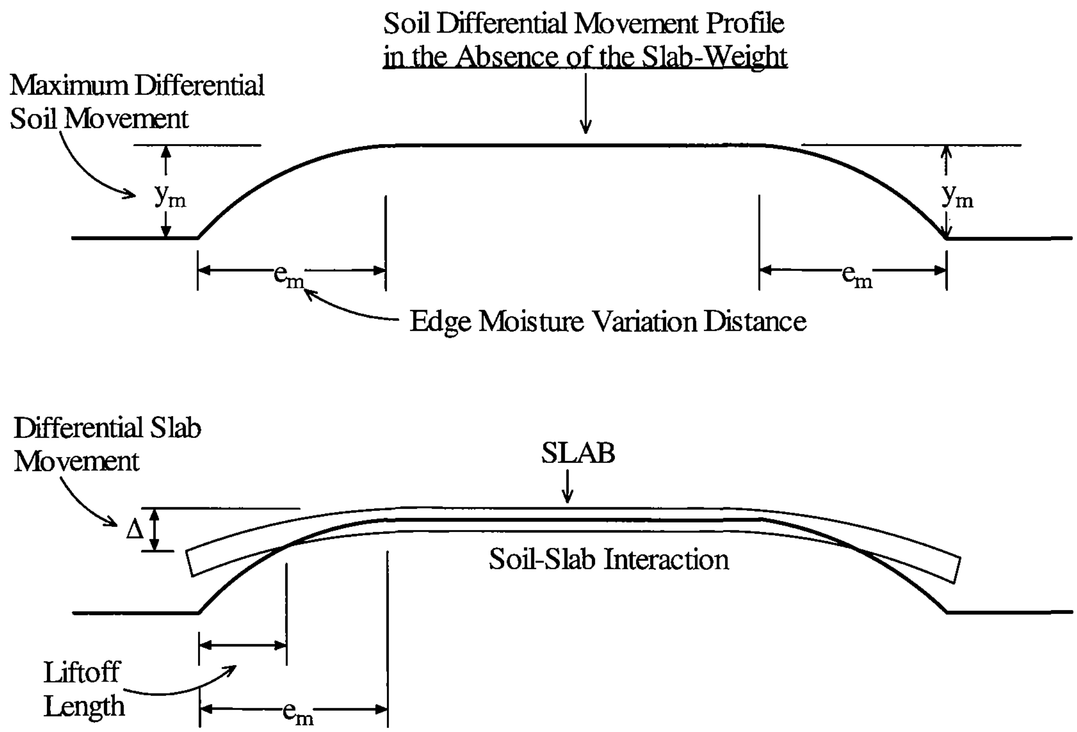


Fig. 5. Soil-Slab Interaction Resulting in Differential Slab Movement.

Verification of Computer Program

The linear elastic analysis verification of the program mainly consists of solving an example problem contained in the PTI slab manual and then comparing the results with the PTI results. Details of the example problem (i.e., example one: a residential slab on expansive soil constructed in a dry climate) can be found in the PTI slab manual. This slab is constructed in a dry climate, where the Thorntwaite Moisture Index is –16, in which the center lift condition generally controls the flexural design (Lytton and Meyer 1971). However, the slab is being analyzed for both center and edge lift conditions. The slab is discretized into 246 rectangular finite elements with 282 nodal points. The slab plan geometry is depicted in Fig. 6.

Center Lift Analysis

The residential slab example is analyzed with the case of stiffening beams, as the beam locations are shown in Fig. 6, and with the case of constant thickness. The constant thickness slab is obtained by converting the ribbed slab into an equivalent thickness slab that has the same cross-sectional moment of inertia as with the stiffening beam slab. These two analyses help to explain the distribution of the stresses within a constant thickness slab and as well as a slab with the cross stiffening beams both in *x*- and *y*-directions. The comparison of the moments in *x*-

direction, twisting moments, and shears in the x-direction are depicted in Fig. 7, Fig. 8, and Fig. 9, respectively. The complete set of plots for the displacements, moments, and shear forces for both ribbed and constant thickness slabs for this example can be found in Bulut (2001).

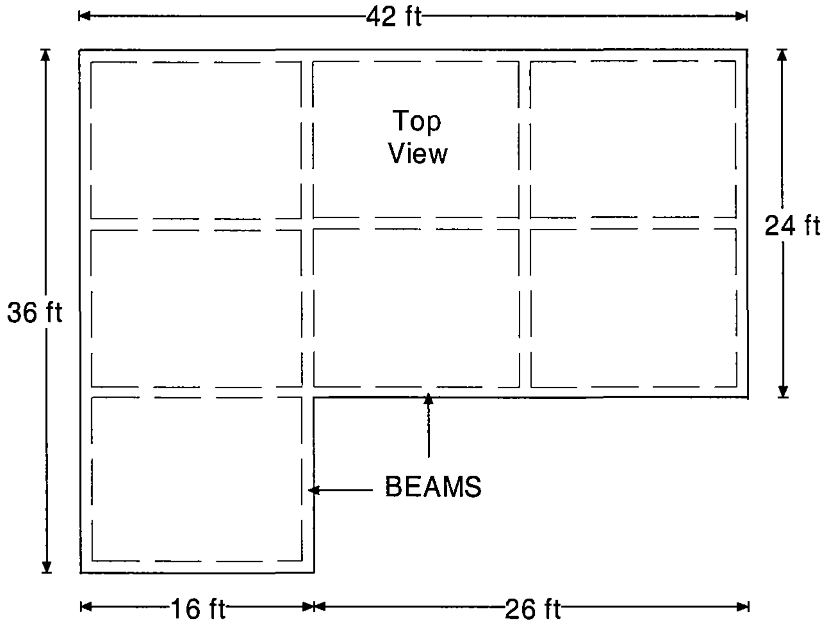
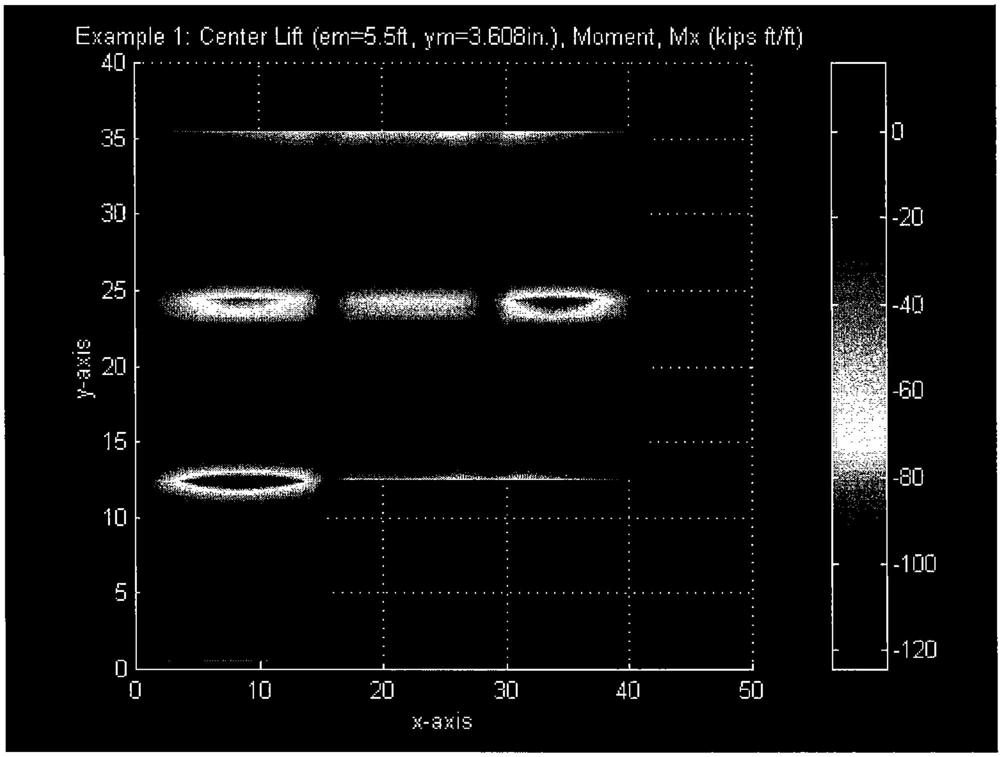


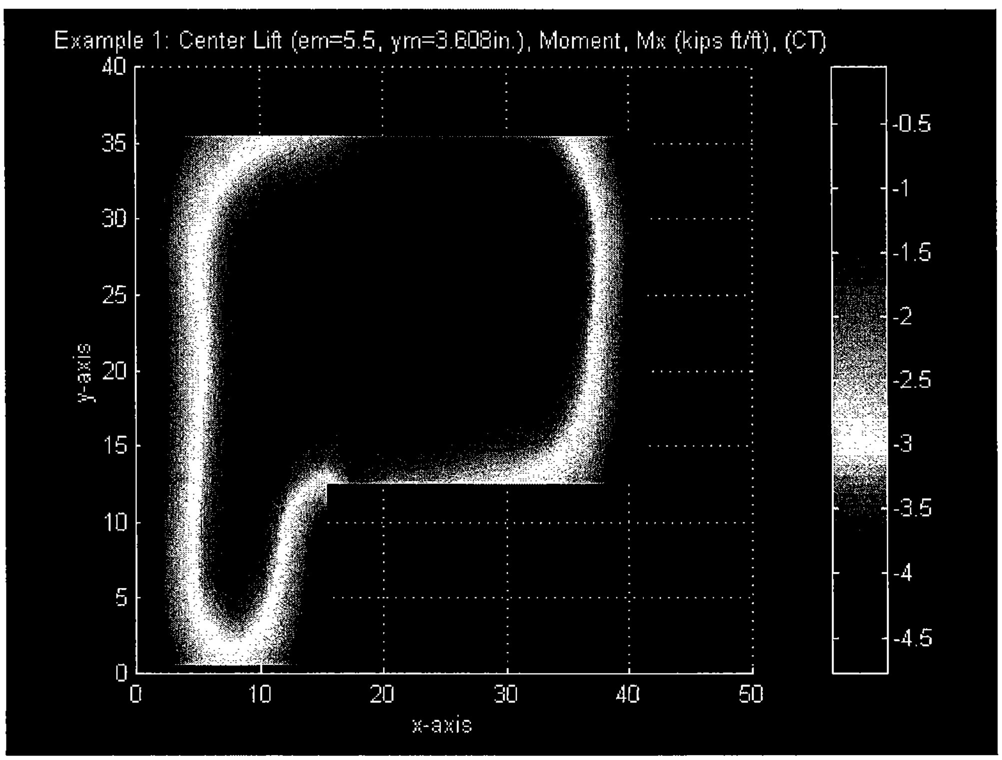
Fig. 6. Slab Geometry.

The comparison of results from the program and the PTI manual for the example problem are depicted in Table 1. As it is seen from Table 1, values for the maximum average moments and shear forces from the ribbed slab analysis are comparable with the PTI results. However, the results from the constant thickness analysis are lower than the ones from both the ribbed and PTI slab analysis. The differential deflection values obtained from the program for both the ribbed and constant thickness slabs are higher than the differential deflections from the PTI example. However, the program results in more conservative the Δ/L ratios. The Δ/L ratios for the RSLAB^{N} analyses were found by determining the largest departure, Δ, from the straight line joining the high point to the low point. The length of the line was taken as L.

The maximum average moment quantities shown in Table 1 for the ribbed slab are determined by adding the products of the moments in each plate element and dividing the sum by the width of the slab. The same process is followed to determine the maximum average shear forces. The RSLAB^{N} finite element computer program will enable to demonstrate the soil-structure interaction behavior of the whole slab, which is simply not possible with the PTI method in which the overlapping method of rectangular slabs misses to show the critical stress points within the slab. As it is seen from Fig. 7, with the capability of handling the ribbed slab analysis, the program can calculate the moment concentrations within the beams and can predict their locations. Structural engineers now will be able to design the beams for these high values of moments developed within the beams. Twisting moments can also be calculated with the program. PTI method does not calculate these moments. Figure 8 shows that twisting moments can reach very high values, which need special attention for design purposes.

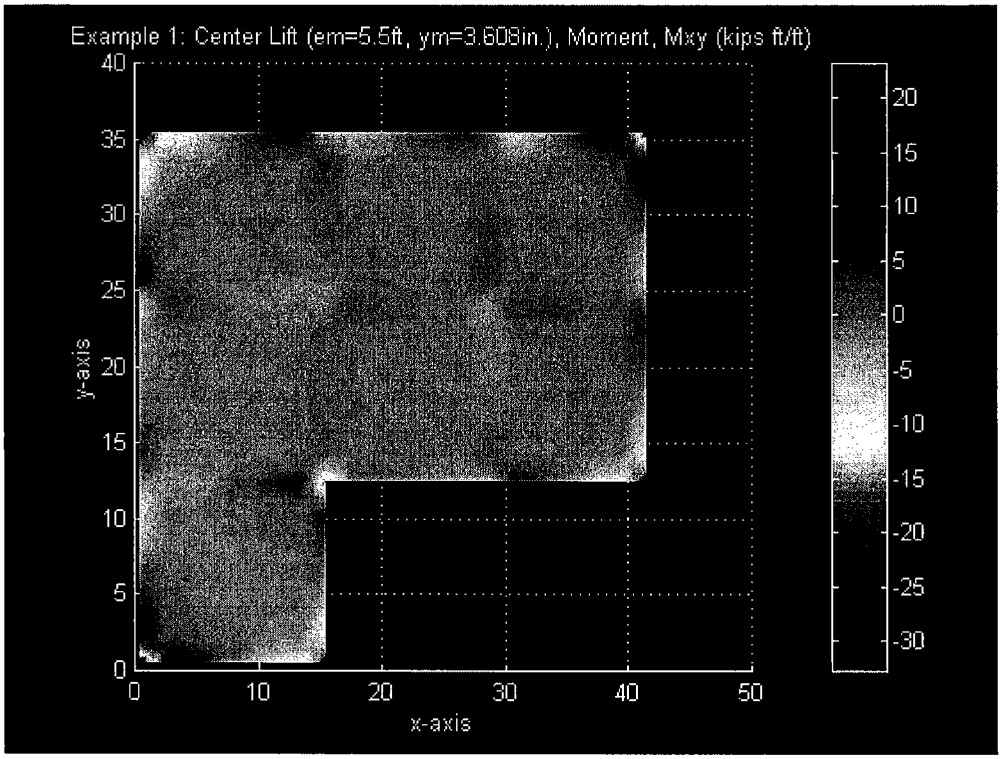


Ribbed Slab



Flat Slab

Fig. 7. Center Lift Analysis, Moment in x-Direction.



Ribbed Slab

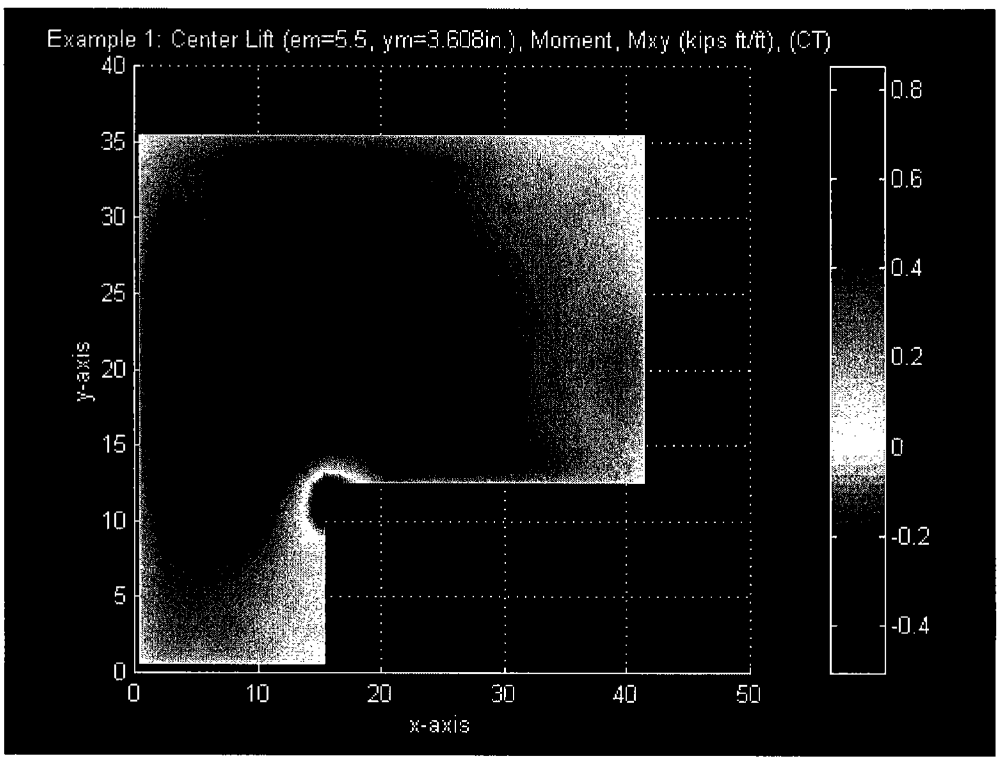


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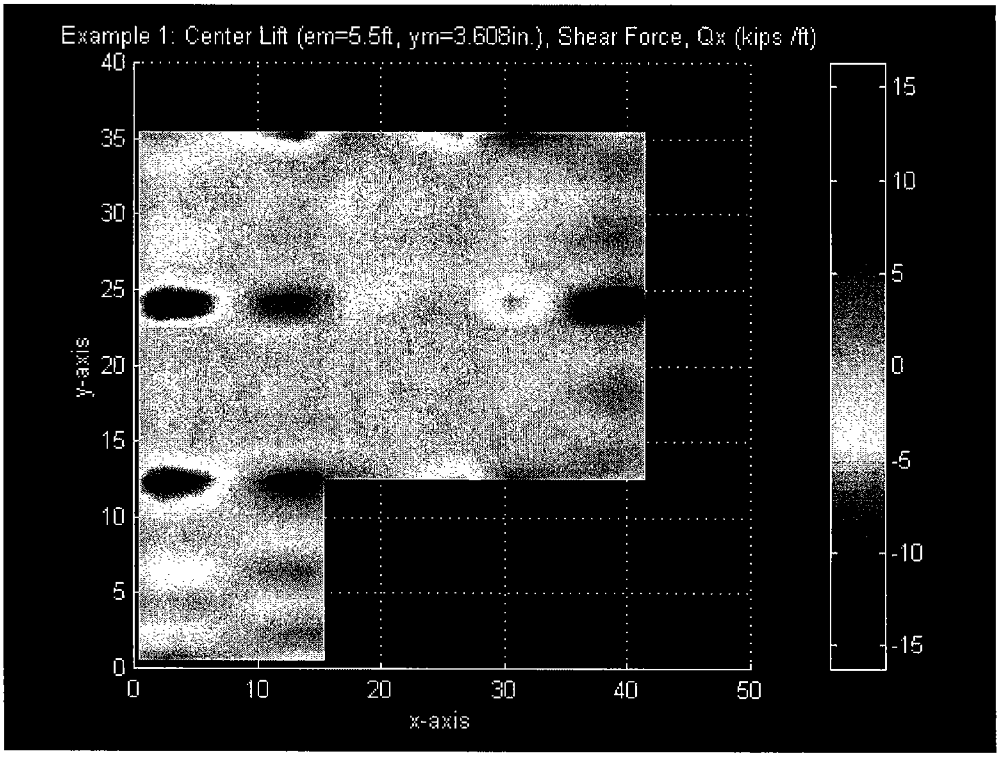
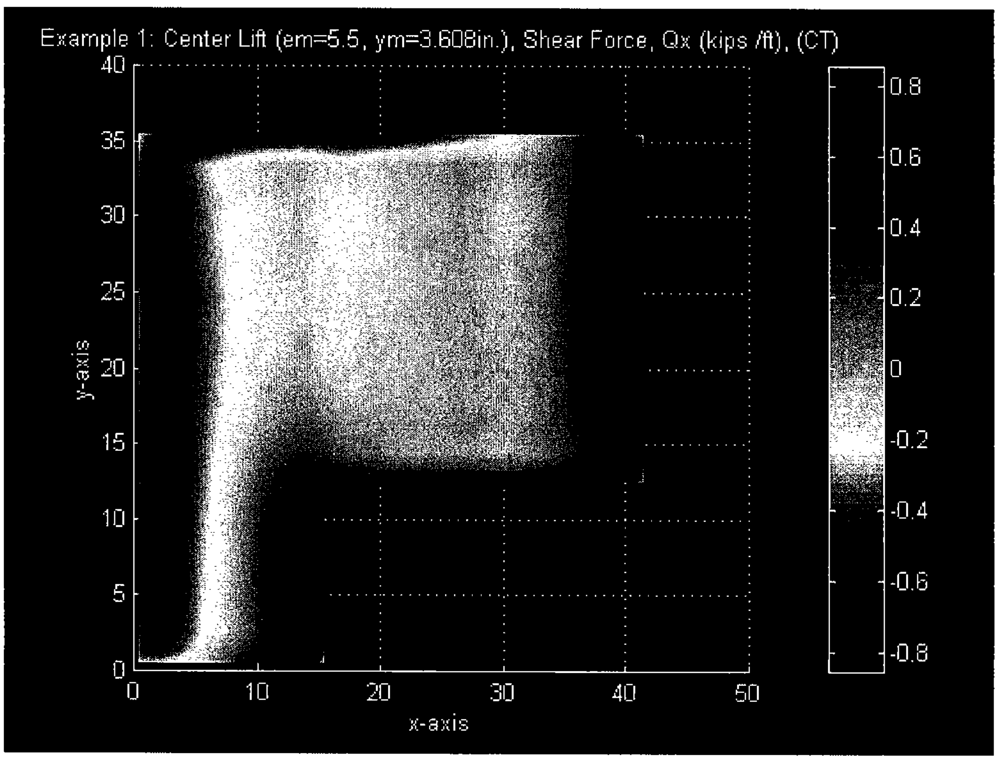
Fig. 8. Center Lift Analysis, Twisting Moment.



Ribbed Slab



Flat Slab

Fig. 9. Center Lift Analysis, Shear in x-Direction.

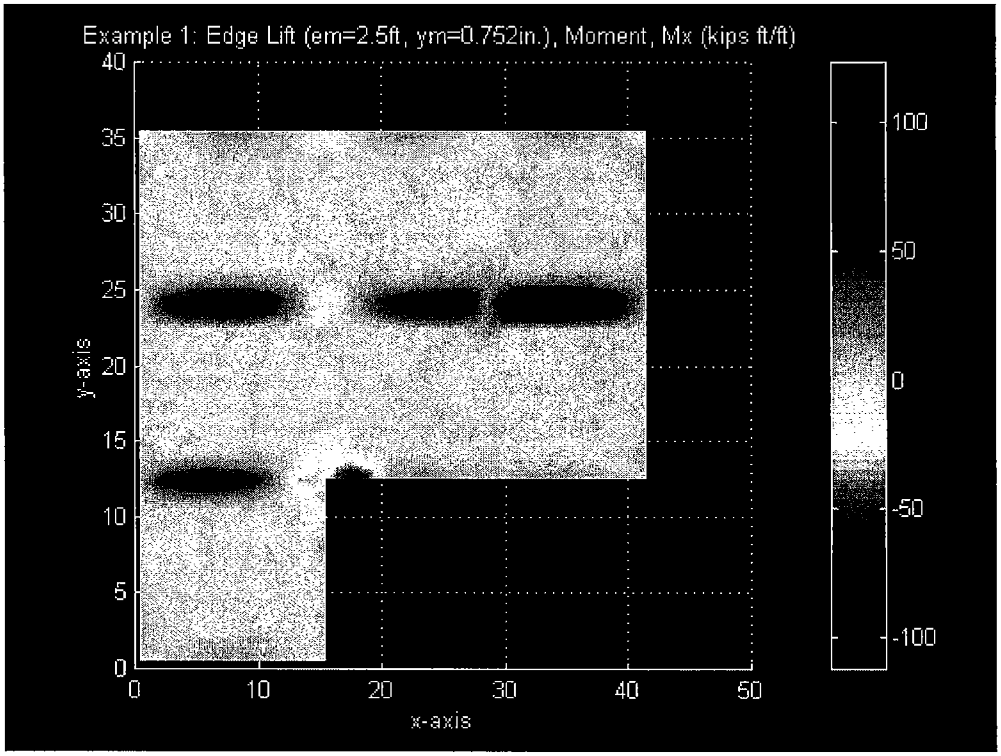
Table 1. Comparison of Stresses and Deflections, Center Lift Case.

	Constant Thickness Slab ($RSLAB^N$ Analysis)	Ribbed Slab ($RSLAB^N$ Analysis)	Example No. 1 PTI Manual (Design)	
Moment, M_x (kips ft/ft)	4.79	11.52	11.509	
Moment, M_y (kips ft/ft)	5.17	9.83	12.18	
Shear Force, Q_x (kips/ft)	0.93	1.85	2.105	
Shear Force, Q_y (kips/ft)	1.03	1.74	1.965	
Differential Deflection, δ (in.) (Δ/L)	1.01 (1/2008)	2.43 (1/701)	*x*-direct. 0.72 (1/400)	*y*-direct. 0.757 (1/665)

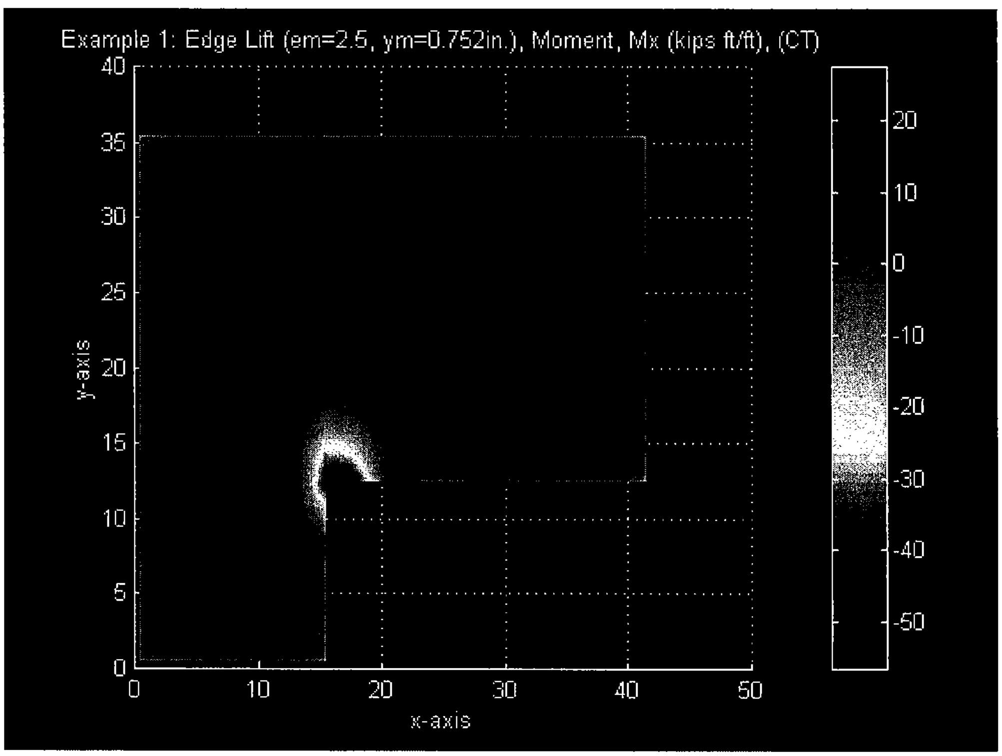
Edge Lift Analysis

The residential slab example is analyzed with the case of stiffening beams and with the case of constant thickness (Fig. 6). The constant thickness slab is obtained by converting the ribbed slab into an equivalent thickness slab that has the same cross-sectional moment of inertia as with the stiffening beam slab. The comparison of the moments in x-direction, twisting moments, and shears in the x-direction are depicted in Fig. 10, Fig. 11, and Fig. 12, respectively. The comparison of results from the program and the PTI manual for the example problem are presented in Table 2 for the edge lift analysis case. As it is seen from Table 2, values for the moments from the constant thickness slab analysis are comparable with the PTI results. However, the results from the ribbed slab analysis are much higher than the ones from both the constant thickness and PTI slab analysis. The program results in higher shear force values for both constant thickness and ribbed slab analysis than the PTI values. The differential deflection values obtained from the program for both the ribbed and constant thickness slabs are higher than the differential deflections from the PTI example. However, the program results in conservative Δ/L ratios.

These conclusions were made from analyzing an example problem from the PTI manual; therefore, it is very difficult to generalize these conclusions for all slab types and different input variables. Using the edge moisture variation distance, e_m, and differential soil movement, y_m, the structural engineer can analyze the slab for displacements, moments, and shear forces and in turn use these results for design of the slab. As explained in the above sections, the geotechnical engineer can provide the two important parameters (i.e., e_m and y_m) employing the new VOLFLO Win 1.0 program which is based on unsaturated soil mechanics principles and employs basic laboratory test results.

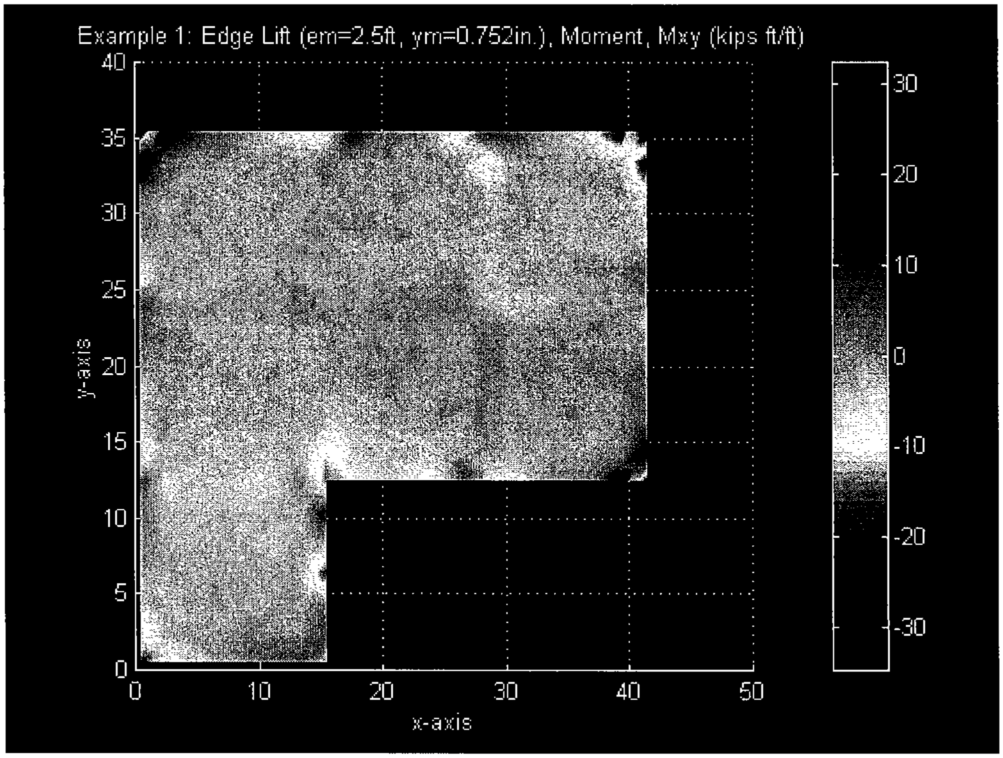


Ribbed Slab

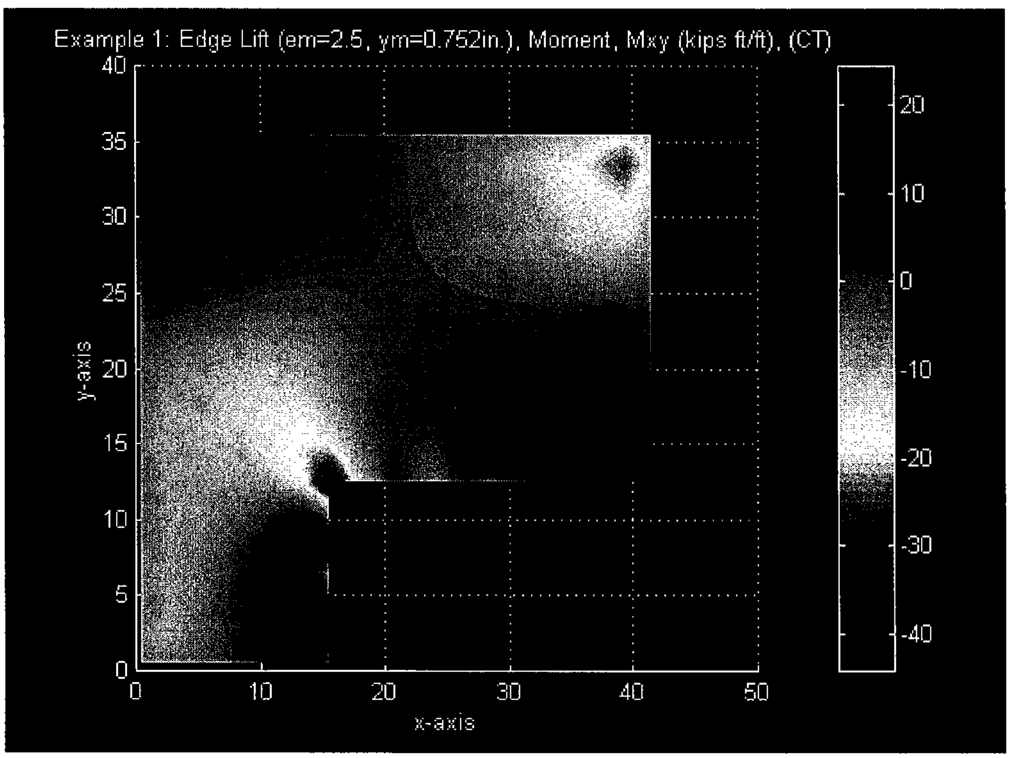


Flat Slab

Fig. 10. Edge Lift Analysis, Moment in x-Direction.

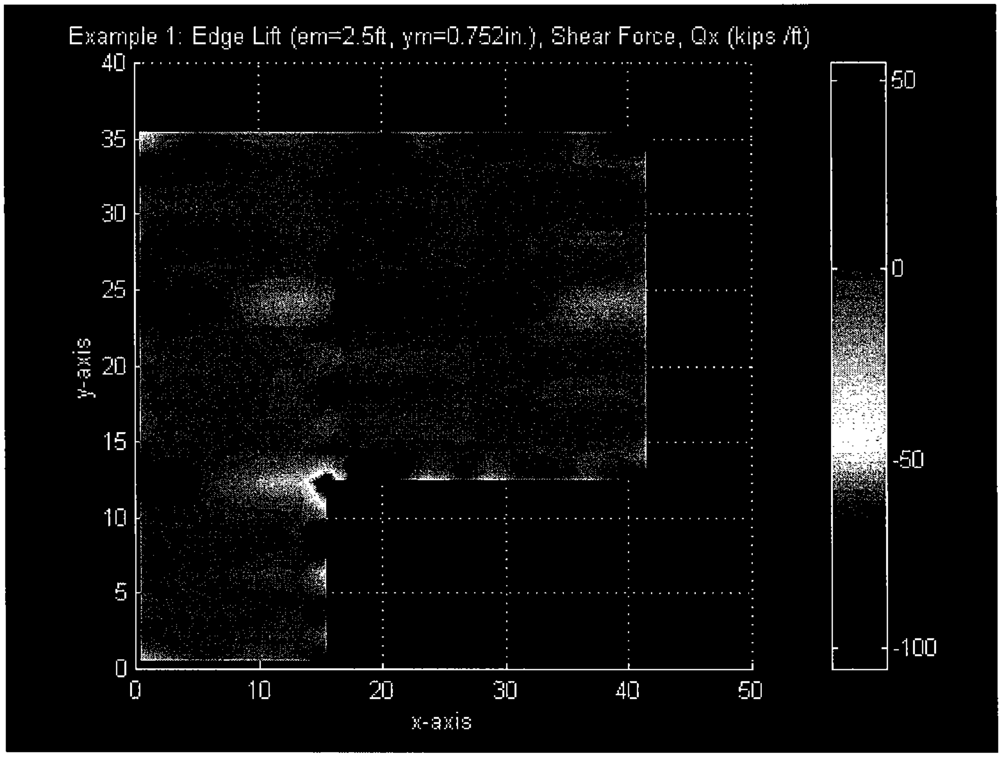


Ribbed Slab

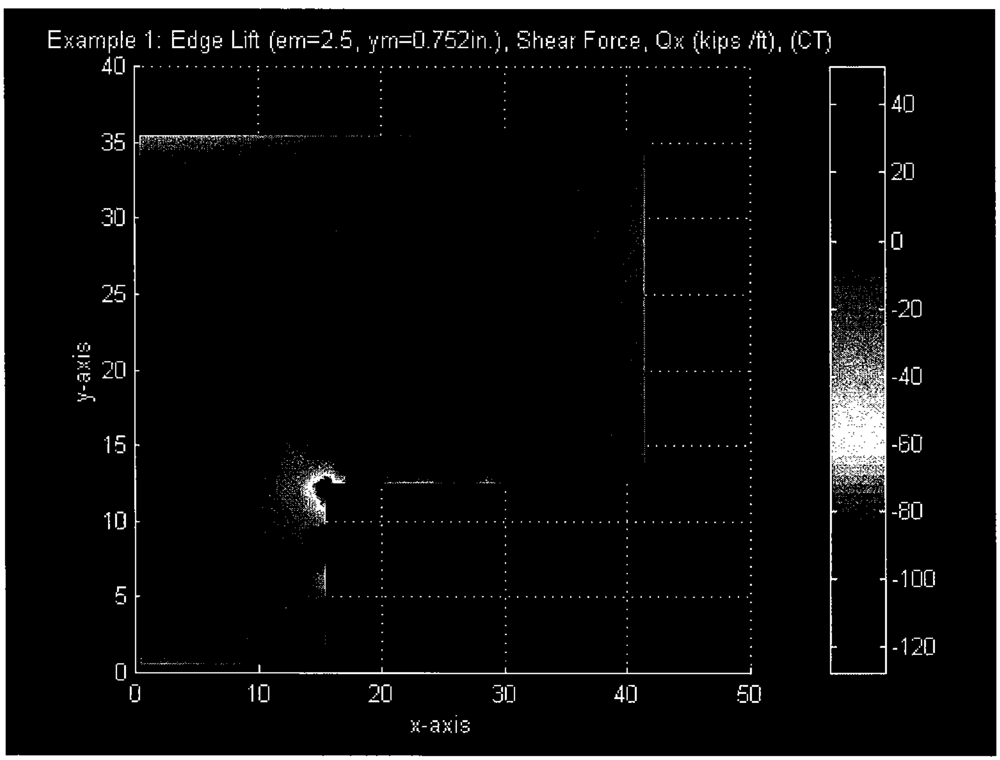


Flat Slab

Fig. 11. Edge Lift Analysis, Twisting Moment.



Ribbed Slab



Flat Slab

Fig. 12. Edge Lift Analysis, Shear in x-Direction.

Table 2. Comparison of Stresses and Deflections, Edge Lift Case.

	Constant Thickness Slab ($RSLAB^N$ Analysis)	Ribbed Slab ($RSLAB^N$ Analysis)	Example No. 1 PTI Manual (Design)	
Moment, M_x (kips ft/ft)	2.38	8.73	2.66	
Moment, M_y (kips ft/ft)	2.29	11.10	3.01	
Shear Force, Q_x (kips/ft)	5.71	4.66	1.752	
Shear Force, Q_y (kips/ft)	3.10	4.37	1.681	
Differential			*x*-direct.	*y*-direct.
Deflection, δ (in.)	1.09	1.46	0.231	0.219
(Δ/L)	(1/1366)	(1/1723)	(1/2182)	(1/1315)

Discussion

An analytical study was undertaken in this study to develop an improved analysis method for calculating the performance of slabs on expansive soils. A Finite element method formulation of slabs on elastic continuum foundations was developed to analyze this complex soil-structure system. To more correctly model the soil-structure interaction, the $RSLAB^N$ program can accommodate any practical geometric shapes composed of rectangular finite elements, stiffening beams, and variable loading conditions. In addition, the calculation of twisting moments is possible with this program. The program can also model the anisotropic properties of the reinforced concrete slab in two perpendicular directions, mainly *x*- and *y*-directions. With the program it is now possible to examine the overall behavior of the slab and to locate the stress concentrations for the purpose of design. This was not entirely possible with the overlapping process of the PTI method, which was missing the stress concentration values and their locations. The analysis emphasizes that the reentrant corners are the critical locations for stress concentrations. It is also seen that the stiffening beams are carrying most of the stresses.

The $RSLAB^N$ program was compared with the example problems in the PTI manual. The analysis was done for a flat slab and a ribbed slab having the same cross sectional moment of inertia and the results were compared with the results in the PTI manual. Based on the conclusions from investigating a single example problem from the PTI manual, the analysis shows that the PTI method is conservative for the center lift case, but is not conservative for the edge lift case. The $RSLAB^N$ finite element computer program can effectively be employed for analyses of slabs constructed on expansive soils. This program can calculate displacements, moments, shear forces based on a realistic soil-structure interaction model.

Acknowledgement

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SOIL SUCTION MEASUREMENTS BY FILTER PAPER

Rifat Bulut, M.ASCE, Robert L. Lytton, F.ASCE, and Warren K. Wray, F.ASCE

Geotechnical Special Publication Number 115

Expansive Clay Soils and Vegetative Influence on Shallow Foundations

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Soil Suction Measurements by Filter Paper

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Abstract

This paper reports on an evaluation of wetting and drying filter paper suction calibration and soil total and matric suction measurement techniques of filter paper method. Calibration of the method was investigated by constructing two calibration curves; one by using the process of wetting the filter papers through vapor flow and the other by using the method of drying the filter papers through fluid flow. The wetting curve was constructed using sodium chloride (NaCl) salt solutions and Schleicher & Schuell No. 589-WH filter papers. It was found that the change in the wetting suction curve is very sensitive to minor changes in filter paper water content below about 1.5 log kPa (2.5 pF) suction. The drying curve was established by employing both pressure plate and pressure membrane devices and the same filter papers. In developing the filter paper calibration curves, the capabilities, pitfalls, and limitations of the method are also discussed.

Introduction

The filter paper method is a soil suction measurement technique. Soil suction is one of the most important parameters describing the moisture condition of unsaturated soils. The measurement of soil suction is crucial for applying the theories of the engineering behavior of unsaturated soils. The filter paper method is an inexpensive and relatively simple laboratory test method, from which both total and matric

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suction measurements are possible. With a reliable soil suction measurement technique, the initial and final soil suction profiles can be obtained from samples taken at convenient depth intervals. The change in suction with seasonal moisture movement is valuable information for many engineering applications.

This paper evaluates calibration techniques for filter paper wetting and drying processes, and soil total and matric suction measurements with filter paper method by construction of two calibration curves. The wetting curve was constructed using NaCl salt solutions and Schleicher & Schuell No. 589-WH filter papers. Salt solutions and filter papers were brought to equilibrium through vapor flow (filter paper wetting process) at isothermal conditions. Equilibrium time and temperature were two weeks and 25^{o}C, respectively. The temperature was maintained at 25^{o}C within ± 0.1^{o}C fluctuations. The drying curve was established using both pressure plate and pressure membrane devices and the same filter papers. The pressure plate apparatus can measure matric suction values up to 150 kPa. However, with the pressure membrane device matric suction values can be extended up to 10,000 kPa. The equilibration periods were selected as 3, 5, and 7 days depending on the testing set up, which will be described below.

A Brief Historical Background

There are many soil suction measurement techniques and instruments in the fields of soil science and engineering. Most of these instruments have limitations with regard to range of measurement, equilibration times, and cost. Therefore, there is a need for a method which can cover the practical suction range, be adopted as a basis for routine testing, and is inexpensive. One of those soil suction measurement techniques is the filter paper method, which was evolved in Europe in the 1920s and came to the United States in 1937 with Gardner (1937). Since then, the filter paper method has been used and investigated by numerous researchers (Fawcett and Collis-George 1967; McQueen and Miller 1968; Al-Khafaf and Hanks 1974; McKeen 1980; Hamblin 1981; Chandler and Guierrez 1986; Houston et al. 1994; Swarbrick 1995), who have tackled different aspects of the filter paper method. Different types of materials were used, such as filter papers and suction measuring devices, and different experimental techniques to calibrate the filter paper and to measure suction of the soil sample. Therefore, it is very difficult to compare these methods on a one-to-one basis.

All the calibration curves established from Gardner (1937) to Swarbrick (1995) appear to have been constructed as a single curve by using different filter papers, a combination of different soil suction measuring devices, and different calibrating testing procedures. However, Houston et al. (1994) developed two different calibration curves; one for total suction and one for matric suction measurements using Fisher quantitative coarse filter papers. For the total suction calibration curve, saturated salt solutions and for the matric suction calibration curve tensiometers and pressure membranes were employed. Houston et al. (1994) reported that the total and matric suction calibration curves were not compatible. This simply implies that two different calibration curves, one for matric and one for total suction, need to be used in soil suction measurements. However, in this paper the fact is presented that the two curves reflect an expected hysteresis between wetting and drying effects and that the appropriate curve for both matric and total suction is the wetting curve since this matches the process that the filter paper undergoes in the measurement process.

Soil Suction Concept

In general, porous materials have a fundamental ability to attract and retain water. The existence of this fundamental property in soils is described in engineering terms as suction, negative stress in the pore water. In engineering practice, soil suction is composed of two components: matric and osmotic suction (Fredlund and Rahardjo 1993). The sum of matric and osmotic suction is called total suction. Matric suction comes from the capillarity, texture, and surface adsorptive forces of the soil. Osmotic suction arises from the dissolved salts contained in the soil water. This relationship can be formed in an equation as follows:

$$h_t = h_m + h_\pi \tag{1}$$

where h_t = total suction (kPa), h_m = matric suction (kPa), and h_π = osmotic suction (kPa).

Total suction can be calculated using Kelvin's equation, which is derived from the ideal gas law using the principles of thermodynamics and is given as:

$$h_t = \frac{RT}{V}\ln\left(\frac{P}{P_o}\right) \tag{2}$$

where h_t = total suction, R = universal gas constant, T = absolute temperature, V = molecular volume of water, P / P_o = relative humidity, P = partial pressure of pore water vapor, and P_o = saturation pressure of water vapor over a flat surface of pure water at the same temperature.

If Eq. (2) is evaluated at a reference temperature of 25^{o}C, the following total suction and relative humidity relationship can be obtained:

$$h_t = 137182 \times \ln\left(P/P_o\right) \tag{3}$$

Figure 1 shows a plot of Eq. (3) at 25^{o}C temperature. From Fig. 1, it can be seen that there is nearly a linear relationship between total suction (h_t) and relative humidity (P/P_o) over a very small relative humidity range. It can be said, in general, that in a closed system under isothermal conditions the relative humidity may be associated with the water content of the system such as 100 percent relative humidity refers to a fully saturated condition. Therefore, the suction value of a soil sample can be inferred from the relative humidity and suction relationship if the relative humidity is evaluated in some way. In a closed system, if the water is pure enough, the partial pressure of the water vapor at equilibrium is equal to the saturated vapor pressure at temperature, T. However, the partial pressure of the water vapor over a partly saturated soil will be less than the saturation vapor pressure of pure water due to the soil matrix structure and the free ions and salts contained in the soil water (Fredlund and Rahardjo 1993).

In engineering practice, soil suction has usually been calculated in *pF* units (Schofield 1935) (i.e., suction in $pF = log_{10}(|suction\ in\ cm\ of\ water|)$). However, soil suction is also currently being represented in *log kPa* unit system (Fredlund and Rahardjo 1993) (i.e., suction in $log\ kPa = log_{10}(|suction\ in\ kPa|)$). The relationship between these two systems of units is approximately *suction in log kPa = suction in pF – 1*.

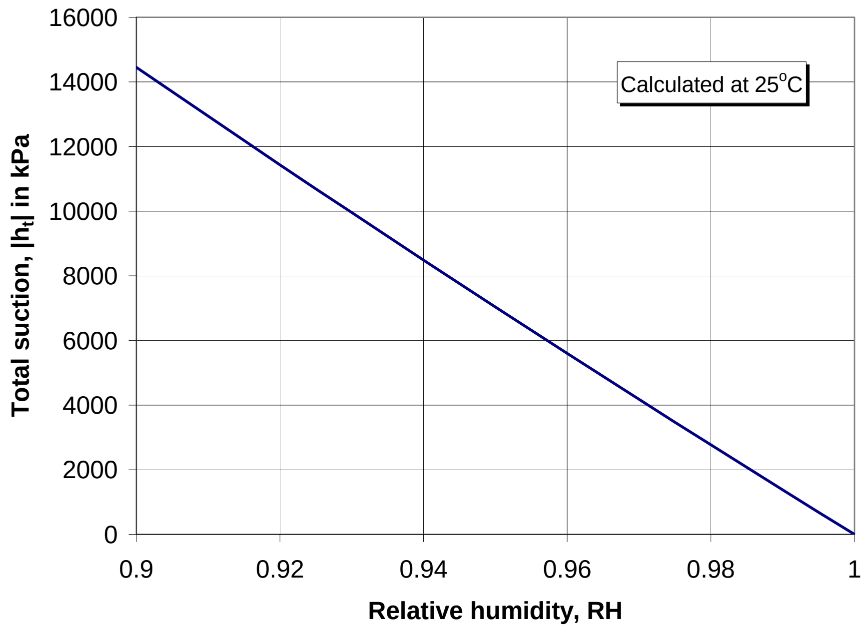


Figure 1. Total Suction versus Relative Humidity.

If total suction in kPa from Fig. 1 is converted to log kPa units, Fig. 2 is obtained. The difference between Fig. 1 and Fig. 2 is only the suction unit. The suction unit in Fig. 1 is kPa whereas it is log kPa in Fig. 2. From Fig. 2 it can clearly be seen that when relative humidity approaches 100 percent, the total suction becomes very sensitive. The sensitivity in the suction is due to the common logarithm used to convert suction from kPa to the log kPa unit.

Matric suction can be calculated from pressure plate and pressure membrane devices as the difference between the applied air pressure and water pressure across a porous plate. Matric suction can be formed in a relationship as follows:

$$h_m = -(u_a - u_w) \tag{4}$$

where h_m = matric suction, u_a = applied air pressure, and u_w = free water pressure at atmospheric condition.

The osmotic suction of electrolyte solutions, that are usually employed in the calibration of filter papers and psychrometers, can be calculated using the relationship between osmotic coefficients and osmotic suction. Osmotic coefficients are readily available in the literature for many different salt solutions. Table 1 gives the osmotic coefficients for several salt solutions. Osmotic coefficients can also be obtained from the following relationship (Lang 1967):

$$\phi = -\frac{\rho_w}{vmw}\ln\left(\frac{P}{P_o}\right) \tag{5}$$

where ϕ = osmotic coefficient, v = number of ions from one molecule of salt (i.e., v = 2 for NaCl, KCl, NH_4Cl and v = 3 for Na_2SO_4, $CaCl_2$, $Na_2S_2O_3$, etc.), m = molality, w = molecular mass of water, and ρ_w = density of water.

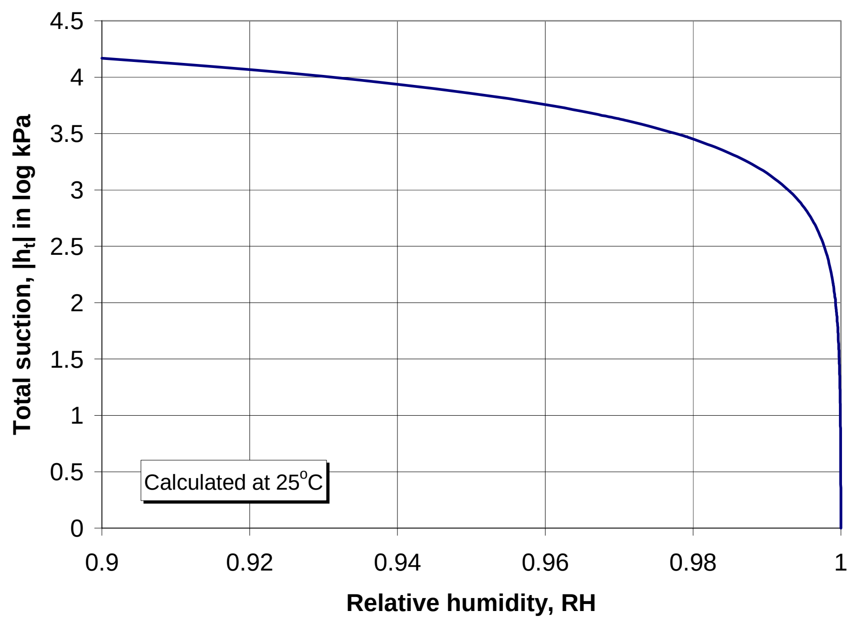


Figure 2. Total Suction and Relative Humidity Relationship.

The relative humidity term (P/P_o) in Eq. (5) is also known as the activity of water (a_w) in physical chemistry of electrolyte solutions. The combination of Eq. (2) and Eq. (5) gives a useful relationship that can be adopted to calculate osmotic suctions for different salt solutions:

$$h_\pi = -vRTm\phi \tag{6}$$

Table 2 gives osmotic suctions for several salt solutions using osmotic coefficients from Table 1 and Eq. (6).

Table 1. Osmotic Coefficients of Several Salt Solutions.

	Osmotic Coefficients at 25°C						
Molality (m)	$NaCl$[a]	KCl[a]	NH_4Cl[a]	Na_2SO_4[b]	$CaCl_2$[c]	$Na_2S_2O_3$[b]	$MgCl_2$[c]
0.001	0.9880	0.9880	0.9880	0.9608	0.9623	0.9613	0.9627
0.002	0.9840	0.9840	0.9840	0.9466	0.9493	0.9475	0.9501
0.005	0.9760	0.9760	0.9760	0.9212	0.9274	0.9231	0.9292
0.010	0.9680	0.9670	0.9670	0.8965	0.9076	0.8999	0.9106
0.020	0.9590	0.9570	0.9570	0.8672	0.8866	0.8729	0.8916
0.050	0.9440	0.9400	0.9410	0.8229	0.8619	0.8333	0.8708
0.100	0.9330	0.9270	0.9270	0.7869	0.8516	0.8025	0.8648
0.200	0.9240	0.9130	0.9130	0.7494	0.8568	0.7719	0.8760
0.300	0.9210	0.9060	0.9060	0.7262	0.8721	0.7540	0.8963
0.400	0.9200	0.9020	0.9020	0.7088	0.8915	0.7415	0.9206
0.500	0.9210	0.9000	0.9000	0.6945	0.9134	0.7320	0.9475
0.600	0.9230	0.8990	0.8980	0.6824	0.9370	0.7247	0.9765
0.700	0.9260	0.8980	0.8970	0.6720	0.9621	0.7192	1.0073
0.800	0.9290	0.8980	0.8970	0.6629	0.9884	0.7151	1.0398
0.900	0.9320	0.8980	0.8970	0.6550	1.0159	0.7123	1.0738
1.000	0.9360	0.8980	0.8970	0.6481	1.0444	0.7107	1.1092
1.200	0.9440	0.9000	0.8980	…	…	…	…
1.400	0.9530	0.9020	0.9000	…	…	…	…
1.500	…	…	…	0.6273	1.2004	0.7166	1.3047
1.600	0.9620	0.9050	0.9020	…	…	…	…
1.800	0.9730	0.9080	0.9050	…	…	…	…
2.000	0.9840	0.9120	0.9080	0.6257	1.3754	0.7410	1.5250
2.500	1.0130	0.9230	0.9170	0.6401	1.5660	0.7793	1.7629

References:
[a]Hamer and Wu, 1972
[b]Goldberg, 1981
[c]Goldberg and Nuttall, 1978

The Filter Paper Method

The filter paper method has long been used in soil science and engineering practice and it has recently been accepted as an adaptable test method for soil suction measurements because of its advantages over other suction measurement devices. Basically, the filter paper comes to equilibrium with the soil either through vapor (total suction measurement) or liquid (matric suction measurement) flow. At equilibrium, the suction value of the filter paper and the soil will be equal. After equilibrium is established between the filter paper and the soil, the water content of the filter paper disc is measured. Then, by using a filter paper water content versus suction calibration curve, the corresponding suction value is found from the curve.

This is the basic approach suggested by ASTM Standard Test Method for Measurement of Soil Potential (Suction) Using Filter Paper (ASTM D 5298). In other words, ASTM D 5298 employs a single calibration curve that has been used to infer both total and matric suction measurements. The ASTM D 5298 calibration curve is a combination of both wetting and drying curves. However, this paper demonstrates that the "wetting" and "drying" suction calibration curves do not match, an observation that was also made by Houston et al. (1994).

Table 2. Osmotic Suctions of Several Salt Solutions.

Osmotic Suctions in kPa at 25°C							
Molality (m)	NaCl	KCl	NH_4Cl	Na_2SO_4	$CaCl_2$	$Na_2S_2O_3$	$MgCl_2$
0.001	5	5	5	7	7	7	7
0.002	10	10	10	14	14	14	14
0.005	24	24	24	34	34	34	35
0.010	48	48	48	67	67	67	68
0.020	95	95	95	129	132	130	133
0.050	234	233	233	306	320	310	324
0.100	463	460	460	585	633	597	643
0.200	916	905	905	1115	1274	1148	1303
0.300	1370	1348	1348	1620	1946	1682	2000
0.400	1824	1789	1789	2108	2652	2206	2739
0.500	2283	2231	2231	2582	3396	2722	3523
0.600	2746	2674	2671	3045	4181	3234	4357
0.700	3214	3116	3113	3498	5008	3744	5244
0.800	3685	3562	3558	3944	5880	4254	6186
0.900	4159	4007	4002	4384	6799	4767	7187
1.000	4641	4452	4447	4820	7767	5285	8249
1.200	5616	5354	5343	…	…	…	…
1.400	6615	6261	6247	…	…	…	…
1.500	…	…	…	6998	13391	7994	14554
1.600	7631	7179	7155	…	…	…	…
1.800	8683	8104	8076	…	…	…	…
2.000	9757	9043	9003	9306	20457	11021	22682
2.500	12556	11440	11366	11901	29115	14489	32776

Calibration for the Suction Wetting Curve

The calibration for the suction wetting curve for filter paper using salt solutions is based upon the thermodynamic relationship between total suction (or osmotic suction) and the relative humidity resulting from a specific concentration of a salt in distilled water. The thermodynamic relationship between total suction and relative humidity is given in Eq. (2).

In this study, NaCl was selected as an osmotic suction source for the filter paper calibration. Salt concentrations from 0 (distilled water) to 2.7 molality were prepared and filter papers were simply placed above salt solutions (in a non-contact manner) in sealed containers. The calibration test configuration adopted for this research is shown in Fig. 3. The filter paper and salt solution setups in the sealed containers were put in a constant temperature environment for equilibrium. Temperature fluctuations were kept as low as possible during a two week equilibration period. A water bath was employed for this purpose, in which temperature fluctuations did not exceed ± 0.1°C.

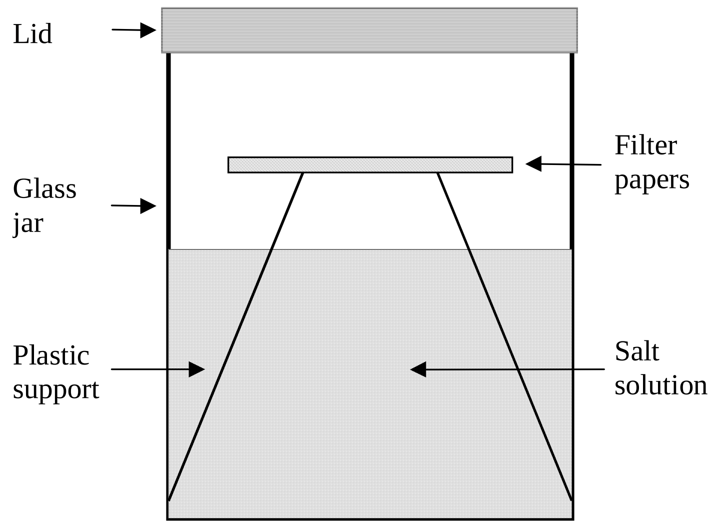


Figure 3. Total Suction Calibration Test Configuration.

Before commencing the filter paper calibration experiments and the soil suction measurements, all the items related to filter paper testing were cleaned carefully. Latex gloves and tweezers were used to handle the materials in nearly all steps of the experiment. The filter papers and aluminum cans for water content measurements were never touched with bare hands because oily hands may cause the filter papers to absorb more water. In addition, it is suggested that the filter paper water content measurements are performed by two persons in order to reduce the time during which the filter papers are exposed to the laboratory atmosphere and, thus, the amount of moisture lost or gained during measurements is kept to a minimum.

Experimental Procedure for Wetting Curve Calibration

The procedure that was adopted for the experiment is as follows:

1. NaCl solutions were prepared from 0 (i.e., distilled water) to 2.7 molality (i.e., the number of moles of NaCl in mass in 1,000 ml of distilled water).
2. A 250 ml glass jar was filled with approximately 150 ml of a solution of known molality of NaCl. Then, a small plastic cup was inserted into the glass jar to function as a support for filter papers. Two filter papers were put on the plastic cup one on top of the other. The glass jar lid was sealed tightly with plastic tapes to ensure air tightness. The configuration of the setup is depicted in Fig. 3.
3. Step 2 was repeated for each different NaCl concentration.

The glass jars were inserted into large plastic containers and the containers were sealed with water proof tape. Then, the containers were put into sealed plastic bags for extra protection. After that, the containers were inserted into the water bath for an equilibration period. After two weeks of equilibrating time, the procedure for the filter paper water content measurements was as follows:

1. Before taking the plastic containers from the water bath, all aluminum cans were weighed to the nearest 0.0001 g. accuracy and recorded on a filter paper water content measurement data sheet, similar to the one provided in ASTM D 5298.
2. After that, all measurements were carried out by two persons. For instance, while one person was opening the sealed glass jar, the other person was transferring the filter paper, using tweezers, into the aluminum can very quickly (i.e., in a few seconds, usually less than 5 seconds). The lid was placed on each aluminum can immediately.
3. Then, the weights of each can with filter papers inside were very quickly measured to the nearest 0.0001 g.
4. Steps 2 and 3 were followed for every glass jar. Then, all the cans were put into the oven with the lids half-open to allow evaporation. All filter papers were kept at $105 \pm 5^{o}C$ temperature for 24 hours inside the oven. This is the standard test method for soil water content measurements. However, it is only necessary to keep the filter paper in the oven for at least 10 hours.
5. Before taking measurements, the cans were closed with their lids and allowed to equilibrate in the oven for about 5 minutes. Then, a can was removed from the oven and put on an aluminum block for about 20 seconds to cool down; the aluminum block acted as a heat sink and expedited the cooling of the can. This is to eliminate temperature fluctuations and air currents in the enclosed weighing scale. After that, the can with dry filter paper inside was weighed to the nearest 0.0001 g. very quickly. The dry filter paper was taken out of the can and the cooled can was also weighed very quickly.
6. Step 5 was repeated for every can.

Wetting Calibration Curve

A wetting curve was constructed from the filter paper test results by following the procedure described above. The curve obtained for Schleicher & Schuell No. 589-WH filter papers using sodium chloride salt solutions is depicted in Fig. 4. Figure 4 clearly shows the sensitivity of total suction to very small changes in filter paper water content values when the relative humidity approaches 100%. The reason behind this sudden drop in suction was briefly explained with Fig. 2 and it will be discussed in detail in the following paragraphs.

There is an inverse relationship between total suction and relative humidity at a constant temperature (i.e., Eq. (2)). Figure 1 was obtained by plotting Kelvin's equation for $25^{o}C$ temperature. From the relationship, total suction is equal to zero when relative humidity is 100 percent (i.e., fully saturated condition). On the other hand, total suction becomes very large when relative humidity decreases, but the change in relative humidity is very small with respect to the change in total suction. For instance, a relative humidity of 94 percent at a temperature of 25°C corresponds

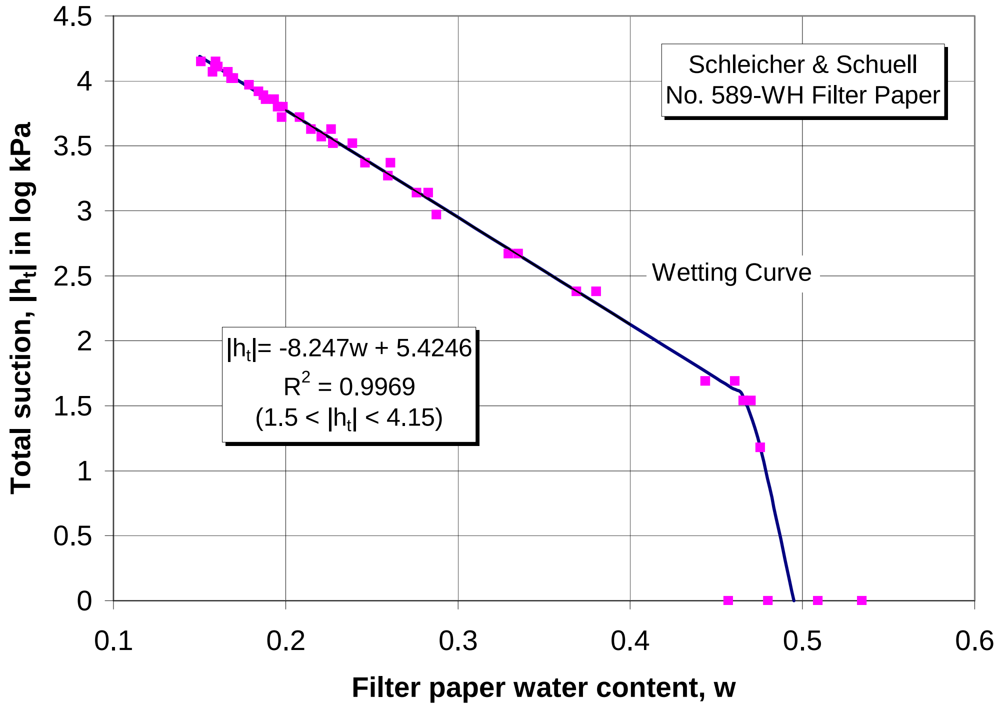


Figure 4. Filter Paper Wetting Calibration Curve.

to a total suction value of 8,488 kPa. Since the total suction values in engineering practice are often represented in logarithmic scales (i.e., pF or log kPa), the total suction values in log kPa units versus relative humidity were plotted in Fig. 2 in order to see the effect of the logarithmic scale on the relationship. From the figure, it is seen that total suction decreases dramatically when relative humidity approaches 100 percent.

Different concentrations of sodium chloride solutions were plotted against corresponding osmotic (or total) suction values both in kPa and log kPa units at 25°C in Figs. 5 and 6, respectively. As expected, the trend of the curves are similar to the trend of the curves obtained for relative humidity versus total suction for both kPa and log kPa units in Figs. 1 and 2, respectively. For example, a high concentration salt solution at a constant temperature in a closed container has low relative humidity above its surface.

Figure 7 depicts a plot of the wetting curve in kPa units versus filter paper water contents obtained in this research. In other words, if the suction values in Fig. 4 are plotted in kPa units, Fig. 7 is obtained. From the figure, the sensitivity of the filter paper water contents and total suction relationship can clearly be seen at very low suction values. From the relationships between total suction and relative humidity (i.e., Figs. 1 and 2), total suction and salt solutions (i.e., Figs. 5 and 6), and total suction and filter paper water contents (i.e., Figs. 4 and 7) it can be concluded that the dramatic decrease in total suction at high water contents depends on the nature of the relationship between total suction and relative humidity from Kelvin's equation and on the use of the logarithmic scale for total suction. In addition, soils

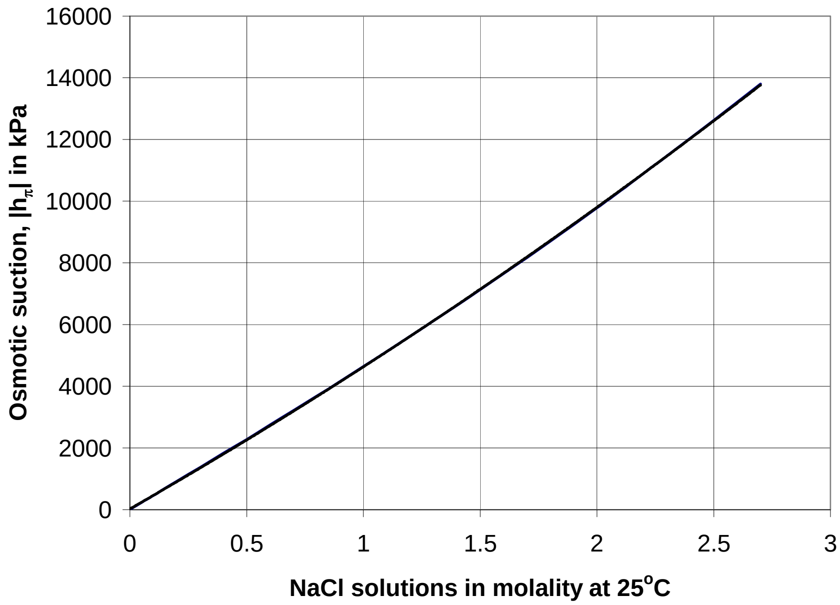


Figure 5. Osmotic Suction versus NaCl Solutions.

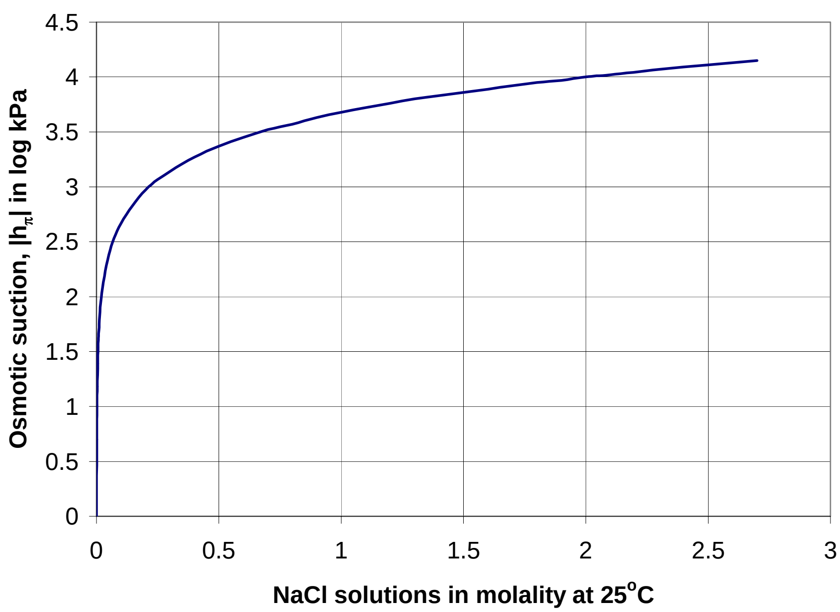


Figure 6. Osmotic Suction versus NaCl Solutions at 25°C.

tend to absorb more water for a small change in suction at very low suction values (Baver et al. 1972), and since filter papers, like soils, are porous materials they are very sensitive for absorbing water at low suction values.

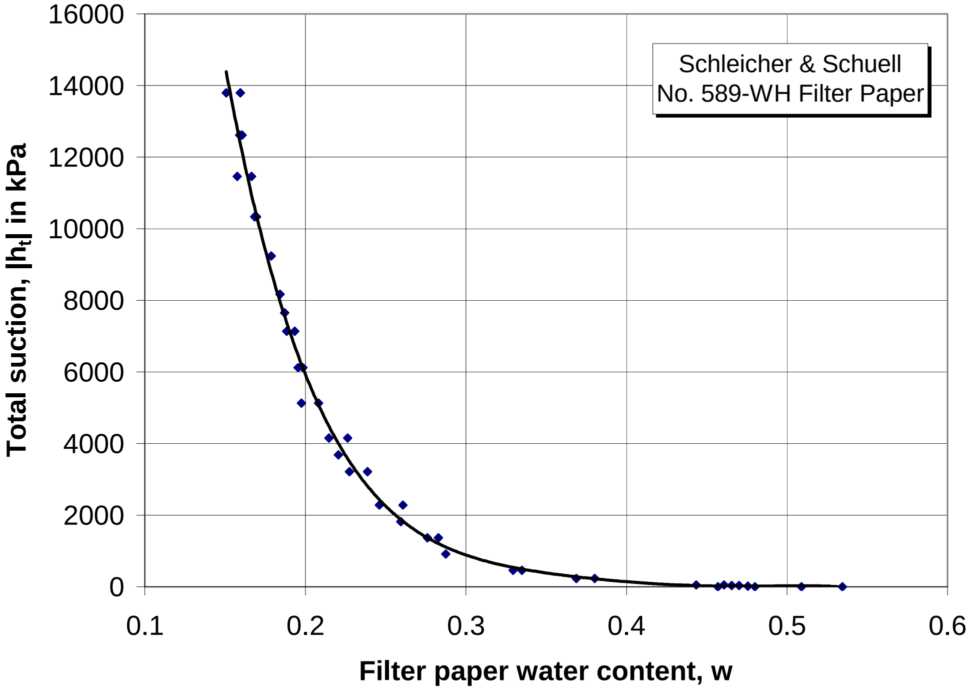
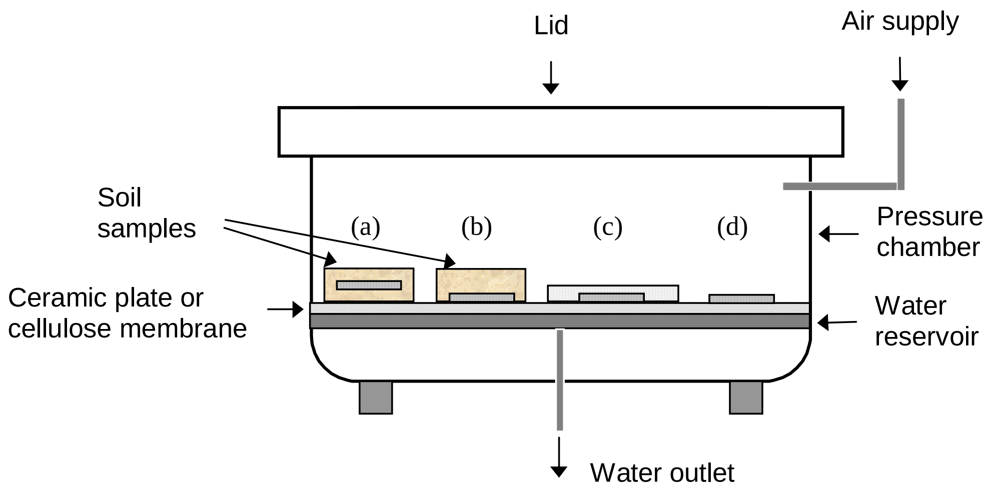


Figure 7. Filter Paper Wetting Calibration Curve in kPa Units.

Calibration for the Suction Drying Curve

Pressure plate and pressure membrane devices were employed in the drying filter paper calibration. A schematic drawing of a pressure plate or pressure membrane apparatus is depicted in Fig. 8. For the drying suction calibration of the filter paper, a contact path is provided between the filter paper and the measuring device so as to eliminate the osmotic suction component of total suction. In other words, if transfer of the soil water is allowed only through fluid flow, dissolved salts will move with the soil water, and the measuring device will not detect the osmotic suction component.

Pressure plate and pressure membrane devices operate by imposing a suction value (i.e., applied air pressure minus water pressure at atmospheric condition) on a given specimen which can be a soil or filter paper. The filter paper is put into the suction measuring device in a manner that ensures good contact with the porous plate or cellulose membrane. In this process, the main concern is to make sure that an intimate contact is provided between the water inside the filter paper and the water inside the porous disk so that transfer of the water is allowed only through continuous water films. To investigate the degree of contact between the filter paper and porous disk, the testing procedure and setup as depicted in Fig. 8 were undertaken in this study. Three different soils (i.e., a fine clay, sandy silt, and pure sand) were used in the calibration process of filter papers in order to investigate the role of soils in establishing a good contact between the filter paper and porous disk.



(a) One filter paper between two larger size protective filter papers embedded into the soil sample.
(b) One filter paper makes contact with the porous plate or membrane and covered on top with a larger size protective filter paper in the soil sample.
(c) One filter paper makes contact with the porous plate or membrane and covered on top with two larger size protective filter papers.
(d) One filter paper on the porous plate or membrane.

Figure 8. Schematic Drawing of a Pressure Plate or Membrane Device.

Experimental Procedure for Drying Curve Calibration

The procedure that was adopted for the experiment is as follows:

1. Prior to each test, the porous disk or membrane and the soils were saturated with distilled water at least one day in advance, so that all the pores were fully saturated with water.
2. The testing configuration as in Fig. 8 was established using one of the soils (i.e., fine clay or sandy silt or pure sand). Figure 8 explains how the filter papers, soil, and protective papers were arranged in the experiment. The soil specimens with the filter papers were placed on the saturated disks and the level of distilled water on the plate was raised enough to cover all of the filter papers. All of the air bubbles were eliminated during placement of the filter paper, soil, and protective paper arrangement on the ceramic disk by carefully pressing the bubbles out to the edges of each. The air bubbles were pressed out of the sample using a small diameter glass pipe and a large diameter glass cylinder.

3. After the pressure chamber was tightened, with the influence of the applied air pressure the water inside the soil specimen and filter papers were forced out through the porous plate or membrane and collected in a graduated cylinder until a suction equilibrium between the soil and filter papers and the applied air pressure was established.

An equilibration period between 3 and 5 days is commonly suggested for matric suction measurements using pressure plates and membranes (ASTM D 5298, Houston et al. 1994, Lee 1991). The equilibrating periods used for this study varied between 3, 5, and 7 days depending on the testing set up. For instance, when filter papers were embedded in the soil, equilibrating periods were 7 days for the fine clay and 5 days for the sandy silt set up, but the equilibrating period was 3 days when filter papers embedded in the pure sand or when only filter papers were used. However, all the three soils were also tested with filter papers inside in the same pressure chamber to check the differences between the filter paper water contents. To obtain the filter paper water contents, the same procedure described in the Wetting Curve Calibration Procedure was followed.

Drying Calibration Curve

A drying curve was established from the filter paper test results by following the procedure described above. The curve obtained for Schleicher & Schuell No. 589-WH filter papers using both pressure plate and pressure membrane devices is depicted in Fig. 9. Each data point on Fig. 9 is an average of at least three tests and each test data is an average of at least four filter papers. The standard errors for the straight line and curved portions of the drying curve are 0.135 and 0.116 log kPa units, respectively. The standard error for the straight line portion of the wetting curve is 0.044 log kPa. With the pressure membrane the highest matric suction obtained was 4,570 kPa and suctions below 150 kPa were obtained using the pressure plate apparatus. The corresponding wetting calibration curve is also shown in Fig. 9. It plots below the drying suction curve, as is expected of the hysteresis process.

Very high filter paper water contents were obtained when all the three soils were used as in the set up (a) as shown in Fig. 8. However, the filter paper water contents were all comparable as obtained from the set ups (b), (c), and (d) as in Fig. 8. The results from (b) were slightly wetter than (c) and the results from (d) were slightly drier than (c). In obtaining the calibration curve, the filter papers from the set up arrangements (b), (c), and (d) were used.

Soil Total and Matric Suction Measurements

Soil total suction measurements are similar to those measurements in the filter paper calibration testing. The same testing procedure can be followed by replacing the salt solution with a soil sample.

Soil matric suction measurements are also similar to the total suction measurements except that an intimate contact should be provided between the filter paper and the soil. A suggested testing procedure for soil total and matric suction measurements using filter papers is outlined in Appendix.

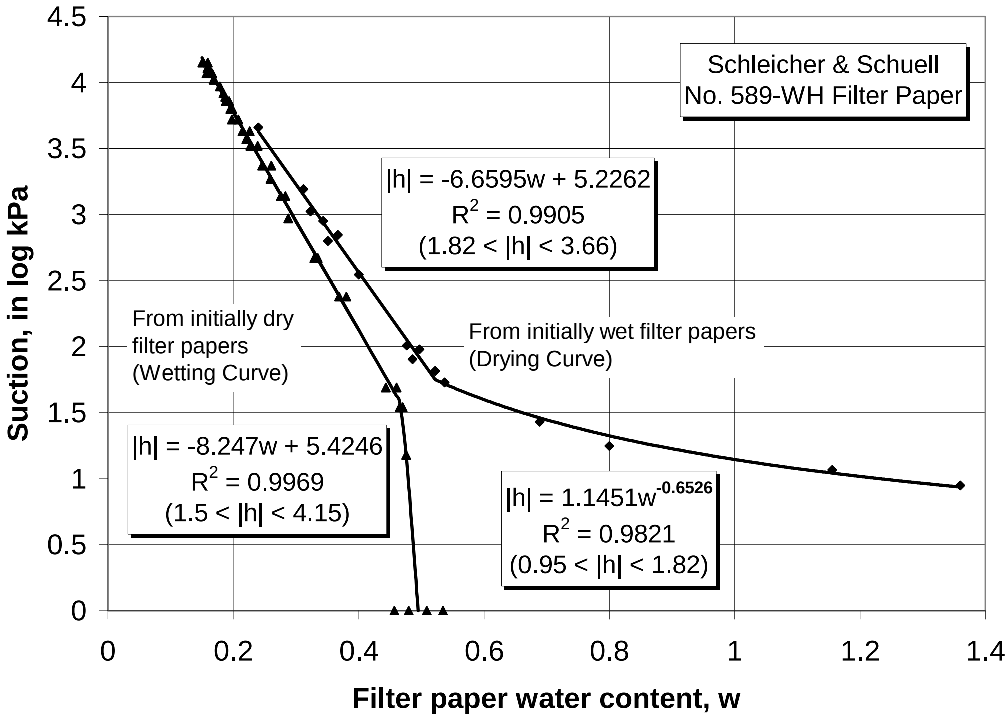


Figure 9. Drying Suction Calibration Curve along with Wetting Suction Curve.

Discussion

The dramatic decrease of total suction at high filter paper water contents is related to the nature of Kelvin's equation and to the use of the logarithmic scale (i.e., log kPa or pF). These results conclude that the filter paper method can give reliable wetting suction results up to a point. In other words, with the Schleicher & Schuell No. 589-WH filter papers reliable wetting suction measurements can be taken at and above 1.5 log kPa (2.5 pF), but below about 1.5 log kPa wetting suction results cannot be relied upon because a small error in measuring water content can result in a large error in the inferred suction. Therefore, a best fit line up to 1.5 log kPa was made to plot Fig. 4, below which there is a sudden drop in the wetting suction.

In the drying filter paper calibration testing, filter papers are initially fully saturated and with the application of air pressure the water inside the filter paper is driven out, which is a drying process. However, the soil matric and total suction measurements follow a wetting process with the filter paper method. Because of hysteresis, the wetting suction calibration curve must always plot below the drying calibration curve. A final point; because both the matric and total suction measurements are <u>wetting</u> processes, they should, by these arguments, both be determined from the wetting calibration curve.

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Appendix. Soil Suction Measurements

Soil Total Suction Measurements

Glass jars that are between 250 to 500 ml volume size are readily available in the market and can be easily adopted for suction measurements. Glass jars, especially, with 3.5 to 4 inch (8.89 to 10.16 cm) diameter can contain the 3 inch (7.62 cm) diameter Shelby tube samples very nicely. A testing procedure for total suction measurements using filter papers can be outlined as follows:

Experimental Procedure

1. At least 75 percent by volume of a glass jar is filled up with the soil; the smaller the empty space remaining in the glass jar, the smaller the time period that the filter paper and the soil system requires to come to equilibrium.
2. A ring type support, which has a diameter smaller than filter paper diameter and about 1 to 2 cm in height, is put on top of the soil to provide a non-contact system between the filter paper and the soil. Care must be taken when selecting the support material; materials that can corrode should be avoided, plastic or glass type materials are much better for this job.
3. Two filter papers one on top of the other are inserted on the ring using tweezers. The filter papers should not touch the soil, the inside wall of the jar, and underneath the lid in any way.
4. Then, the glass jar lid is sealed very tightly with plastic tape.
5. Steps 1, 2, 3, and 4 are repeated for every soil sample.
6. After that, the glass jars are put into the ice-chests in a controlled temperature room for equilibrium.

Researchers suggest a minimum equilibrating period of one week (ASTM D 5298, Houston et al. 1994, Lee 1991). After the equilibration time, the procedure for the filter paper water content measurements can be as follows:

1. Before removing the glass jar containers from the temperature room, all aluminum cans that are used for moisture content measurements are weighed to the nearest 0.0001 g. accuracy and recorded.
2. After that, all measurements are carried out by two persons. For example, while one person is opening the sealed glass jar, the other is putting the filter paper into the aluminum can very quickly (i.e., in a few seconds) using tweezers.
3. Then, the weights of each can with wet filter paper inside are taken very quickly.

4. Steps 2 and 3 are followed for every glass jar. Then, all cans are put into the oven with the lids half-open to allow evaporation. All filter papers are kept at 105 ± 5°C temperature inside the oven for at least 10 hours.
5. Before taking measurements on the dried filter papers, the cans are closed with their lids and allowed to equilibrate for about 5 minutes. Then, a can is removed from the oven and put on an aluminum block (i.e., heat sinker) for about 20 seconds to cool down; the aluminum block functions as a heat sink and expedites the cooling of the can. After that, the can with the dry filter paper inside is weighed very quickly. The dry filter paper is taken from the can and the cooled can is weighed again in a few seconds.
6. Step 5 is repeated for every can.

After obtaining all of the filter paper water contents an appropriate calibration curve, such as the one in Fig. 4, is employed to get total suction values of the soil samples.

Soil Matric Suction Measurements

Soil matric suction measurements are similar to the total suction measurements except instead of inserting filter papers in a non-contact manner with the soil for total suction testing, a good intimate contact should be provided between the filter paper and the soil for matric suction measurements. Both matric and total suction measurements can be performed on the same soil sample in a glass jar as shown in Fig. A1. A testing procedure for matric suction measurements using filter papers can be outlined as follows:

Experimental Procedure

1. A filter paper is sandwiched between two larger size protective filter papers. The filter papers used in suction measurements are 5.5 cm in diameter, so either a filter paper is cut to a smaller diameter and sandwiched between two 5.5 cm papers or bigger diameter (bigger than 5.5 cm) filter papers are used as protectives.
2. Then, these sandwiched filter papers are inserted into the soil sample in a very good contact manner (i.e., as in Fig. A1). An intimate contact between the filter paper and the soil is very important.
3. After that, the soil sample with embedded filter papers is put into the glass jar container. The glass container is sealed up very tightly with plastic tape.
4. Steps 1, 2, and 3 are repeated for every soil sample.
5. The prepared containers are put into ice-chests in a controlled temperature room for equilibrium.

Researchers suggest an equilibration period of 3 to 5 days for matric suction testing (ASTM D 5298, Houston et al. 1994, Lee 1991). However, if both matric and total suction measurements are performed on the same sample in the glass jar, then the final equilibrating time will be at least 7 days of total suction equilibrating period. The procedure for the filter paper water content measurements at the end of the equilibration is exactly same as the one outlined for the total suction water content measurements. After obtaining all the filter paper water contents the appropriate calibration curve may be employed to get the matric suction values of the soil samples.

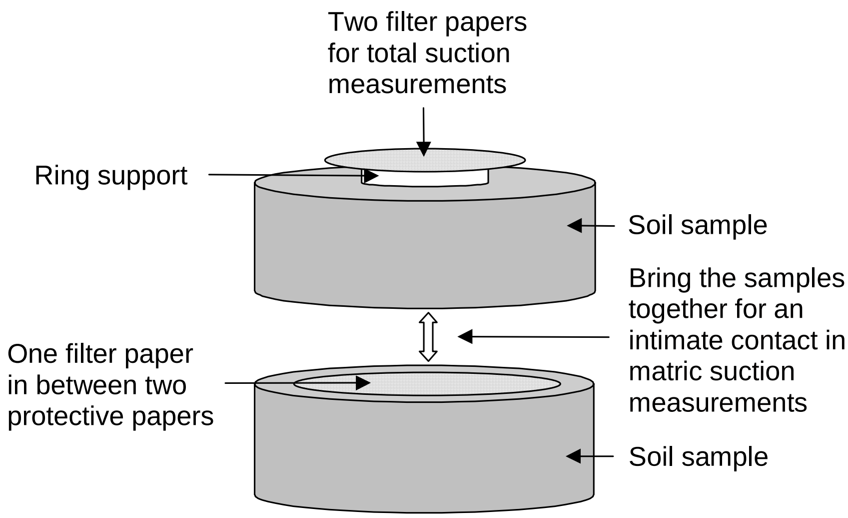


Figure. A1. Total and Matric Suction Measurements.

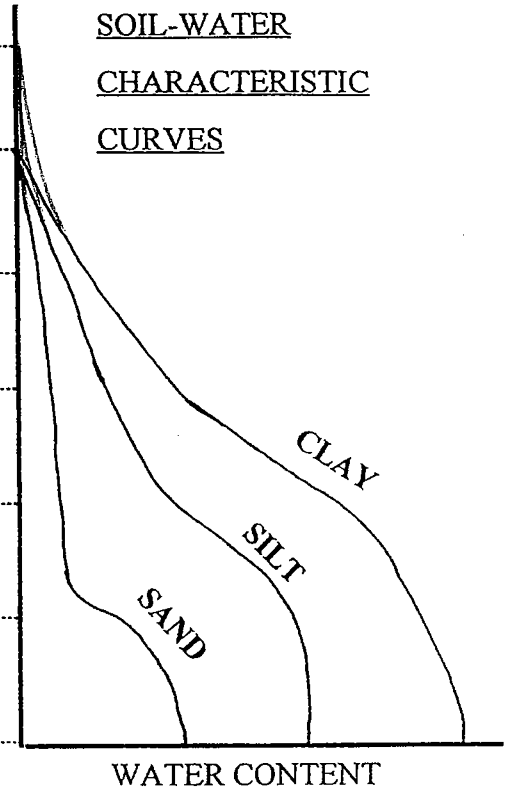
SOIL WATER POTENTIAL ENERGY

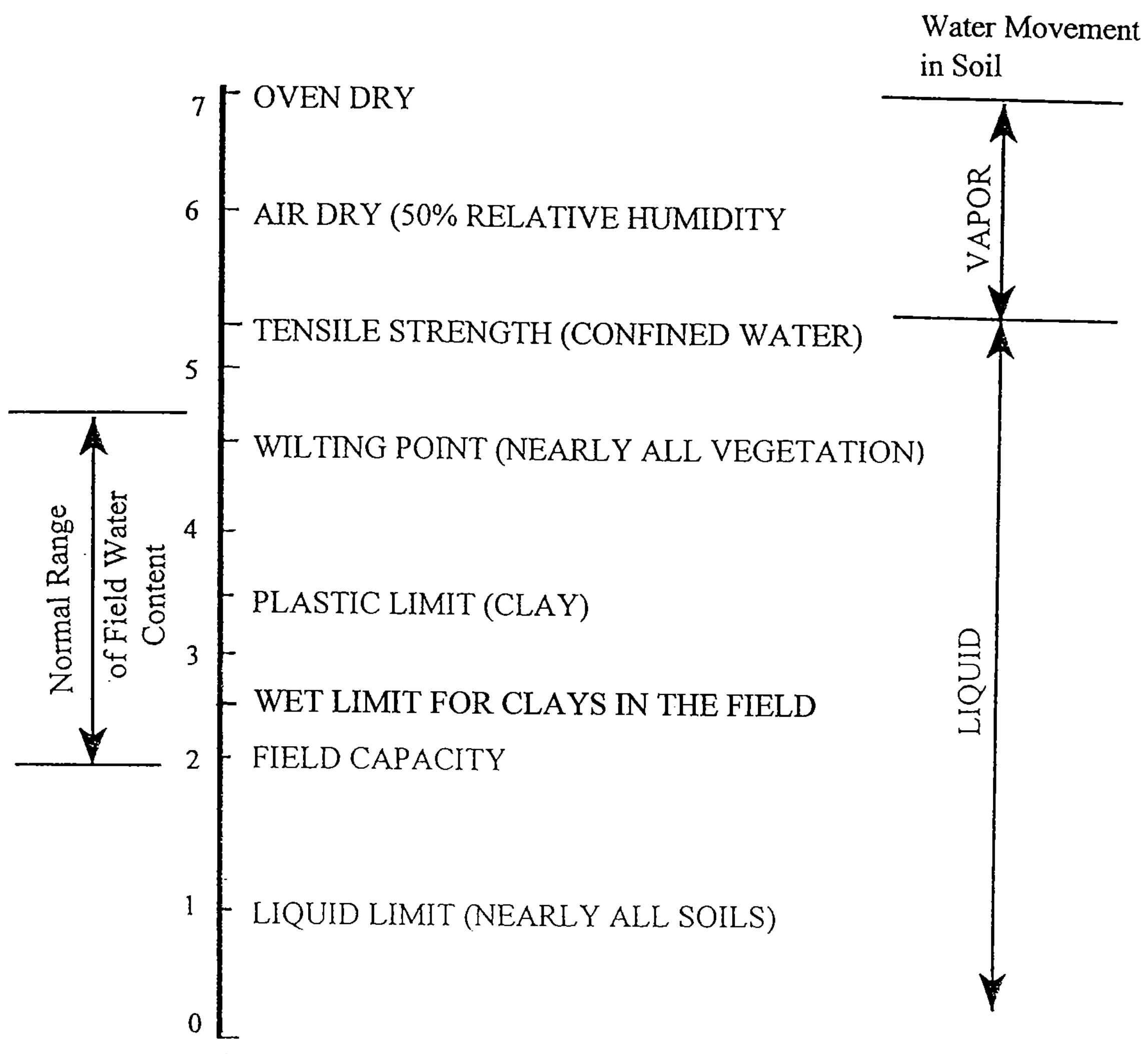
SCALES

PSI	Atm (Bar)	pF	cm	mm	kPa
-14200	-10000	7	-10^7	-10^8	-10^6
-14200	-1000	6	-10^6	-10^7	-10^5
-1420	-100	5	-10^5	-10^6	-10^4
-142	-10	4	-10^4	-10^5	-10^3
-14.2	-1	3	-10^3	-10^4	-10^2
-1.42	-0.1	2	-10^2	-10^3	-10^1
-0.142	-0.01	1	-10^1	-10^2	-10^0

pF and cm: soil physics units; mm and kPa: SI units

SOIL-WATER CHARACTERISTIC CURVES




Water Movement
in Soil
7
OVEN DRY
6
AIR DRY (50% RELATIVE HUMIDITY
TENSILE STRENGTH (CONFINED WATER)
5
WILTING POINT (NEARLY ALL VEGETATION)
4
PLASTIC LIMIT (CLAY)
3
WET LIMIT FOR CLAYS IN THE FIELD
2
FIELD CAPACITY
1
LIQUID LIMIT (NEARLY ALL SOILS)
0
Normal Range of Field Water Content
VAPOR
LIQUID

pF - SCALE

SOIL SUCTION CONCEPT

In general, porous materials have a fundamental ability to retain water. The existence of this fundamental property in soils is described in engineering terms as suction, negative stress in the pore water. In engineering practice, soil suction is composed of two components: matric and osmotic suction. The sum of matric and osmotic suction is called total suction. Matric suction comes from the capillarity, texture, and surface adsorptive forces of the soil. Osmotic suction arises from the dissolved salts contained in the soil water. This relationship can be formed in an equation as follows:

$$h_t = h_m + h_\pi \tag{1}$$

where h_t = total suction (kPa), h_m = matric suction (kPa), and h_π = osmotic suction (kPa).

Total suction can be calculated using Kelvin's equation, which is derived from the ideal gas law using the principles of thermodynamics and is given as:

$$h_t = \frac{RT}{V}\ln\left(\frac{P}{P_o}\right) \tag{2}$$

where h_t = total suction, R = universal gas constant, T = absolute temperature, V = molecular volume of water, P/P_o = relative humidity, P = partial pressure of pore water vapor, and P_o = saturation pressure of water vapor over a flat surface of pure water at the same temperature.

If Eq. (2) is evaluated at a reference temperature of 25°C, the following total suction and relative humidity relationship can be obtained:

$$h_t = 137182 \times \ln(P/P_o) \tag{3}$$

Figure 1 shows a plot of Eq. (3) at 25°C temperature. From Fig. 1, it can be seen that there is nearly a linear relationship between total suction (h_t) and relative humidity (P/P_o) over a very small relative humidity range. It can be said, in general, that in a closed system under isothermal conditions the relative humidity may be associated with the water content of the system such as 100 percent relative humidity refers to a fully saturated condition. Therefore, the suction value of a soil sample can be inferred from the relative humidity and suction relationship if the relative humidity is evaluated in some way. In a closed system, if the water is pure enough, the partial pressure of the water vapor at equilibrium is equal to the saturated vapor pressure at temperature, T. However, the partial pressure of the water vapor over a partly saturated soil will be less than the saturation vapor pressure of pure water due to the soil matrix structure and the free ions and salts contained in the soil water (Fredlund and Rahardjo 1993).

In engineering practice, soil suction has usually been calculated in *pF* units (Schofield 1935) (i.e., suction in $pF = log_{10}(|suction\ in\ cm\ of\ water|)$). However, soil suction is also currently being represented in *log kPa* unit system (Fredlund and Rahardjo 1993) (i.e., suction in $log\ kPa = log_{10}(|suction\ in\ kPa|)$). The relationship between these two systems of units is approximately *suction in log kPa = suction in pF – 1.*

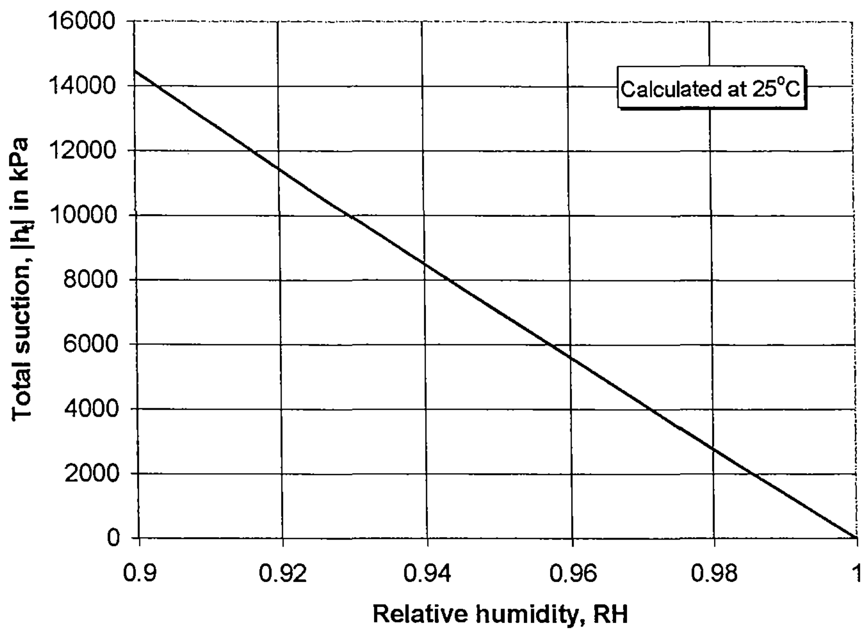


FIG. 1. Total Suction versus Relative Humidity

If total suction in kPa from Fig. 1 is converted to log kPa units, Fig. 2 is obtained. The difference between Fig. 1 and Fig. 2 is only the suction unit. The suction unit in Fig. 1 is kPa whereas it is log kPa in Fig. 2. From Fig. 2 it can clearly be seen that when relative humidity approaches 100 percent, the total suction becomes very sensitive. The sensitivity in the suction is due to the common logarithm used to convert suction from kPa to the log kPa unit.

Matric suction can be calculated from pressure plate and pressure membrane devices as the difference between the applied air pressure and water pressure across a porous plate. Matric suction can be formed in a relationship as follows:

$$-h_m = u_a - u_w \tag{4}$$

where h_m = matric suction, u_a = applied air pressure, and u_w = free water pressure at atmospheric condition.

The osmotic suction of electrolyte solutions, that are usually employed in the calibration of filter papers and psychrometers, can be calculated using the relationship between osmotic coefficients and osmotic suction. Osmotic coefficients are readily available in the literature for many different salt solutions. Table 1 gives the osmotic coefficients for several salt solutions. Osmotic coefficients can also be obtained from the following relationship (Lang 1967):

$$\phi = -\frac{\rho_w}{vmw}\ln\left(\frac{P}{P_o}\right) \tag{5}$$

where ϕ = osmotic coefficient, v = number of ions from one molecule of salt (i.e., v = 2 for NaCl, KCl, NH_4Cl and v = 3 for Na_2SO_4, $CaCl_2$, $Na_2S_2O_3$, etc.), m = molality, w = molecular mass of water, and ρ_w = density of water.

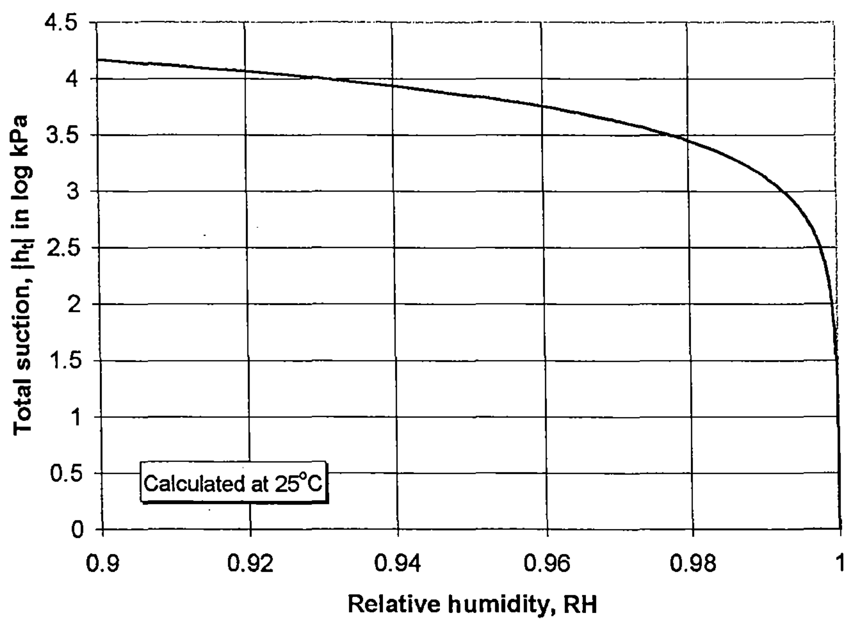


FIG. 2. Total Suction and Relative Humidity Relationship

The relative humidity term (P/P_o) in Eq. (5) is also known as the activity of water (a_w) in physical chemistry of electrolyte solutions. The combination of Eq. (2) and Eq. (5) gives a useful relationship that can be adopted to calculate osmotic suctions for different salt solutions:

$$h_\pi = -vRTm\phi \tag{6}$$

Table 2 gives osmotic suctions for several salt solutions using osmotic coefficients from Table 1 and Eq. (6).

THE FILTER PAPER METHOD

The filter paper method has long been used in soil science and engineering practice and it has recently been accepted as an adaptable test method for soil suction measurements because of its advantages over other suction measurement devices. Basically, the filter paper comes to equilibrium with the soil either through vapor (total suction measurement) or liquid (matric suction measurement) flow. At

TABLE 1. Osmotic Coefficients of Several Salt Solutions

	Osmotic Coefficients at 25°C						
Molality (m)	NaCl[a]	KCl[a]	NH_4Cl[a]	Na_2SO_4[b]	$CaCl_2$[c]	$Na_2S_2O_3$[b]	$MgCl_2$[c]
0.001	0.9880	0.9880	0.9880	0.9608	0.9623	0.9613	0.9627
0.002	0.9840	0.9840	0.9840	0.9466	0.9493	0.9475	0.9501
0.005	0.9760	0.9760	0.9760	0.9212	0.9274	0.9231	0.9292
0.010	0.9680	0.9670	0.9670	0.8965	0.9076	0.8999	0.9106
0.020	0.9590	0.9570	0.9570	0.8672	0.8866	0.8729	0.8916
0.050	0.9440	0.9400	0.9410	0.8229	0.8619	0.8333	0.8708
0.100	0.9330	0.9270	0.9270	0.7869	0.8516	0.8025	0.8648
0.200	0.9240	0.9130	0.9130	0.7494	0.8568	0.7719	0.8760
0.300	0.9210	0.9060	0.9060	0.7262	0.8721	0.7540	0.8963
0.400	0.9200	0.9020	0.9020	0.7088	0.8915	0.7415	0.9206
0.500	0.9210	0.9000	0.9000	0.6945	0.9134	0.7320	0.9475
0.600	0.9230	0.8990	0.8980	0.6824	0.9370	0.7247	0.9765
0.700	0.9260	0.8980	0.8970	0.6720	0.9621	0.7192	1.0073
0.800	0.9290	0.8980	0.8970	0.6629	0.9884	0.7151	1.0398
0.900	0.9320	0.8980	0.8970	0.6550	1.0159	0.7123	1.0738
1.000	0.9360	0.8980	0.8970	0.6481	1.0444	0.7107	1.1092
1.200	0.9440	0.9000	0.8980	...	...	...	...
1.400	0.9530	0.9020	0.9000	...	...	...	...
1.500	...	...	...	0.6273	1.2004	0.7166	1.3047
1.600	0.9620	0.9050	0.9020	...	...	...	...
1.800	0.9730	0.9080	0.9050	...	...	...	...
2.000	0.9840	0.9120	0.9080	0.6257	1.3754	0.7410	1.5250
2.500	1.0130	0.9230	0.9170	0.6401	1.5660	0.7793	1.7629

References:
[a]Hamer and Wu, 1972
[b]Goldberg, 1981
[c]Goldberg and Nuttall, 1978

equilibrium, the suction value of the filter paper and the soil will be equal. After equilibrium is established between the filter paper and the soil, the water content of the filter paper disc is measured. Then, by using a filter paper water content versus suction calibration curve, the corresponding suction value is found from the curve. This is the basic approach suggested by ASTM Standard Test Method for

Measurement of Soil Potential (Suction) Using Filter Paper (D 5298). In other words, ASTM D 5298 employs a single calibration curve that has been used to infer both total and matric suction measurements. The ASTM D 5298 calibration curve is a combination of both wetting and drying curves. However, this paper demonstrates that the "wetting" and "drying" suction calibration curves do not match, an observation that was also made by Houston et al. (1994).

TABLE 2. Osmotic Suctions of Several Salt Solutions

	Osmotic Suctions in kPa at 25°C						
Molality (m)	NaCl	KCl	NH_4Cl	Na_2SO_4	$CaCl_2$	$Na_2S_2O_3$	$MgCl_2$
0.001	5	5	5	7	7	7	7
0.002	10	10	10	14	14	14	14
0.005	24	24	24	34	34	34	35
0.010	48	48	48	67	67	67	68
0.020	95	95	95	129	132	130	133
0.050	234	233	233	306	320	310	324
0.100	463	460	460	585	633	597	643
0.200	916	905	905	1115	1274	1148	1303
0.300	1370	1348	1348	1620	1946	1682	2000
0.400	1824	1789	1789	2108	2652	2206	2739
0.500	2283	2231	2231	2582	3396	2722	3523
0.600	2746	2674	2671	3045	4181	3234	4357
0.700	3214	3116	3113	3498	5008	3744	5244
0.800	3685	3562	3558	3944	5880	4254	6186
0.900	4159	4007	4002	4384	6799	4767	7187
1.000	4641	4452	4447	4820	7767	5285	8249
1.200	5616	5354	5343	...	...	...	...
1.400	6615	6261	6247	...	...	...	...
1.500	...	...	...	6998	13391	7994	14554
1.600	7631	7179	7155	...	...	...	...
1.800	8683	8104	8076	...	...	...	...
2.000	9757	9043	9003	9306	20457	11021	22682
2.500	12556	11440	11366	11901	29115	14489	32776

Calibration for the Suction Wetting Curve

The calibration for the suction wetting curve for filter paper using salt solutions is based upon the thermodynamic relationship between total suction (or osmotic suction) and the relative humidity resulting from a specific concentration of a salt in distilled water. The thermodynamic relationship between total suction and relative humidity is given in Eq. (2). In this study, NaCl was selected as an osmotic suction source for the filter paper calibration. Salt concentrations from 0 (distilled water) to

SOIL TOTAL AND MATRIC SUCTION MEASUREMENTS

Soil total suction measurements are similar to those measurements in the filter paper calibration testing. The same testing procedure can be followed by replacing the salt solution with a soil sample. Soil matric suction measurements are also similar to the total suction measurements except that an intimate contact should be provided between the filter paper and the soil. A suggested testing procedure for soil total and matric suction measurements using filter papers is outlined in Appendix II.

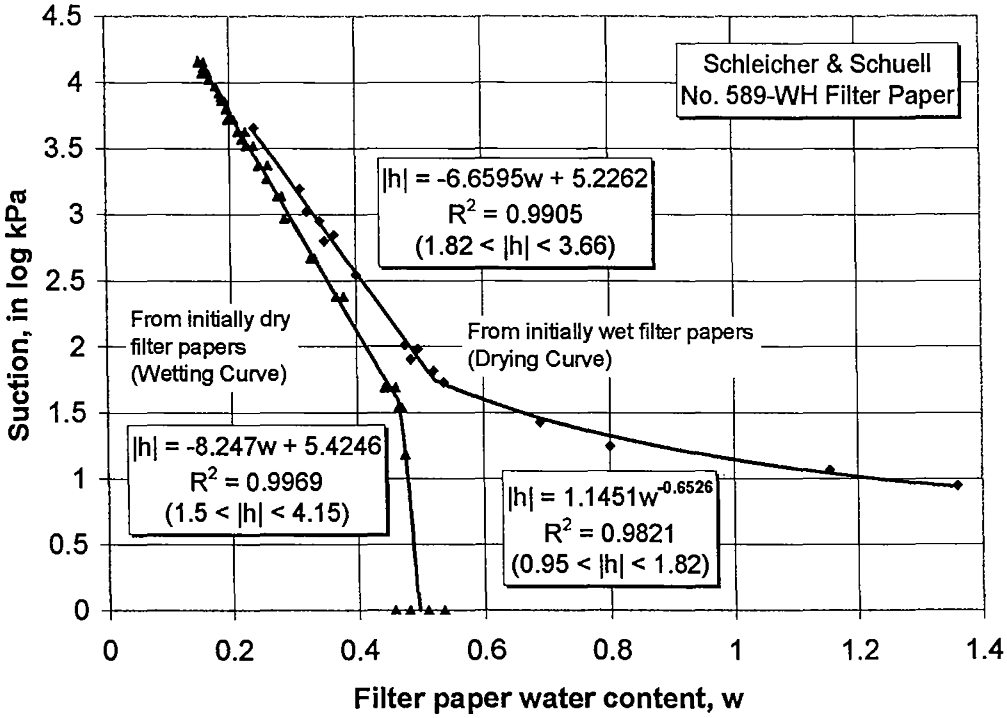


FIG. 9. Drying Suction Calibration Curve along with Wetting Suction Curve

DISCUSSION

The dramatic decrease of total suction at high filter paper water contents is related to the nature of Kelvin's equation and to the use of the logarithmic scale (i.e., log kPa or pF). These results conclude that the filter paper method can give reliable wetting suction results up to a point. In other words, with the Schleicher & Schuell No. 589-WH filter papers reliable wetting suction measurements can be taken at and above 1.5 log kPa (2.5 pF), but below about 1.5 log kPa wetting suction results cannot be relied upon because a small error in measuring water content can result in a large error in the inferred suction. Therefore, a best fit line up to 1.5 log kPa was made to plot Fig. 4, below which there is a sudden drop in the wetting suction.

In the drying filter paper calibration testing, filter papers are initially fully saturated and with the application of air pressure the water inside the filter paper is driven out, which is a drying process. However, the soil matric and total suction measurements follow a wetting process with the filter paper method. Because of hysteresis, the wetting suction calibration curve must always plot below the drying calibration curve. A final point; because both the matric and total suction measurements are <u>wetting</u> processes, they should, by these arguments, both be determined from the wetting calibration curve.

ACKNOWLEDGEMENTS

The authors wish to thank Dr. Hsiu-Chung Lee for helping with the total suction calibration testing and Dr. Seong-Wan Park for helping with the matric suction calibration testing.

APPENDIX I. REFERENCES

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APPENDIX II. SOIL SUCTION MEASUREMENTS

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Experimental Procedure

1. At least 75 percent by volume of a glass jar is filled up with the soil; the smaller the empty space remaining in the glass jar, the smaller the time period that the filter paper and the soil system requires to come to equilibrium.
2. A ring type support, which has a diameter smaller than filter paper diameter and about 1 to 2 cm in height, is put on top of the soil to provide a non-contact system between the filter paper and the soil. Care must be taken when selecting the support material; materials that can corrode should be avoided, plastic or glass type materials are much better for this job.
3. Two filter papers one on top of the other are inserted on the ring using tweezers. The filter papers should not touch the soil, the inside wall of the jar, and underneath the lid in any way.
4. Then, the glass jar lid is sealed very tightly with plastic tape.
5. Steps 1, 2, 3, and 4 are repeated for every soil sample.
6. After that, the glass jars are put into the ice-chests in a controlled temperature room for equilibrium.

Researchers suggest a minimum equilibrating period of one week (ASTM D5298, Houston et al. 1994, Lee 1991). After the equilibration time, the procedure for the filter paper water content measurements can be as follows:

1. Before removing the glass jar containers from the temperature room, all aluminum cans that are used for moisture content measurements are weighed to the nearest 0.0001 g. accuracy and recorded.
2. After that, all measurements are carried out by two persons. For example, while one person is opening the sealed glass jar, the other is putting the filter paper into the aluminum can very quickly (i.e., in a few seconds) using tweezers.
3. Then, the weights of each can with wet filter paper inside are taken very quickly.

Osmotic suction is computed from Van't Hoff's equation (and its higher order variants):

Number of ions from one molecule of solute

Molal concentration (moles/1000g of solvent)

Osmotic coefficient

$$\pi = \nu \; R \; T \; C \; \phi$$

$$\frac{gm - cm}{gm} = \frac{8.314 \times 10^{7}}{981 \times 1000 \; \frac{g}{kg}} \; \frac{ergs}{K - mol} \qquad (273 + {}^{\circ}C)\;K$$

Molal concentration is the number of <u>moles</u> of a substance that is dissolved in 1000gm of a solution:

Mass of substance dissolved in 1000g of a solvent

$$C = \frac{(\ldots)\;\dfrac{gm}{1000gm}}{(\ldots)\;\dfrac{gm}{mole}}$$

Molecular weight of the substance

$$\frac{number\ of\ moles}{1000g\ solvent}$$

<u>Examples</u>: Osmotic suction at 25°C

$$\pi_{\frac{gmcm}{gm}} = \nu \frac{8.314 \times 10^{7}}{981 \times 1000}(273 + 25) \times C\phi = 2.52556 \times 10^{4}\; \nu\, C\,\phi$$

$$\log_{10} \pi = pF = 4.40 + \log_{10}[C\phi] + \log_{10}[\nu]$$

Example Continued

Molal Concentration, C	π Osmotic Suction $\nu = 1, \phi = 1$ $\frac{gm - cm}{gm}$	pF
0.0001	2.53	0.40
0.001	2.53 x 10	1.40
0.01	2.53×10^2	2.40
0.1	2.53×10^3	3.40
1.0	2.53×10^4	4.40
10	2.53×10^5	5.40
100	2.53×10^6	6.40

Molecular Weights of Common Solutes

Solute	Molecular Wt gms/mole	Solute	Molecular Wt gms/mole
Ethanol	46.07	Acetone	58.08
Methyl Ethyl Ketone	72.10	Benzene	78.11
2-Propanol	60.09	Toluene	92.13
Methanol	32.04	Trichloroethylene	131.40
O-nitroaniline	138.12	Tetrachloroethylene	165.85
Polychlorobiphenyls	188.65		
Sodium Chloride	58.46		

Solution PF (25°C)	C $\frac{Moles}{kg\ Solvent}$ Molal Concentration		Solute Concentrations: Sodium Chloride (mg/ℓ*) ν = 2	Solute Concentrations: Ethanol (mg/ℓ) ν = 1	Solute Concentrations: Methyl Ethyl Ketone (mg/ℓ*) ν = 1
	ν = 1	ν = 2			
0.40	10^{-4}	0.5×10^{-4}	2.92	4.61	7.21
1.40	10^{-3}	0.5×10^{-3}	29.23	46.07	72.10
2.40	10^{-2}	0.5×10^{-2}	292.3	460.7	721.0
3.40	0.1	0.05	2923	4,607	7210
4.40	1	0.5	29,230	46,070	72,100
5.40	10	5	292,300	460,700	721,000
6.40	100	50	2,923,000	4,607,000	7,210,000

* The concentration is moles/kg, C, is related to the concentration in mg/liter as follows:

$$C\frac{moles}{kg} = \frac{(concentration)\frac{mg}{liter} \; x \; \frac{1\ liter}{1000g} \; x \; \frac{1000g}{kg}}{1000\frac{mg}{gm} \; x \; (Molecular\ Wt)\frac{gm}{mole}}$$

A larger concentrations and with heavier molecules this form of Van't Hoff's equation becomes more approximate. However, for molal concentrations less than about 0.1 moles/kg Van't Hoff's equation is accurate.

Exercises

1. Generate a table of pF and osmotic suction values for temperatures ranging from 0°C to 40°C for every 10°C.

2. Find the molecular weights of inorganic salts that occur frequently in soils and determine their concentrations in mg/ℓ that will produce suction levels of 2, 3, 4, 4.5 (wilting pt), 5, 5.3, 6, and 7. Some of these salts are: sodium chloride, calcium carbonate, calcium sulfate, potassium chloride.

Table 1. VALUES OF LEACHATE SUCTION (Assuming $\nu = \phi = 1$)

Landfill Site	Type Landfill	Concentration of Leachate mg/1	Principal Component	Minimum PF of Leachate $\nu = 1$
Lyon	Municipal	213.0	Ethanol	2.06
Meeker	Municipal	14.4	Methyl Ethyl Ketone	0.70
Rochester	Municipal	98.3	2-Propanol	1.54
La Bounty	Mixed	281.2	O-Nitroaniline	1.71
Love Canal	Industrial	142.8	Methanol	1.89
Kin-Buc	Industrial	1837.3	Polychloro-biphenyls	2.39
				Nonelectrolyte

Designation: D 4318 – 84

Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils[1]

This standard is issued under the fixed designation D 4318; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

Scope

1.1 This test method covers the determination of the ıuid limit, plastic limit, and the plasticity index of soils as :fined in Section 3.

1.1.1 Two procedures for preparing test specimens and ′o procedures for performing the liquid limit are provided follows:

A Multipoint test using a wet preparation procedure, described in Sections 10.1, 11, and 12.

B Multipoint test using a dry preparation procedure, described in Sections 10.2, 11, and 12.

C One-point test using a wet preparation procedure, described in Sections 13, 14, and 15.

D One-point test using a dry preparation procedure, described in Sections 13, 14, and 15.

ıe procedure to be used shall be specified by the requesting ıthority. If no procedure is specified, Procedure A shall be ›ed.

NOTE 1—Prior to the adoption of this test method, a curved grooving ɔl was specified as part of the apparatus for performing the liquid limit ɪt. The curved tool is not considered to be as accurate as the flat tool scribed in 6.2 since it does not control the depth of the soil in the ·uid limit cup. However, there are some data which indicate that ɔically the liquid limit is slightly increased when the flat tool is used ɪtead of the curved tool.

1.1.2 The plastic limit test procedure is described in ɔctions 16, 17, and 18. The plastic limit test is performed on aterial prepared for the liquid limit test. In effect, there are ′o procedures for preparing test specimens for the plastic nit test.

1.1.3 The procedure for calculating the plasticity index is ven in Section 19.

1.2 The liquid limit and plastic limit of soils (along with e shrinkage limit) are often collectively referred to as the tterberg limits in recognition of their formation by Swedish il scientist, A. Atterberg. These limits distinguish the ›undaries of the several consistency states of plastic soils.

1.3 As used in this test method, soil is any natural ɡregation of mineral or organic materials, mixtures of such materials, or artificial mixtures of aggregates and natural mineral and organic particles.

1.4 The multipoint liquid limit procedure is somewhat more time consuming than the one-point procedure when both are performed by experienced operators. However, the one-point procedure requires the operator to judge when the test specimen is approximately at its liquid limit. In cases where this is not done reliably, the multipoint procedure is as fast as the one-point procedure and provides additional precision due to the information obtained from additional trials. It is particularly recommended that the multipoint procedure be used by inexperienced operators.

1.5 The correlations on which the calculations of the one-point procedure are based may not be valid for certain soils, such as organic soils or soils from a marine environment. The liquid limit of these soils should therefore be determined by the multipoint procedure (Procedure A).

1.6 The liquid and plastic limits of many soils that have been allowed to dry before testing may be considerably different from values obtained on undried samples. If the liquid and plastic limits of soils are used to correlate or estimate the engineering behavior of soils in their natural moist state, samples should not be permitted to dry before testing unless data on dried samples are specifically desired.

1.7 The composition and concentration of soluble salts in a soil affect the values of the liquid and plastic limits as well as the water content values of soils (see Method D 2216). Special consideration should therefore be given to soils from a marine environment or other sources where high soluble salt concentrations may be present. The degree to which the salts present in these soils are diluted or concentrated must be given consideration if meaningful results are to be obtained.

1.8 Since the tests described herein are performed only on that portion of a soil which passes the 425-μm (No. 40) sieve, the relative contribution of this portion of the soil to the properties of the sample as a whole must be considered when using these tests to evaluate the properties of a soil.

1.9 The values stated in acceptable metric units are to be regarded as the standard. The values given in parentheses are for information only.

1.10 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

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[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil ı Rock and is the direct responsibility of Subcommittee D18.03 on Texture, ısticity, and Density Characteristics of Soils.

Current edition approved Oct. 26, 1984. Published December 1984. Originally blished as D 4318 – 83. Last previous edition D 4318 – 83$^{\epsilon 1}$.

2. Referenced Documents

2.1 *ASTM Standards:*

C 702 Methods for Reducing Field Samples of Aggregate to Testing Size[2]

D 75 Practice for Sampling Aggregates[4]

D 420 Practice for Investigating and Sampling Soil and Rock for Engineering Purposes[4]

D 653 Terminology Relating to Soil, Rock, and Contained Fluids[4]

D 1241 Specification for Materials for Soil-Aggregate Subbase, Base, and Surface Courses[4]

D 2216 Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures[4]

D 2240 Test Method for Rubber Property—Durometer Hardness[5]

D 2487 Test Method for Classification of Soils for Engineering Purposes[4]

D 2488 Practice for Description and Identification of Soils (Visual-Manual Procedure)[4]

D 3282 Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes[4]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[6]

E 319 Methods of Testing Single-Arm Balances[6]

E 898 Method of Testing Top-Loading, Direct-Reading Laboratory Scales and Balances[6]

3. Definitions

3.1 *Atterberg limits*—originally, seven "limits of consistency" of fine-grained soils were defined by Albert Atterberg. In current engineering usage, the term usually refers only to the liquid limit, plastic limit, and in some references, the shrinkage limit.

3.2 *consistency*—the relative ease with which a soil can be deformed.

3.3 *liquid limit (LL)*—the water content, in percent, of a soil at the arbitrarily defined boundary between the liquid and plastic states. This water content is defined as the water content at which a pat of soil placed in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (½ in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of 2 shocks per second.

NOTE 2—The undrained shear strength of soil at the liquid limit is considered to be 2 ± 0.2 kPa (0.28 psi).

3.4 *plastic limit (PL)*—the water content, in percent, of a soil at the boundary between the plastic and brittle states. The water content at this boundary is the water content at which a soil can no longer be deformed by rolling into 3.2 mm (⅛ in.) in diameter threads without crumbling.

3.5 *plastic soil*—a soil which has a range of water content over which it exhibits plasticity and which will retain its shape on drying.

3.6 *plasticity index (PI)*—the range of water content over which a soil behaves plastically. Numerically, it is the difference between the liquid limit and the plastic limit.

3.7 *liquidity index*—the ratio, expressed as a percentage, of (*1*) the natural water content of a soil minus its plastic limit, to (*2*) its plasticity index.

3.8 *activity number (A)*—the ratio of (*1*) the plasticity index of a soil to (*2*) the percent by weight of particles having an equivalent diameter smaller than 0.002 mm.

4. Summary of Method

4.1 The sample is processed to remove any material retained on a 425-μm (No. 40) sieve. The liquid limit is determined by performing trials in which a portion of the sample is spread in a brass cup, divided in two by a grooving tool, and then allowed to flow together from the shocks caused by repeatedly dropping the cup in a standard mechanical device. The multipoint liquid limit, Procedures A and B, requires three or more trials over a range of water contents to be performed and the data from the trials plotted or calculated to make a relationship from which the liquid limit is determined. The one-point liquid limit, Procedures C and D, uses the data from two trials at one water content multiplied by a correction factor to determine the liquid limit.

4.2 The plastic limit is determined by alternately pressing together and rolling into a 3.2 mm (⅛ in.) diameter thread a small portion of plastic soil until its water content is reduced to a point at which the thread crumbles and is no longer able to be pressed together and rerolled. The water content of the soil at this stage is reported as the plastic limit.

4.3 The plasticity index is calculated as the difference between the liquid limit and the plastic limit.

5. Significance and Use

5.1 This test method is used as an integral part of several engineering classification systems to characterize the fine-grained fractions of soils (see Test Method D 2487 and Practice D 3282) and to specify the fine-grained fraction of construction materials (see Specification D 1241). The liquid limit, plastic limit, and plasticity index of soils are also used extensively, either individually or together with other soil properties to correlate with engineering behavior such as compressibility, permeability, compactibility, shrink-swell, and shear strength.

5.2 The liquid and plastic limits of a soil can be used with the natural water content of the soil to express its relative consistency or liquidity index and can be used with the percentage finer than 2-μm size to determine its activity number.

5.3 The one-point liquid limit procedure is frequently used for routine classification purposes. When greater precision is required, as when used for the acceptance of a material or for correlation with other test data, the multipoint procedure should be used.

5.4 These methods are sometimes used to evaluate the weathering characteristics of clay-shale materials. When subjected to repeated wetting and drying cycles, the liquid limits of these materials tend to increase. The amount of increase is considered to be a measure of a shale's susceptibility to weathering.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.
[3] *Annual Book of ASTM Standards*, Vols 04.02, 04.03, and 04.08.
[4] *Annual Book of ASTM Standards*, Vol 04.08.
[5] *Annual Book of ASTM Standards*, Vol 09.01.
[6] *Annual Book of ASTM Standards*, Vol 14.02.

5.5 The liquid limit of a soil containing substantial amounts of organic matter decreases dramatically when the soil is oven-dried before testing. Comparison of the liquid limit of a sample before and after oven-drying can therefore be used as a qualitative measure of organic matter content of a soil.

6. Apparatus

6.1 *Liquid Limit Device*—A mechanical device consisting of a brass cup suspended from a carriage designed to control its drop onto a hard rubber base. A drawing showing the essential features of the device and the critical dimensions is given in Fig. 1. The design of the device may vary provided that the essential functions are preserved. The device may be operated either by a hand crank or by an electric motor.

6.1.1 *Base*—The base shall be hard rubber having a D Durometer hardness of 80 to 90, and a resilience such that an 8-mm (5⁄16-in.) diameter polished steel ball, when dropped from a height of 25 cm (9.84 in.) will have an average rebound of at least 80 % but no more than 90 %. The tests shall be conducted on the finished base with feet attached.

6.1.2 *Feet*—The base shall be supported by rubber feet designed to provide isolation of the base from the work surface and having an A Durometer hardness no greater than 60 as measured on the finished feet attached to the base.

6.1.3 *Cup*—The cup shall be brass and have a weight, including cup hanger, of 185 to 215 g.

6.1.4 *Cam*—The cam shall raise the cup smoothly and continuously to its maximum height, over a distance of at least 180° of cam rotation. The preferred cam motion is a uniformly accelerated lift curve. The design of the cam and follower combination shall be such that there is no upward or downward velocity of the cup when the cam follower leaves the cam.

NOTE 3—The cam and follower design in Fig. 1 is for uniformly accelerated (parabolic) motion after contact and assures that the cup has no velocity at drop off. Other cam designs also provide this feature and may be used. However, if the cam-follower lift pattern is not known, zero velocity at drop off can be assured by carefully filing or machining the cam and follower so that the cup height remains constant over the last 20 to 45° of cam rotation.

6.1.5 *Carriage*—The cup carriage shall be constructed in a way that allows convenient but secure adjustment of the height of drop of the cup to 10 mm (0.394 in.). The cup hanger shall be attached to the carriage by means of a pin which allows removal of the cup and cup hanger for cleaning and inspection.

6.1.6 *Optional Motor Drive*—As an alternative to the hand crank shown in Fig. 1, the device may be equipped with a motor to turn the cam. Such a motor must turn the cam at 2 ± 0.1 revolutions per second and must be isolated from the rest of the device by rubber mounts or in some other way that prevents vibration from the motor being transmitted to the rest of the apparatus. It must be equipped with an ON-OFF switch and a means of conveniently positioning the cam for height of drop adjustments. The results obtained using a motor-driven device must not differ from those obtained using a manually operated device.

DIMENSIONS

LETTER	A^Δ	B^Δ	C^Δ	E^Δ	F	G	H	J^Δ	K^Δ	L^Δ	M^Δ
MM	54 ± 0.5	2 ± 0.1	27 ± 0.5	56 ± 2.0	32	10	16	60 ± 1.0	50 ± 2.0	150 ± 2.0	125 ± 2.0

LETTER	N	P	R	T	U^Δ	V	W	Z
MM	24	28	24	45	47 ± 1.0	3.8	13	6.5

Δ ESSENTIAL DIMENSIONS

CAM ANGLE DEGREES	CAM RADIUS
0	0.742 R
30	0.753 R
60	0.764 R
90	0.773 R
120	0.784 R
150	0.796 R
180	0.818 R
210	0.854 R
240	0.901 R
270	0.945 R
300	0.974 R
330	0.995 R
360	1.000 R

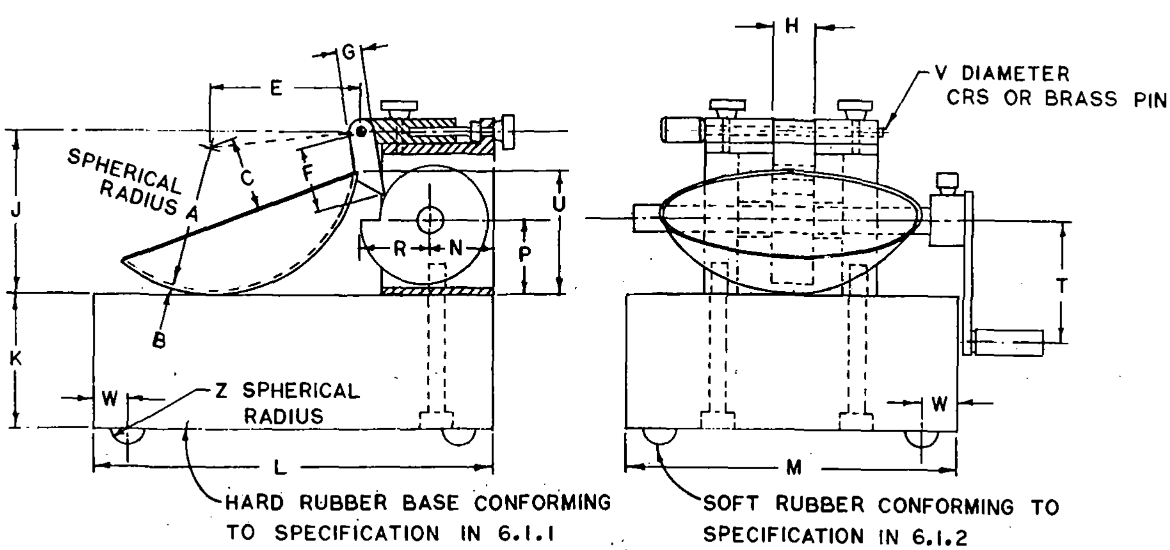


FIG. 1 Hand-Operated Liquid Limit Device

DIMENSIONS

LETTER	A△	B△	C△	D△	E△	F△
MM	2 ±0.1	11 ±0.2	40 ±0.5	8 ±0.1	50 ±0.5	2 ±0.1
LETTER	G	H	J	K△	L△	N
MM	10 MINIMUM	13	60	10 ±0.05	60 DEG ±1 DEG	20

△ ESSENTIAL DIMENSIONS

□ BACK AT LEAST 15 MM FROM TIP

NOTE: DIMENSION A SHOULD BE 1.9–2.0 AND DIMENSION D SHOULD BE 8.0–8.1 WHEN NEW TO ALLOW FOR ADEQUATE SERVICE LIFE

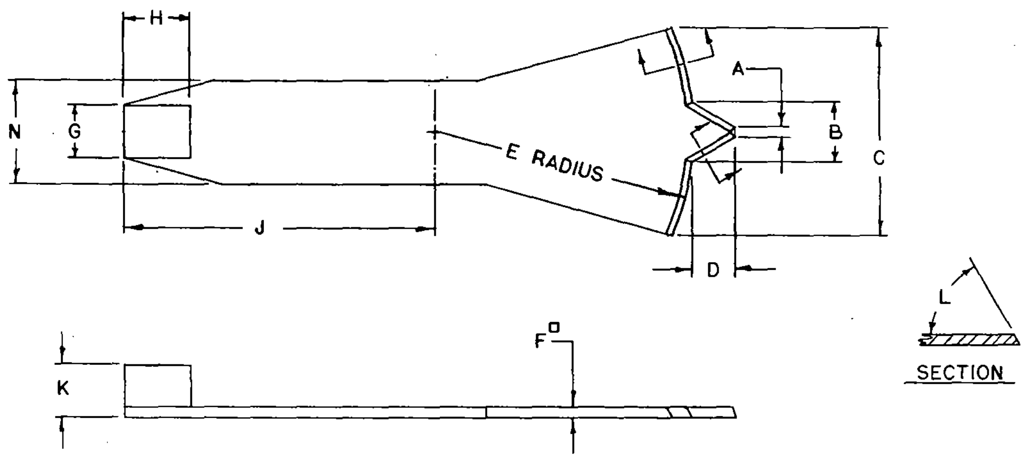
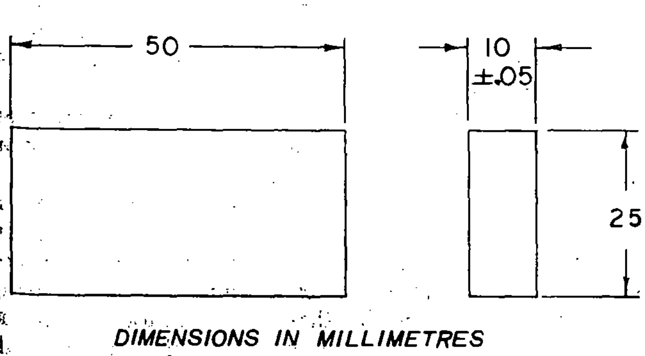


FIG. 2 Grooving Tool (Optional Height-of-Drop Gage Attached)

6.2 *Flat Grooving Tool*—A grooving tool having dimensions shown in Fig. 2. The tool shall be made of plastic or noncorroding metal. The design of the tool may vary as long as the essential dimensions are maintained. The tool may, but need not, incorporate the gage for adjusting the height of drop of the liquid limit device.

6.3 *Gage*—A metal gage block for adjusting the height of drop of the cup, having the dimensions shown in Fig. 3. The design of the tool may vary provided the gage will rest securely on the base without being susceptible to rocking, and the edge which contacts the cup during adjustment is straight, at least 10 mm (3/8 in.) wide, and without bevel or radius.



DIMENSIONS IN MILLIMETRES

FIG. 3 Height of Drop Gage

6.4 *Containers*—Small corrosion-resistant containers with snug-fitting lids for water content specimens. Aluminum or stainless steel cans 2.5 cm (1 in.) high by 5 cm (2 in.) in diameter are appropriate.

6.5 *Balance*—A balance readable to at least 0.01 g and having an accuracy of 0.03 g within three standard deviations within the range of use. Within any 15-g range, a difference between readings shall be accurate within 0.01 g (Notes 4 and 5).

NOTE 4—See Methods E 898 and E 319 for an explanation of terms relating to balance performance.

NOTE 5—For frequent use, a top-loading type balance with automatic load indication, readable to 0.01 g, and having an index of precision (standard deviation) of 0.003 or better is most suitable for this method. However, nonautomatic indicating equal-arm analytical balances and some small equal arm top pan balances having readabilities and sensitivities of 0.002 g or better provide the required accuracy when used with a weight set of ASTM Class 4 (National Bureau of Standards Class P) or better. Ordinary commercial and classroom type balances such as beam balances are not suitable for this method.

6.6 *Storage Container*—A container in which to store the prepared soil specimen that will not contaminate the specimen in any way, and which prevents moisture loss. A porcelain, glass, or plastic dish about 11.4 cm (4½ in.) in diameter and a plastic bag large enough to enclose the dish and be folded over is adequate.

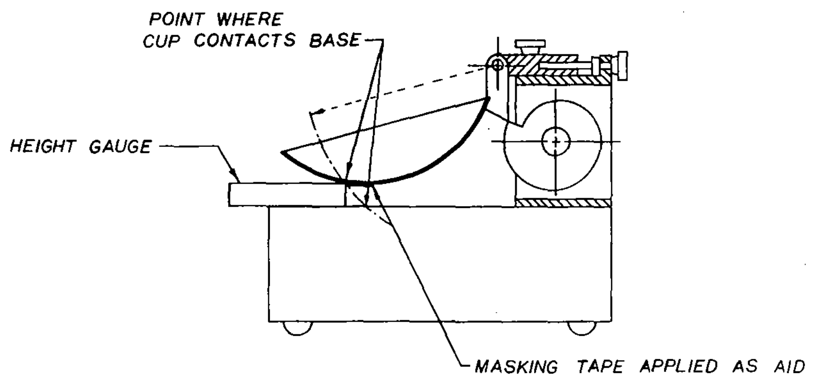


FIG. 4 Calibration for Height of Drop

6.7 *Ground Glass Plate*—A ground glass plate at least 30 cm (12 in.) square by 1 cm (3/8 in.) thick for mixing soil and rolling plastic limit threads.

6.8 *Spatula*—A spatula or pill knife having a blade about 2 cm (3/4 in.) wide by about 10 cm (4 in.) long. In addition, a spatula having a blade about 2.5 cm (1 in.) wide and 15 cm (6 in.) long has been found useful for initial mixing of samples.

6.9 *Sieve*—A 20.3 cm (8 in.) diameter, 425-μm (No. 40) sieve conforming to the requirements of Specification E 11 and having a rim at least 5 cm (2 in.) above the mesh. A 2-mm (No. 10) sieve meeting the same requirements may also be needed.

6.10 *Wash Bottle*, or similar container for adding controlled amounts of water to soil and washing fines from coarse particles.

6.11 *Drying Oven*—A thermostatically controlled oven, preferably of the forced-draft type, capable of continuously maintaining a temperature of 110 ± 5°C throughout the drying chamber. The oven shall be equipped with a thermometer of suitable range and accuracy for monitoring oven temperature.

6.12 *Washing Pan*—A round, flat-bottomed pan at least 7.6 cm (3 in.) deep, slightly larger at the bottom than a 20.3-cm (8-in.) diameter sieve.

6.13 *Rod* (optional)—A metal or plastic rod or tube 3.2 mm (1/8 in.) in diameter and about 10 cm (4 in.) long for judging the size of plastic limit threads.

7. Materials

7.1 A supply of distilled or demineralized water.

8. Sampling

8.1 Samples may be taken from any location that satisfies testing needs. However, Methods C 702, Practice D 75, and Recommended Practice D 420 should be used as guides for selecting and preserving samples from various types of sampling operations. Samples which will be prepared using the wet preparation procedure, 10.1, must be kept at their natural water content prior to preparation.

8.2 Where sampling operations have preserved the natural stratification of a sample, the various strata must be kept separated and tests performed on the particular stratum of interest with as little contamination as possible from other strata. Where a mixture of materials will be used in construction, combine the various components in such proportions that the resultant sample represents the actual construction case.

8.3 Where data from this test method are to be used for correlation with other laboratory or field test data, use the same material as used for these tests where possible.

8.4 Obtain a representative portion from the total sample sufficient to provide 150 to 200 g of material passing the 425-μm (No. 40) sieve. Free flowing samples may be reduced by the methods of quartering or splitting. Cohesive samples shall be mixed thoroughly in a pan with a spatula, or scoop and a representative portion scooped from the total mass by making one or more sweeps with a scoop through the mixed mass.

9. Calibration of Apparatus

9.1 *Inspection of Wear:*

9.1.1 *Liquid Limit Device*—Determine that the liquid limit device is clean and in good working order. The following specific points should be checked:

9.1.1.1 *Wear of Base*—The spot on the base where the cup makes contact should be worn no greater than 10 mm (3/8 in.) in diameter. If the wear spot is greater than this, the base can be machined to remove the worn spot provided the resurfacing does not make the base thinner than specified in 6.1 and the other dimensional relationships are maintained.

9.1.1.2 *Wear of Cup*—The cup must be replaced when the grooving tool has worn a depression in the cup 0.1 mm (0.004 in.) deep or when the edge of the cup has been reduced to half its original thickness. Verify that the cup is firmly attached to the cup hanger.

9.1.1.3 *Wear of Cup Hanger*—Verify that the cup hanger pivot does not bind and is not worn to an extent that allows more than 3-mm (1/8-in.) side-to-side movement of the lowest point on the rim.

9.1.1.4 *Wear of Cam*—The cam shall not be worn to an extent that the cup drops before the cup hanger (cam follower) loses contact with the cam.

9.1.2 *Grooving Tools*—Inspect grooving tools for wear on a frequent and regular basis. The rapidity of wear depends on the material from which the tool is made and the types of soils being tested. Sandy soils cause rapid wear of grooving tools; therefore, when testing these materials, tools should be inspected more frequently than for other soils. Any tool with a tip width greater than 2.1 mm must not be used. The depth of the tip of the grooving tool must be 7.9 to 8.1 mm.

NOTE 6—The width of the tip of grooving tools is conveniently checked using a pocket-sized measuring magnifier equipped with a millimetre scale. Magnifiers of this type are available from most laboratory supply companies. The depth of the tip of grooving tools can be checked using the depth measuring feature of vernier calipers.

9.2 *Adjustment of Height of Drop*—Adjust the height of drop of the cup so that the point on the cup that comes in contact with the base rises to a height of 10 ± 0.2 mm. See Fig. 4 for proper location of the gage relative to the cup during adjustment.

NOTE 7—A convenient procedure for adjusting the height of drop is as follows: place a piece of masking tape across the outside bottom of the cup parallel with the axis of the cup hanger pivot. The edge of the tape away from the cup hanger should bisect the spot on the cup that contacts the base. For new cups, placing a piece of carbon paper on the base and allowing the cup to drop several times will mark the contact spot. Attach the cup to the device and turn the crank until the cup is raised to its maximum height. Slide the height gage under the cup from the front, and observe whether the gage contacts the cup or the tape. See Fig. 4. If the tape and cup are both contacted, the height of drop is approximately correct. If not, adjust the cup until simultaneous contact is made. Check adjustment by turning the crank at 2 revolutions per second while holding the gage in position against the tape and cup. If a ringing or clicking sound is heard without the cup rising from the gage, the adjustment is correct. If no ringing is heard or if the cup rises from the gage, readjust the height of drop. If the cup rocks on the gage during this checking operation, the cam follower pivot is excessively worn and the worn parts should be replaced. Always remove tape after completion of adjustment operation.

MULTIPOINT LIQUID LIMIT—PROCEDURES A AND B

10. Preparation of Test Specimens

10.1 *Wet Preparation*—Except where the dry method of specimen preparation is specified (10.2), prepare specimens for test as described in the following sections.

10.1.1 *Samples Passing the 425-µm (No. 40) Sieve*—When by visual and manual procedures it is determined that the sample has little or no material retained on a 425-µm (No. 40) sieve, prepare a specimen of 150 to 200 g by mixing thoroughly with distilled or demineralized water on the glass plate using the spatula. If desired, soak soil in a storage dish with small amount of water to soften the soil before the start of mixing. Adjust the water content of the soil to bring it to a consistency that would require 25 to 35 blows of the liquid limit device to close the groove (Note 8). If, during mixing, a small percentage of material is encountered that would be retained on a 425-µm (No. 40) sieve, remove these particles by hand, if possible. If it is impractical to remove the coarser material by hand, remove small percentages (less than about 15 %) of coarser material by working the specimen through a 425-µm (No. 40) sieve using a piece of rubber sheeting, rubber stopper, or other convenient device provided the operation does not distort the sieve or degrade material that would be retained if the washing method described in 10.1.2 were used. If larger percentages of coarse material are encountered during mixing, or it is considered impractical to remove the coarser material by the methods just described, wash the sample as described in 10.1.2. When the coarse particles found during mixing are concretions, shells, or other fragile particles, do not crush these particles to make them pass a 425-µm (No. 40) sieve, but remove by hand or by washing. Place the mixed soil in the storage dish, cover to prevent loss of moisture, and allow to stand for at least 16 h (overnight). After the standing period and immediately before starting the test, thoroughly remix the soil.

NOTE 8—The time taken to adequately mix a soil will vary greatly, depending on the plasticity and initial water content. Initial mixing times of more than 30 min may be needed for stiff, fat clays.

10.1.2 *Samples Containing Material Retained on a 425-µm (No. 40) Sieve:*

10.1.2.1 Select a sufficient quantity of soil at natural water content to provide 150 to 200 g of material passing the 425-µm (No. 40) sieve. Place in a pan or dish and add sufficient water to cover the soil. Allow to soak until all lumps have softened and the fines no longer adhere to the surfaces of the coarse particles (Note 9).

NOTE 9—In some cases, the cations of salts present in tap water will exchange with the natural cations in the soil and significantly alter the test results should tap water be used in the soaking and washing operations. Unless it is known that such cations are not present in the tap water, distilled or demineralized water should be used. As a general rule, water containing more than 100 mg/L of dissolved solids should not be used for washing operations.

10.1.2.2 When the sample contains a large percentage of material retained on the 425-µm (No. 40) sieve, perform the following washing operation in increments, washing no more than 0.5 kg (1 lb) of material at one time. Place the 425-µm (No. 40) sieve in the bottom of the clean pan. Pour the soil water mixture onto the sieve. If gravel or coarse sand particles are present, rinse as many of these as possible with small quantities of water from a wash bottle, and discard. Alternatively, pour the soil water mixture over a 2-mm (No. 10) sieve nested atop the 425-µm (No. 40) sieve, rinse the fine material through and remove the 2-mm (No. 10) sieve. After washing and removing as much of the coarser material as possible, add sufficient water to the pan to bring the level to about 13 mm (½ in.) above the surface of the 425-µm (No. 40) sieve. Agitate the slurry by stirring with the fingers while raising and lowering the sieve in the pan and swirling the suspension so that fine material is washed from the coarser particles. Disaggregate fine soil lumps that have not slaked by gently rubbing them over the sieve with the fingertips. Complete the washing operation by raising the sieve above the water surface and rinsing the material retained with a small amount of clean water. Discard material retained on the 425-µm (No. 40) sieve.

10.1.2.3 Reduce the water content of the material passing the 425-µm (No. 40) sieve until it approaches the liquid limit. Reduction of water content may be accomplished by one or a combination of the following methods: (*a*) exposing the air currents at ordinary room temperature, (*b*) exposing to warm air currents from a source such as an electric hair dryer, (*c*) filtering in a Büchner funnel or using filter candles, (*d*) decanting clear water from surface of suspension, or (*e*) draining in a colander or plaster of paris dish lined with high retentivity, high wet-strength filter paper.[7] If a plaster of paris dish is used, take care that the dish never becomes sufficiently saturated that it fails to actively absorb water into its surface. Thoroughly dry dishes between uses. During evaporation and cooling, stir the sample often enough to prevent

[7] S and S 595 filter paper available in 32-cm circles, has proven satisfactory.

overdrying of the fringes and soil pinnacles on the surface of the mixture. For soil samples containing soluble salts, use a method of water reduction such as *a* or *b* that will not eliminate the soluble salts from the test specimen.

10.1.2.4 Thoroughly mix the material passing the 425-μm (No. 40) sieve on the glass plate using the spatula. Adjust the water content of the mixture, if necessary, by adding small increments of distilled or demineralized water or by allowing the mixture to dry at room temperature while mixing on the glass plate. The soil should be at a water content that will result in closure of the groove in 25 to 35 blows. Return the mixed soil to the mixing dish, cover to prevent loss of moisture, and allow to stand for at least 16 h. After the standing period, and immediately before starting the test, remix the soil thoroughly.

10.2 *Dry Preparation:*

10.2.1 Select sufficient soil to provide 150 to 200 g of material passing the 425-μm (No. 40) sieve after processing. Dry the sample at room temperature or in an oven at a temperature not exceeding 60°C until the soil clods will pulverize readily. Disaggregation is expedited if the sample is not allowed to completely dry. However, the soil should have a dry appearance when pulverized. Pulverize the sample in a mortar with a rubber tipped pestle or in some other way that does not cause breakdown of individual grains. When the coarse particles found during pulverization are concretions, shells, or other fragile particles, do not crush these particles to make them pass a 425-μm (No. 40) sieve, but remove by hand or other suitable means, such as washing.

10.2.2 Separate the sample on a 425-μm (No. 40) sieve, shaking the sieve by hand to assure thorough separation of the finer fraction. Return the material retained on the 425-μm (No. 40) sieve to the pulverizing apparatus and repeat the pulverizing and sieving operations as many times as necessary to assure that all finer material has been disaggregated and material retained on the 425-μm (No. 40) sieve consists only of individual sand or gravel grains.

10.2.3 Place material remaining on the 425-μm (No. 40) sieve after the final pulverizing operations in a dish and soak in a small amount of water. Stir the soil water mixture and pour over the 425-μm (No. 40) sieve, catching the water and any suspended fines in the washing pan. Pour this suspension into a dish containing the dry soil previously sieved through the 425-μm (No. 40) sieve. Discard material retained on the 425-μm (No. 40) sieve.

10.2.4 Adjust the water content as necessary by drying as described in 10.1.2.3 or by mixing on the glass plate, using the spatula while adding increments of distilled or demineralized water, until the soil is at a water content that will result in closure of the groove in 25 to 35 blows.

10.2.5 Put soil in the storage dish, cover to prevent loss of moisture and allow to stand for at least 16 h. After the standing period, and immediately before starting the test, thoroughly remix the soil (Note 8).

11. Procedure

11.1 Place a portion of the prepared soil in the cup of the liquid limit device at the point where the cup rests on the base, squeeze it down, and spread it into the cup to a depth of about 10 mm at its deepest point, tapering to form an approximately horizontal surface. Take care to eliminate air bubbles from the soil pat but form the pat with as few strokes as possible. Heap the unused soil on the glass plate and cover with the inverted storage dish or a wet towel.

11.2 Form a groove in the soil pat by drawing the tool, beveled edge forward, through the soil on a line joining the highest point to the lowest point on the rim of the cup. When cutting the groove, hold the grooving tool against the surface of the cup and draw in an arc, maintaining the tool perpendicular to the surface of the cup throughout its

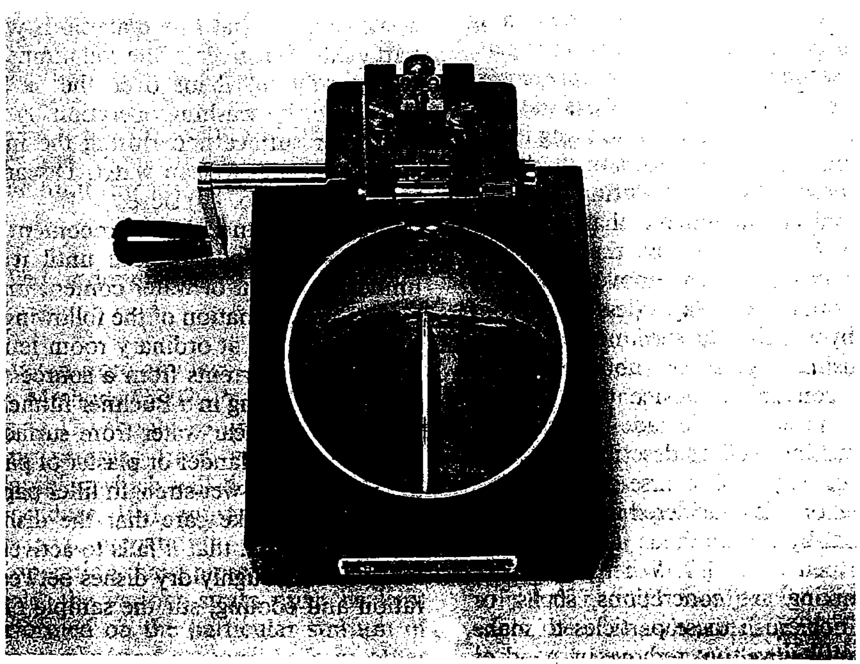


FIG. 5 Grooved Soil Pat in Liquid Limit Device

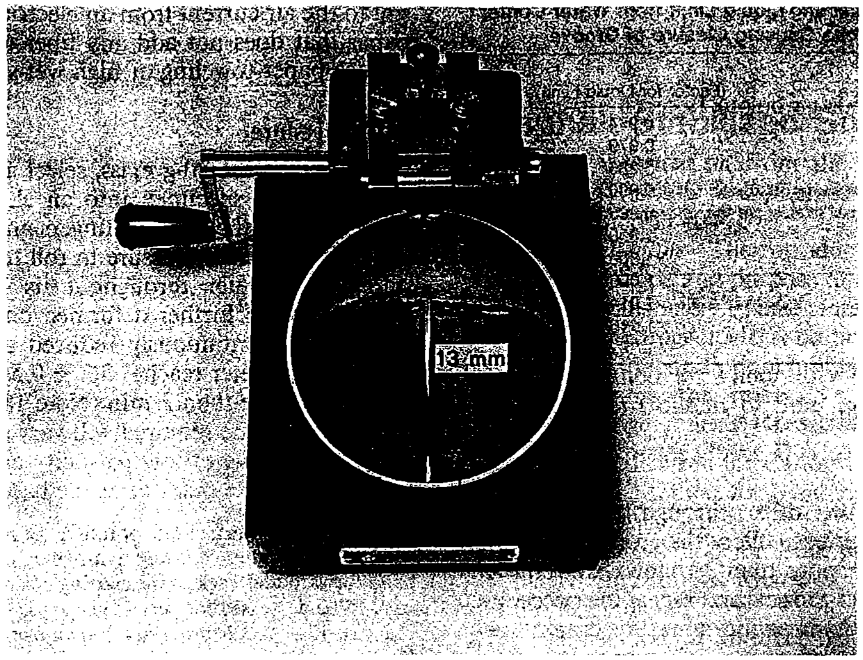


FIG. 6 Soil Pat After Groove Has Closed

movement. See Fig. 5. In soils where a groove cannot be made in one stroke without tearing the soil, cut the groove with several strokes of the grooving tool. Alternatively, cut the groove to slightly less than required dimensions with a spatula and use the grooving tool to bring the groove to final dimensions. Exercise extreme care to prevent sliding the soil pat relative to the surface of the cup.

11.3 Verify that no crumbs of soil are present on the base or the underside of the cup. Lift and drop the cup by turning the crank at a rate of 1.9 to 2.1 drops per second until the two halves of the soil pat come in contact at the bottom of the groove along a distance of 13 mm (½ in.). See Fig. 6.

NOTE 10—Use the end of the grooving tool, Fig. 2, or a scale to verify that the groove has closed 13 mm (½ in.).

11.4 Verify that an air bubble has not caused premature closing of the groove by observing that both sides of the groove have flowed together with approximately the same shape. If a bubble has caused premature closing of the groove, reform the soil in the cup, adding a small amount of soil to make up for that lost in the grooving operation and repeat 11.1 to 11.3. If the soil slides on the surface of the cup, repeat 11.1 through 11.3 at a higher water content. If, after several trials at successively higher water contents, the soil pat continues to slide in the cup or if the number of blows required to close the groove is always less than 25, record that the liquid limit could not be determined, and report the soil as nonplastic without performing the plastic limit test.

11.5 Record the number of drops, N, required to close the groove. Remove a slice of soil approximately the width of the spatula, extending from edge to edge of the soil cake at right angles to the groove and including that portion of the groove in which the soil flowed together, place in a weighed container, and cover.

11.6 Return the soil remaining in the cup to the glass plate. Wash and dry the cup and grooving tool and reattach the cup to the carriage in preparation for the next trial

11.7 Remix the entire soil specimen on the glass adding distilled water to increase the water content of th and decrease the number of blows required to clos groove. Repeat 11.1 through 11.6 for at least two addit trials producing successively lower numbers of blows to the groove. One of the trials shall be for a closure requ 25 to 35 blows, one for closure between 20 and 30 blows one trial for a closure requiring 15 to 25 blows.

11.8 Determine the water content, W_N, of the soil imen from each trial in accordance with Method D Make all weighings on the same balance. Initial weig should be performed immediately after completion o test. If the test is to be interrupted for more than abo min, the specimens already obtained should be weigh the time of the interruption.

12. Calculations

12.1 Plot the relationship between the water content, and the corresponding number of drops, N, of the cup semilogarithmic graph with the water content as ordinat the arithmetical scale, and the number of drops as abs on the logarithmic scale. Draw the best straight line thr the three or more plotted points.

12.2 Take the water content corresponding to the i section of the line with the 25-drop abscissa as the li limit of the soil. Computational methods may be substi for the graphical method for fitting a straight line to the and determining the liquid limit.

ONE-POINT LIQUID LIMIT—PROCEDURES C AND I

13. Preparation of Test Specimens

13.1 Prepare the specimen in the same manner as scribed in Section 10, except that at mixing, adjust the content to a consistency requiring 20 to 30 drops of

LE 1 Factors for Obtaining Liquid Limit from Water Content and Number of Drops Causing Closure of Groove

N (Number of Drops)	K (Factor for Liquid Limit)
20	0.974
21	0.979
22	0.985
23	0.990
24	0.995
25	1.000
26	1.005
27	1.009
28	1.014
29	1.018
30	1.022

id limit cup to close the groove.

Procedure

ł.1 Proceed as described in 11.1 through 11.5 except that number of blows required to close the groove shall be 20 0. If less than 20 or more than 30 blows are required, ıst the water content of the soil and repeat the procedure.

ł.2 Immediately after removing a water content specn as described in 11.5, reform the soil in the cup, adding nall amount of soil to make up for that lost in the ›ving and water content sampling operations. Repeat : through 11.5, and, if the second closing of the groove ıires the same number of drops or no more than two ›s difference, secure another water content specimen. ɜrwise, remix the entire specimen and repeat.

ɔTE 11—Excessive drying or inadequate mixing will cause the ber of blows to vary.

ł.3 Determine water contents of specimens as described 1.8.

Calculations

5.1 Determine the liquid limit for each water content imen using one of the following equations:

$$LL = W_N\left(\frac{N}{25}\right)^{0.121} \quad \text{or}$$

$$LL = K(W_N)$$

re:

= the number of blows causing closure of the groove at water content,

= water content, and

= a factor given in Table 1.

he liquid limit is the average of the two trial liquid limit es.

5.2 If the difference between the two trial liquid limit es is greater than one percentage point, repeat the test.

PLASTIC LIMIT

Preparation of Test Specimen

5.1 Select a 20-g portion of soil from the material ›ared for the liquid limit test, either after the second ing before the test, or from the soil remaining after pletion of the test. Reduce the water content of the soil consistency at which it can be rolled without sticking to ıands by spreading and mixing continuously on the glass ɜ. The drying process may be accelerated by exposing the soil to the air current from an electric fan, or by blotting with paper that does not add any fiber to the soil, such as hard surface paper toweling or high wet-strength filter paper.

17. Procedure

17.1 From the 20-g mass, select a portion of 1.5 to 2.0 g. Form the test specimen into an ellipsoidal mass. Roll this mass between the palm or fingers and the ground-glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length (Note 12). The thread shall be further deformed on each stroke so that its diameter is continuously reduced and its length extended until the diameter reaches 3.2 ± 0.5 mm (0.125 ± .020 in.), taking no more than 2 min (Note 13). The amount of hand or finger pressure required will vary greatly, according to the soil. Fragile soils of low plasticity are best rolled under the outer edge of the palm or at the base of the thumb.

NOTE 12—A normal rate of rolling for most soils should be 80 to 90 strokes per minute, counting a stroke as one complete motion of the hand forward and back to the starting position. This rate of rolling may have to be decreased for very fragile soils.

NOTE 13—A 3.2-mm (⅛-in.) diameter rod or tube is useful for frequent comparison with the soil thread to ascertain when the thread has reached the proper diameter, especially for inexperienced operators.

17.1.1 When the diameter of the thread becomes 3.2 mm, break the thread into several pieces. Squeeze the pieces together, knead between the thumb and first finger of each hand, reform into an ellipsoidal mass, and reroll. Continue this alternate rolling to a thread 3.2 mm in diameter, gathering together, kneading and rerolling, until the thread crumbles under the pressure required for rolling and the soil can no longer be rolled into a 3.2-mm diameter thread (See Fig. 7). It has no significance if the thread breaks into threads of shorter length. Roll each of these shorter threads to 3.2 mm in diameter. The only requirement for continuing the test is that they are able to be reformed into an ellipsoidal mass and rolled out again. The operator shall at no time attempt to produce failure at exactly 3.2 mm diameter by allowing the thread to reach 3.2 mm, then reducing the rate of rolling or the hand pressure, or both, while continuing the rolling without further deformation until the thread falls apart. It is permissible, however, to reduce the total amount of deformation for feebly plastic soils by making the initial diameter of the ellipsoidal mass nearer to the required 3.2-mm final diameter. If crumbling occurs when the thread has a diameter greater than 3.2 mm, this shall be considered a satisfactory end point, provided the soil has been previously rolled into a thread 3.2 mm in diameter. Crumbling of the thread will manifest itself differently with the various types of soil. Some soils fall apart in numerous small aggregations of particles, others may form an outside tubular layer that starts splitting at both ends. The splitting progresses toward the middle, and finally, the thread falls apart in many small platy particles. Fat clay soils require much pressure to deform the thread, particularly as they approach the plastic limit. With these soils, the thread breaks into a series of barrel-shaped segments about 3.2 to 9.5 mm (⅛ to ⅜ in.) in length.

17.2 Gather the portions of the crumbled thread together and place in a weighed container. Immediately cover the container.

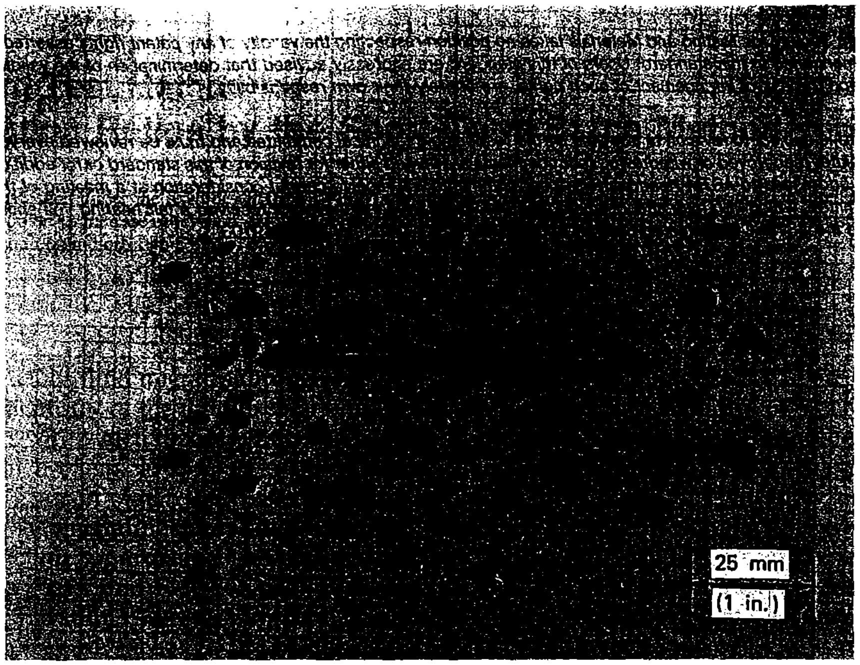


FIG. 7 Lean Clay Soil at the Plastic Limit

17.3 Select another 1.5 to 2.0 g portion of soil from the original 20-g specimen and repeat the operations described in 17.1 and 17.2 until the container has at least 6 g of soil.

17.4 Repeat 17.1 through 17.3 to make another container holding at least 6 g of soil. Determine the water content, in percent, of the soil contained in the containers in accordance with Method D 2216. Make all weighings on the same balance.

NOTE 14—The intent of performing two plastic limit trials is to verify the consistency of the test results. It is acceptable practice to perform only one plastic limit trial when the consistency in the test results can be confirmed by other means.

18. Calculations

18.1 Compute the average of the two water contents. If the difference between the two water contents is greater than two percentage points, repeat the test. The plastic limit is the average of the two water contents.

PLASTICITY INDEX

19. Calculations

19.1 Calculate the plasticity index as follows:

$$PI = LL - PL$$

where:
LL = the liquid limit,
PL = the plastic limit.

Both LL and PL are whole numbers. If either the liquid limit or plastic limit could not be determined, or if the plastic limit is equal to or greater than the liquid limit, report the soil as nonplastic, NP.

20. Report

20.1 Report the following information:

20.1.1 Sample identifying information,

20.1.2 Any special specimen selection process used, such as removal of sand lenses from undisturbed sample,

20.1.3 Report sample as airdried if the sample was airdried before or during preparation,

20.1.4 Liquid limit, plastic limit, and plasticity index to the nearest whole number and omitting the percent designation. If the liquid limit or plastic limit tests could not be performed, or if the plastic limit is equal to or greater than the liquid limit, report the soil as nonplastic, NP,

20.1.5 An estimate of the percentage of sample retained on the 425-µm (No. 40) sieve, and

20.1.6 Procedure by which liquid limit was performed, if it differs from the multipoint method.

21. Precision and Bias

21.1 No interlaboratory testing program has as yet been conducted using this test method to determine multilaboratory precision.

21.2 The within laboratory precision of the results of tests performed by different operators at one laboratory on two soils using Procedure A for the liquid limit is shown in Table 2.

TABLE 2 Within Laboratory Precision for Liquid Limit

	Average Value, $\bar{x}$	Standard Deviation, s
Soil A:		
PL	21.9	1.07
LL	27.9	1.07
Soil B:		
PL	20.1	1.21
LL	32.6	0.98

Standard Test Method for Measurement of Soil Potential (Suction) Using Filter Paper[1]

1. Scope

1.1 This test method covers the use of laboratory filter papers as passive sensors to evaluate the soil matric (matrix) and total potential (suction), a measure of the free energy of the pore-water or tension stress exerted on the pore-water by the soil matrix (**1, 2**).[2] The term potential or suction is descriptive of the energy status of soil water.

1.2 This test method controls the variables for measurement of the water content of filter paper that is in direct contact with soil or in equilibrium with the partial pressure of water vapor in the air of an airtight container enclosing a soil specimen. The partial pressure of water vapor in the air is assumed to be in equilibrium with the vapor pressure of pore-water in the soil specimen.

1.3 This test method provides a procedure for calibrating different types of filter paper for use in evaluating soil matric and total potential.

1.4 The values stated in SI units are to be regarded as the standard. The inch-pound units given in parentheses are approximate and for information only.

1.5 *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 114 Test Methods for Chemical Analysis of Hydraulic Cement[3]

D 653 Terminology Relating to Soil, Rock, and Contained Fluids[4]

D 1125 Test Method for Electrical Conductivity and Resistivity of Water[5]

D 2216 Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock[4]

D 2325 Test Method for Capillary-Moisture Relationships for Coarse and Medium-Textured Soils by Porous-Plate Apparatus[4]

D 3152 Test Method for Capillary-Moisture Relationships for Fine-Textured Soils by Pressure-Membrane Apparatus[4]

D 4542 Test Method for Pore-Water Extraction and Determination of the Solute Salt Content of Soils by Refractometer[4]

D 4753 Specification for Evaluating, Selecting, and Specifying Balances and Scales for Use in Soil and Rock Testing[4]

E 337 Test Method for Measuring Humidity With a Psychrometer (the Measurement of Wet- and Dry-Bulb Temperatures)[6]

E 832 Specification for Laboratory Filter Papers[6]

3. Terminology

3.1 *Definitions:*

3.1.1 Refer to Terminology D 653 for definitions of terms applicable to this test method.

3.2 *Descriptions of Terms Specific to This Standard:*

3.2.1 *atmosphere*—a unit of pressure equal to 76 cm mercury or 101 kPa at 0°C.

3.2.2 *matric (matrix) suction, hm (kPa)*—the negative pressure (expressed as a positive value), relative to ambient atmospheric pressure on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water; pressure equivalent to that measured by Test Methods D 2325 and D 3152. Matric suction is also the decrease in relative humidity due to the difference in air and water pressure across the water surface; the relative humidity or water vapor pressure decreases as the radius of curvature of the water surface decreases. The term "matric" is grammatically correct, while matrix is commonly used in the civil engineering literature.

3.2.3 *molality, moles/1000 g*—number of moles of solute per 1000 g of solvent.

3.2.4 *mole, n*—molecular weight of a substance in grams.

3.2.5 *osmotic (solute) suction, hs (kPa)*—the negative pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the soil water; decrease in relative humidity due to the presence of dissolved salts in pore-water.

3.2.6 *pF*—a unit of negative pressure expressed as the logarithm to the base 10 of the height in centimeters that a column of water will rise by capillary action or negative gage pressure (Mg/m^2) divided by the unit weight of water (Mg/m^3) times 1000. pF ≈ 3 + logarithm to the base 10 of the negative pressure in atmospheres. Refer to capillary head or capillary rise in Terminology D 653.

3.2.7 *soil relative humidity, R_h*—the ratio of the vapor

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.04 on Hydrologic Properties of Soil and Rocks.

Current edition approved Sept. 15, 1992. Published November 1992.

[2] The boldface numbers given in parentheses refer to a list of references at the end of the text.

[3] *Annual Book of ASTM Standards*, Vol 04.01.

[4] *Annual Book of ASTM Standards*, Vol 04.08.

[5] *Annual Book of ASTM Standards*, Vol 11.01.

[6] *Annual Book of ASTM Standards*, Vol 15.09.

pressure of pore water in the soil to the vapor pressure of free pure water. Relative humidity in the soil is defined as relative humidity measured by Test Method E 337.

3.2.8 *total potential (kPa)*—the sum of gravitational, pressure, osmotic, and external gas potentials. Potential may be identified with suction when gravitational and external gas potentials are neglected.

3.2.9 *total soil suction, h (kPa)*—the negative pressure, relative to the external gas pressure on the soil water, to which a pool of pure water must be subjected to be in equilibrium with the soil water through a semipermeable membrane that is permeable to water molecules only. Total soil suction (expressed as a positive value) is the sum of osmotic (solute) and matric (matrix) suctions.

3.2.10 *vapor pressure of free pure water (kPa)*—the saturation vapor pressure of free pure water at a given dry-bulb temperature.

3.2.11 *vapor pressure of pore-water in soil (kPa)*—the partial pressure of water vapor that is in equilibrium with pore-water in soil at a given dry-bulb temperature.

4. Summary of Test Method

4.1 Filter papers are placed in an airtight container with a specimen for seven days to allow sufficient time for the vapor pressure of pore-water in the specimen, vapor pressure of pore water in the filter paper, and partial vapor pressure of water in the air inside the container to reach equilibrium. The mass of the filter papers is subsequently determined and the suction of the specimen is determined from a calibration relationship of the filter paper water content with suction applicable to the type of filter paper and the test procedure of this test method.

5. Significance and Use

5.1 Soil suction is a measure of the free energy of the pore-water in a soil. Soil suction in practical terms is a measure of the affinity of soil to retain water and can provide information on soil parameters that are influenced by the soil water; for example, volume change, deformation, and strength characteristics of the soil.

5.2 Soil suction is related with soil water content through water retention characteristic curves (see Test Method D 2325). Soil water content may be found from Test Method D 2216.

5.3 Measurements of soil suction may be used with other soil and environmental parameters to evaluate hydrologic processes (1) and to evaluate the potential for heave or shrinkage, shear strength, modulus, in situ stress, and hydraulic conductivity of unsaturated soils.

5.4 The filter paper method of evaluating suction is simple and economical with a range from 10 to 100 000 kPa (0.1 to 1000 bars).

6. Apparatus

6.1 *Filter Paper*—The paper used must be ash-free quantitative Type II filter paper, in accordance with Specification E 832; for example, Whatman No. 42, Fisherbrand 9-790A, or Schleicher and Schuell No. 589 White Ribbon. A suitable diameter is 5.5 cm (2.2 in.).

6.2 *Specimen Container*, 115 to 230 g (4 to 8 oz) capacity metal or glass (rust free) container and lid (for example, coated with zinc chromate to retard rusting) to contain the specimen and filter papers. The inside of these containers may also be coated with wax to retard rusting.

6.3 *Filter Paper Container*—This container holds filter paper following the equilibration of suction and removal from the specimen container.

6.3.1 *Metal Container Alternate*, two nominal 60 g (2 oz) capacity metal moisture containers (aluminum or stainless) with lids to dry the filter paper. The containers should be numbered by imprinting with a metal stamp. The containers should not be written on with any type of marker or labelled in any manner. Throw-away vinyl surgical non-powdered or similar gloves should be used anytime the small containers designated for filter paper measurements are handled to prevent body oils from influencing any mass measurements made prior to handling.

6.3.2 *Plastic Bag Alternate*—Plastic bag large enough to accommodate the filter paper disks (approximately 50 mm in dimension) capable of an airtight seal.

6.4 *Insulated Chest*—A box of approximately 0.03 m^3 (1 ft^3) capacity insulated with foamed polystyrene or other material capable of maintaining temperature within ±1°C when external temperatures vary ±3°C.

6.5 *Balance*—A balance or scale having a minimum capacity of 20 g and meeting the requirements of 4.2.1.1 of Specification C 114, for a balance of 0.0001 g readability. In addition, balances for performance of Test Method D 2216, meeting requirements of Specification D 4753.

6.6 *Drying Oven*, thermostatically-controlled, preferably of the forced-draft type, and capable of maintaining a uniform temperature of 110 ± 5°C throughout the drying chamber and meeting requirements of Test Method D 2216.

6.7 *Metal Block*—A metal block > 500 g mass with a flat surface to hasten cooling of the metal tare cans.

6.8 *Thermometer*—An instrument to determine the temperature of the tested soil to an accuracy of ±1°C.

6.9 *Miscellaneous Equipment*, tweezers, trimming knife, flexible plastic electrical tape, O-rings, screen wire, brass discs, etc. Tweezers should be at least 110 mm (4.5 in.) in length.

7. Calibration

7.1 Obtain a calibration curve applicable to a specific filter paper by following the procedure in Section 8, except for replacing the soil specimen with salt solutions such as reagent grade potassium chloride or sodium chloride of known molality in distilled water.

7.1.1 Suspend the filter paper above at least 50 cc of a salt solution in the specimen container, see 6.2, by placing it on an improvised platform made of inert material such as plastic tubing or stainless steel screen.

7.1.2 Calculate the suction of the filter paper from the relative humidity of the air above the solution by the following:

$$h = \frac{RT}{v} \cdot \ln R_h \qquad (1)$$

where:
h = suction, kPa,
R = ideal gas constant, 8.31432 Joules/mole·K,
T = absolute temperature, degrees kelvin (K),
v = volume of a mole of liquid water, 0.018 kilomoles/m^3, and
R_h = relative humidity, fraction.

7.1.3 Use standard critical tables to evaluate the relative humidity of water in equilibrium with the salt solution as illustrated in Table 1. Refer to Test Method E 337 for further information on relative humidity.

7.2 Typical calibration curves for filter papers (for example, Whatman No. 42, Schleicher and Schuell No. 589), see Fig. 1, consists of two parts. The upper segment represents moisture retained as films adsorbed to particle surfaces, while the lower segment represents moisture retained by capillary or surface tension forces between particles. The filter paper water content break point is w_f = 45.3 % for Whatman No. 42 (3, 4) and w_f = 54 % for Schleicher and Schuell No. 589 (2, 4).

7.3 The calibration curves in Fig. 1 are applicable to total suction (2, 5). Variability in results is less than 2 % of the suction above 100 kPa. Soil disturbance has minimal influence on suction above 20 kPa. At moisture contents with suctions less than 20 kPa, sample disturbance increases variability of measurement (2, 4). The right vertical axis of Fig. 1 provides the suction in units pF and atmospheres pressure; for example, h = 2 log atmospheres is a suction of 100 atmospheres, while pF = 5 or 100 000 cm water.

NOTE 1—Filter paper may be calibrated by using the pressure membrane, Test Method D 3152 for the range 100 to 1500 kPa (1 to 15 atm), and the ceramic plate, Test Method D 2325 for the range 10 to 100 kPa (0.1 to 1 atm).

8. Procedure

8.1 *Filter Paper Preparation*—Dry filter papers selected for testing at least 16 h or overnight in the drying oven. Place filter papers in a desiccant jar over desiccant after drying for storage until use.

8.2 *Measurement of Suction*—Total suction will be measured if filter papers are not in contact with the soil specimen. Moisture transfer will be limited to vapor transfer through the air inside the specimen container. Matric suction will be measured if the filter paper is in physical contact with the soil. Physical contact between the soil and filter paper allows fluid transfer including transfer of salts that may be dissolved in the pore water.

TABLE 1 Salt Solution Concentrations for Evaluating Soil Suction

kPa	log kPa	pF	atm	R_h	20°C g NaCl / 1000 mL water	20°C g KCl / 1000 mL water
−98	1.99	3.0	−0.97	0.99927	1.3	1.7
−310	2.49	3.5	−3.02	0.99774	3.8	5.3
−980	2.99	4.0	−9.68	0.99278	13.1	17.0
−3099	3.49	4.5	−30.19	0.97764	39.0	52.7
−9800	3.99	5.0	−96.77	0.93008	122.5	165.0

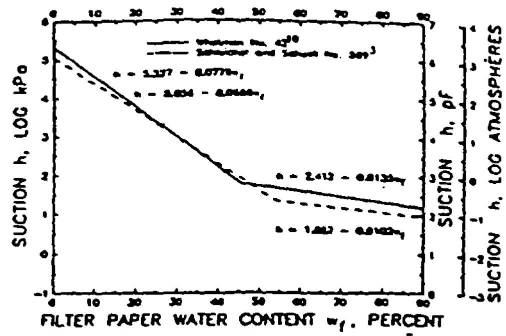


FIG. 1 Calibration Suction-Water Content Curves for Wettin Filter Paper (3) (Coefficient of Determination r > 0.99)

NOTE 2—When the soil is not sufficiently moist, adequate phy contact between the filter paper and soil may not always be poss This can cause an inaccurate measure of matric suction. Matric suc may be inferred by subtracting the osmotic suction from the t suction. The osmotic suction may be determined by measuring electrical conductivity (see Test Method D 1125) of pore-water extra from the soil using a pore fluid squeezer (6) or using Test Met D 4542; a calibration curve (7) may be used to relate the electr conductivity to the osmotic suction.

8.3 *Filter Paper Placement*—Place an intact soil specim or fragments of a soil sample, 115 to 230 g mass, in t specimen container. The soil specimen should nearly fill t specimen container to reduce equilibration time and minimize suction changes in the specimen.

8.3.1 *Measurement of Total Suction*—Remove two filt papers from the desiccator and immediately place over t specimen, but isolate from the specimen by inserting scre wire, O-rings, or other inert item with minimal surface ar between the filter papers and the soil, see Fig. 2(*a*). A filt paper edge should be bent up or offset slightly to hasten lat removal of the filter paper from these large containers wit tweezers, see 8.6.

8.3.2 *Measurement of Matric Suction*—Place thre stacked filter papers in contact with the soil specimen, se Fig. 2(*b*). The outer filter papers prevent soil contaminatio of the center filter paper used for analysis of the matri suction. The outer filter papers should be slightly larger i diameter than the center filter paper. This can be accomplished by cutting the center paper so that the diameter is at least 3 to 4 mm smaller than the outer filter papers. This will help prevent direct soil contact with the center filter paper.

8.4 *Equilibrating Suction*—Put the lid of the specimen container in place and seal with at least one wrapping of plastic electrical tape. Then place the sealed container in an insulted chest and place in a location with temperature variations less than 3°C. A typical nominal temperature is 20°C. The suction of the filter paper and the specimen in the container should be allowed to come to equilibration for a minimum of seven days.

NOTE 3—If filter papers are placed with soil specimens while in the field, the filter papers should be oven dried overnight then stored in an airtight container over desiccant to minimize moisture in the filter paper. Moisture in the filter paper prior to testing expands the fibers and alters the filter paper void space that may lead to a change in the calibration curve of the filter paper. The insulated chest while in the field should be kept in the shade during hot summer days and in a heated area during cold winter days. The chest with the sealed containers

RL 005238

should be placed in a temperature controlled room at about 20°C following return from the field.

NOTE 4—Equilibration of suction between the soil, filter paper, and air in the closed container is the desired result of the equilibration period. It must be recognized that the equilibration process is dependent upon the initial suction of the soil, initial relative humidity of the air, soil mass, and space in the container. The seven day period is sufficient for conditions normally involved in soil mechanics; however, under many conditions equilibration will be completed more quickly. This suction measurement must avoid condensation so thermostatic control may be necessary. Sample temperature control during equilibration will ensure that condensation effects are minimized. Storing the specimen containers containing the soil specimen and filter paper in a thermostatic box (for example, ice chest) made of polystyrene insulation and packing expanded vermiculite or similar material around the box will help minimize thermal fluctuations. It is possible to limit thermal fluctuations to ±0.01°C with such an insulation scheme.

8.5 *Predetermining Mass of Filter Paper Containers*—At the end of the equilibration period, place each of the two filter papers, if total suction is to be measured, or the center filter paper of a three-layer stack, if matrix suction is to be measured, in a separate filter paper container of predetermined mass. 1 Determine the mass to the nearest 0.0001 g, designated T_c (tare-cold), before the specimen container is removed from the insulated chest. It is suggested that the mass of the filter paper container be determined immediately prior to determining the total mass of the filter paper and filter paper container.

8.6 *Transferring the Filter Papers*— 2 Utilizing a pair of tweezers, transfer each filter paper from the specimen container into a metal container alternate or plastic bag alternate of predetermined mass (T_c). This entire process must be completed in 3 to 5 s. The key to successful measurements of filter paper water content is to minimize water loss during transfer of filter paper from the specimen container and during mass determination prior to oven drying. Observations have been made of 5 % or more mass loss due to evaporation during a 5 to 10 s exposure of the filter paper to room humidity of 30 to 50 R_h.

8.6.1 *Metal Container Alternate*— 3 Place lids loosely on metal container alternates (not ajar). Care must be taken to seal the metal container alternate after each transfer; that is, take the filter paper from the specimen container and place the filter paper into a metal container, then seal the container. Repeat this procedure for the second filter paper using the second container of predetermined mass if total suction is to be determined. The containers should be sealed as quickly as possible to ensure that ambient air does not alter the moisture condition of the soil specimen or filter papers.

8.6.2 *Plastic Bag Alternate*—Quickly transfer a filter paper to a plastic bag of predetermined initial mass and seal the bag. Repeat this procedure for additional filter papers.

8.7 *Determining Mass of Filter Paper and Filter Paper Containers*— 4 Immediately determine the mass of each of the filter paper containers with the filter papers to the nearest 0.0001 g. This mass, M_1, is

$$M_1 = M_f + M_w + T_c \tag{2}$$

where:
M_1 = total mass of filter paper container and filter paper prior to oven drying, g,
M_f = mass of dry filter paper, g,
M_w = mass of water in the filter paper, g, and
T_c = mass of the cold filter paper container, g.

8.8 *Equilibrating Temperature:*

8.8.1 *Metal Container Alternate*— 5 Place the metal filt paper containers in an oven at 110 ± 5°C with the li slightly adjar or unsealed to permit moisture to escape. Th containers should remain in the oven for a minimum of 2 . After the minimum time 6 seal the containers and leave in th oven for at least 15 min to allow temperature equilibratio Remove the tares from the oven and then determine in ma to 0.0001 g to calculate the dry total mass:

$$M_2 = M_f + T_h \tag{3}$$

where:
M_2 = dry total mass, g, and
T_h = hot container mass, g.

NOTE 5—If the filter paper containers are metal, 7 they should b placed on a metal block for approximately 30 s to cool. The metal bloc acts as a heat sink and will reduce the temperature variation durin determination of mass. Immediately remove and discard the filter pape and redetermine the mass of the filter paper container to 0.0001 g, tha is the mass of the hot container, T_h. This procedure is repeated fo additional containers.

8.8.2 *Plastic Bag Alternate*—Place the filter paper in the drying oven for a minimum of 2 h, then place in a desiccant jar over silica jel or standard desiccant to cool for a minimum of 2 to 3 min. Place in the plastic bag and determine the mass (M_2) from Eq 3. 8 Remove the filter paper and 9 determine the final mass of the plastic bag (T_h).

8.8.3 Once the masses of the dried filter papers have been determined, discard the filter papers. Under no circumstances shall oven-dried filter papers be re-used in conducting this test method.

9. Calculation

9.1 Calculate the following for each filter paper:

$$M_f = M_2 - T_h \tag{4}$$

$$M_w = M_1 - M_2 + T_h - T_c \tag{5}$$

from the measured quantities:

$$M_1, M_2, T_c \text{ and } T_h$$

NOTE 6—The hot container mass, T_h, may be consistently less than the cold tare mass, T_c, if metal filter paper containers are used because of the loss of surface adsorbed moisture when heated. Air currents from rising of air heated by the hot metal tare may also contribute to a smaller hot tare mass. The average difference between hot and cold tare mass for 69 measurements is 4.6 ± 0.9 % of the filter paper mass and must be considered if measurements of the filter paper mass are to have an error less than 5 %. No test results are available for plastic bags.

9.2 The water content of the filter paper, w_f, by mass is as follows:

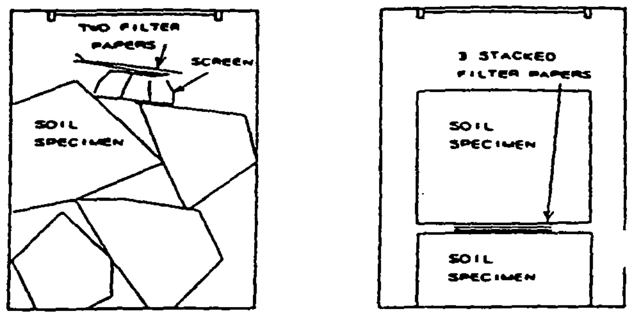
$$w_f = \frac{M_w}{M_f} \cdot 100 \tag{6}$$

where:
w_f = filter paper water content, percent.

9.3 Convert the filter paper water content, w_f, to a suction value by reference to a calibration curve or calculate the suction from the following:

$$h = m \cdot w_f + b \tag{7}$$

where:



A. Total Suction B. Matrix Suction

FIG. 2 Setup for Equilibrating Suction in Large Container

m = slope of filter paper calibration curve, $\log_{10}$ kPa/% water content, and

b = intercept of the filter paper calibration, $\log_{10}$ kPa.

9.4 A calibration curve defined by Eq 7 is unique for each type of filter paper and consists of a line with a relatively steep slope and a relatively flat slope, see Fig. 2. Take the suction determined from the calibration curve as the average of the suctions evaluated from the water contents if two filter papers were used to determine the soil suction. Discard the test results if the difference in suction between the two filter papers exceeds 0.5 log kPa.

10. Report

10.1 Figure 3 is an example data sheet for evaluating soil suction using filter paper.

10.2 Report the soil water content corresponding to the total soil suction, temperature of measurement and equilibration time, method of calibrating filter paper, and bulk density of soil.

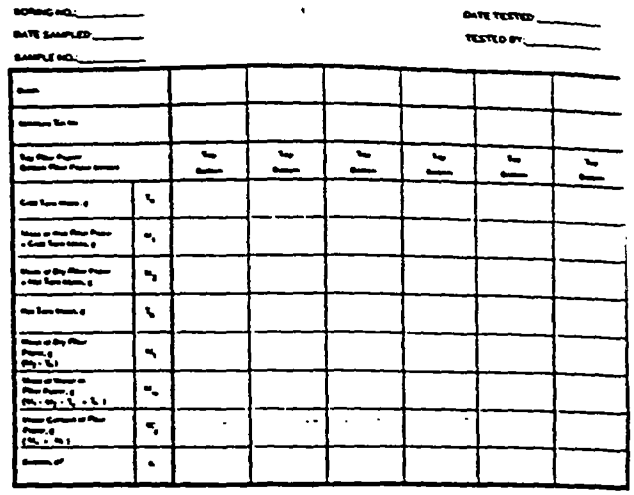


FIG. 3 Evaluation of Soil Suction Using Filter Paper

10.3 Report the salinity of the pore water if determined to permit evaluation of osmotic suction and calculation of matric suction $hm = h - hs$.

11. Precision and Bias

11.1 *Precision*—Data are being evaluated to determine the precision of this test method. In addition, Subcommittee D18.04 is seeking pertinent data from users of this test method.

11.2 *Bias*—There is no accepted reference value for this test method, therefore, bias cannot be determined.

12. Keywords

12.1 filter paper; soil relative humidity; soil suction

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(4) Orig. Footnote 12 in text.

(5) Fawcett, R. G., and Collis-George, N., "A Filter Paper Method for Determining the Moisture Characteristic of Soil," *Australian Journal of Experimental Agriculture and Animal Husbandry*, Vol 7, 1967, pp. 162–167.

(6) Manheim, F. T., "A Hydraulic Squeezer for Obtaining Interstitial Water from Consolidated and Unconsolidated Sediment," *Geological Survey Research 1966*, Chapter C, U.S. Geological Survey Professional Paper 550-C, pp. C256–C261.

(7) United States Salinity Laboratory Staff, *Diagnosis and Improvement of Saline and Alkali Soils*, U.S.D.A. Agricultural Handbook No. 60, Richards, L. A., editor, U.S. Department of Agriculture, Washington, D.C., 1954, p. 15.

RL 005240

RL 005241

Standard Test Method for Particle-Size Analysis of Soils[1]

This standard is issued under the fixed designation D 422; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

ε1 NOTE—Section 19 was added editorially in September 1990.

1. Scope

1.1 This test method covers the quantitative determination of the distribution of particle sizes in soils. The distribution of particle sizes larger than 75 μm (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75 μm is determined by a sedimentation process, using a hydrometer to secure the necessary data (Notes 1 and 2).

NOTE 1—Separation may be made on the No. 4 (4.75-mm), No. 40 (425-μm), or No. 200 (75-μm) sieve instead of the No. 10. For whatever sieve used, the size shall be indicated in the report.

NOTE 2—Two types of dispersion devices are provided: (*1*) a high-speed mechanical stirrer, and (*2*) air dispersion. Extensive investigations indicate that air-dispersion devices produce a more positive dispersion of plastic soils below the 20-μm size and appreciably less degradation on all sizes when used with sandy soils. Because of the definite advantages favoring air dispersion, its use is recommended. The results from the two types of devices differ in magnitude, depending upon soil type, leading to marked differences in particle size distribution, especially for sizes finer than 20 μm.

2. Referenced Documents

2.1 *ASTM Standards:*

D 421 Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants[2]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[3]

E 100 Specification for ASTM Hydrometers[4]

3. Apparatus

3.1 *Balances*—A balance sensitive to 0.01 g for weighing the material passing a No. 10 (2.00-mm) sieve, and a balance sensitive to 0.1 % of the mass of the sample to be weighed for weighing the material retained on a No. 10 sieve.

3.2 *Stirring Apparatus*—Either apparatus A or B may be used.

3.2.1 Apparatus A shall consist of a mechanically operated stirring device in which a suitably mounted electric motor turns a vertical shaft at a speed of not less than 10 000 rpm without load. The shaft shall be equipped with a replaceable stirring paddle made of metal, plastic, or hard rubber, as shown in Fig. 1. The shaft shall be of such length that the stirring paddle will operate not less than ¾ in. (19.0 mm) nor more than 1½ in. (38.1 mm) above the bottom of the dispersion cup. A special dispersion cup conforming to either of the designs shown in Fig. 2 shall be provided to hold the sample while it is being dispersed.

3.2.2 Apparatus B shall consist of an air-jet dispersion cup[5] (Note 3) conforming to the general details shown in Fig. 3 (Notes 4 and 5).

NOTE 3—The amount of air required by an air-jet dispersion cup is of the order of 2 ft^3/min; some small air compressors are not capable of supplying sufficient air to operate a cup.

NOTE 4—Another air-type dispersion device, known as a dispersion tube, developed by Chu and Davidson at Iowa State College, has been shown to give results equivalent to those secured by the air-jet dispersion cups. When it is used, soaking of the sample can be done in the sedimentation cylinder, thus eliminating the need for transferring the slurry. When the air-dispersion tube is used, it shall be so indicated in the report.

NOTE 5—Water may condense in air lines when not in use. This water must be removed, either by using a water trap on the air line, or by blowing the water out of the line before using any of the air for dispersion purposes.

3.3 *Hydrometer*—An ASTM hydrometer, graduated to read in either specific gravity of the suspension or grams per litre of suspension, and conforming to the requirements for hydrometers 151H or 152H in Specifications E 100. Dimensions of both hydrometers are the same, the scale being the only item of difference.

3.4 *Sedimentation Cylinder*—A glass cylinder essentially 18 in. (457 mm) in height and 2½ in. (63.5 mm) in diameter, and marked for a volume of 1000 mL. The inside diameter shall be such that the 1000-mL mark is 36 ± 2 cm from the bottom on the inside.

3.5 *Thermometer*—A thermometer accurate to 1°F (0.5°C).

3.6 *Sieves*—A series of sieves, of square-mesh woven-wire cloth, conforming to the requirements of Specification E 11. A full set of sieves includes the following (Note 6):

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.03 on Texture, Plasticity, and Density Characteristics of Soils.

Current edition approved Nov. 21, 1963. Originally published 1935. Replaces D 422 – 62.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

[3] *Annual Book of ASTM Standards*, Vol 14.02.

[4] *Annual Book of ASTM Standards*, Vol 14.03.

[5] Detailed working drawings for this cup are available at a nominal cost from the American Society for Testing and Materials, 1916 Race St., Philadelphia, PA 19103. Order Adjunct No. 12-404220-00.

HYDROMETER AND SIEVE ANALYSIS OF PORTION PASSING THE NO. 10 (2.00-mm) SIEVE

7. Determination of Composite Correction for Hydrometer Reading

7.1 Equations for percentages of soil remaining in suspension, as given in 14.3, are based on the use of distilled or demineralized water. A dispersing agent is used in the water, however, and the specific gravity of the resulting liquid is appreciably greater than that of distilled or demineralized water.

7.1.1 Both soil hydrometers are calibrated at 68°F (20°C), and variations in temperature from this standard temperature produce inaccuracies in the actual hydrometer readings. The amount of the inaccuracy increases as the variation from the standard temperature increases.

7.1.2 Hydrometers are graduated by the manufacturer to be read at the bottom of the meniscus formed by the liquid on the stem. Since it is not possible to secure readings of soil suspensions at the bottom of the meniscus, readings must be taken at the top and a correction applied.

7.1.3 The net amount of the corrections for the three items enumerated is designated as the composite correction, and may be determined experimentally.

7.2 For convenience, a graph or table of composite corrections for a series of 1° temperature differences for the range of expected test temperatures may be prepared and used as needed. Measurement of the composite corrections may be made at two temperatures spanning the range of expected test temperatures, and corrections for the intermediate temperatures calculated assuming a straight-line relationship between the two observed values.

7.3 Prepare 1000 mL of liquid composed of distilled or demineralized water and dispersing agent in the same proportion as will prevail in the sedimentation (hydrometer) test. Place the liquid in a sedimentation cyclinder and the cylinder in the constant-temperature water bath, set for one of the two temperatures to be used. When the temperature of the liquid becomes constant, insert the hydrometer, and, after a short interval to permit the hydrometer to come to the temperature of the liquid, read the hydrometer at the top of the meniscus formed on the stem. For hydrometer 151H the composite correction is the difference between this reading and one; for hydrometer 152H it is the difference between the reading and zero. Bring the liquid and the hydrometer to the other temperature to be used, and secure the composite correction as before.

8. Hygroscopic Moisture

8.1 When the sample is weighed for the hydrometer test, weigh out an auxiliary portion of from 10 to 15 g in a small metal or glass container, dry the sample to a constant mass in an oven at 230 ± 9°F (110 ± 5°C), and weigh again. Record the masses.

9. Dispersion of Soil Sample

9.1 When the soil is mostly of the clay and silt sizes, weigh out a sample of air-dry soil of approximately 50 g. When the soil is mostly sand the sample should be approximately 100 g.

9.2 Place the sample in the 250-mL beaker and cover with 125 mL of sodium hexametaphosphate solution (40 g/L). Stir until the soil is thoroughly wetted. Allow to soak for at least 16 h.

9.3 At the end of the soaking period, disperse the sample further, using either stirring apparatus A or B. If stirring apparatus A is used, transfer the soil - water slurry from the beaker into the special dispersion cup shown in Fig. 2, washing any residue from the beaker into the cup with distilled or demineralized water (Note 9). Add distilled or demineralized water, if necessary, so that the cup is more than half full. Stir for a period of 1 min.

NOTE 9—A large size syringe is a convenient device for handling the water in the washing operation. Other devices include the wash-water bottle and a hose with nozzle connected to a pressurized distilled water tank.

9.4 If stirring apparatus B (Fig. 3) is used, remove the cover cap and connect the cup to a compressed air supply by means of a rubber hose. A air gage must be on the line between the cup and the control valve. Open the control valve so that the gage indicates 1 psi (7 kPa) pressure (Note 10). Transfer the soil - water slurry from the beaker to the air-jet dispersion cup by washing with distilled or demineralized water. Add distilled or demineralized water, if necessary, so that the total volume in the cup is 250 mL, but no more.

NOTE 10—The initial air pressure of 1 psi is required to prevent the soil - water mixture from entering the air-jet chamber when the mixture is transferred to the dispersion cup.

9.5 Place the cover cap on the cup and open the air control valve until the gage pressure is 20 psi (140 kPa). Disperse the soil according to the following schedule:

Plasticity Index	Dispersion Period, min
Under 5	5
6 to 20	10
Over 20	15

Soils containing large percentages of mica need be dispersed for only 1 min. After the dispersion period, reduce the gage pressure to 1 psi preparatory to transfer of soil - water slurry to the sedimentation cylinder.

10. Hydrometer Test

10.1 Immediately after dispersion, transfer the soil - water slurry to the glass sedimentation cylinder, and add distilled or demineralized water until the total volume is 1000 mL.

10.2 Using the palm of the hand over the open end of the cylinder (or a rubber stopper in the open end), turn the cylinder upside down and back for a period of 1 min to complete the agitation of the slurry (Note 11). At the end of 1 min set the cylinder in a convenient location and take hydrometer readings at the following intervals of time (measured from the beginning of sedimentation), or as many as may be needed, depending on the sample or the specification for the material under test: 2, 5, 15, 30, 60, 250, and 1440 min. If the controlled water bath is used, the sedimentation cylinder should be placed in the bath between the 2- and 5-min readings.

NOTE 11—The number of turns during this minute should be approximately 60, counting the turn upside down and back as two turns.

Any soil remaining in the bottom of the cylinder during the first few turns should be loosened by vigorous shaking of the cylinder while it is in the inverted position.

10.3 When it is desired to take a hydrometer reading, carefully insert the hydrometer about 20 to 25 s before the reading is due to approximately the depth it will have when the reading is taken. As soon as the reading is taken, carefully remove the hydrometer and place it with a spinning motion in a graduate of clean distilled or demineralized water.

NOTE 12—It is important to remove the hydrometer immediately after each reading. Readings shall be taken at the top of the meniscus formed by the suspension around the stem, since it is not possible to secure readings at the bottom of the meniscus.

10.4 After each reading, take the temperature of the suspension by inserting the thermometer into the suspension.

11. Sieve Analysis

11.1 After taking the final hydrometer reading, transfer the suspension to a No. 200 (75-µm) sieve and wash with tap water until the wash water is clear. Transfer the material on the No. 200 sieve to a suitable container, dry in an oven at 230 ± 9°F (110 ± 5°C) and make a sieve analysis of the portion retained, using as many sieves as desired, or required for the material, or upon the specification of the material under test.

CALCULATIONS AND REPORT

12. Sieve Analysis Values for the Portion Coarser than the No. 10 (2.00-mm) Sieve

12.1 Calculate the percentage passing the No. 10 sieve by dividing the mass passing the No. 10 sieve by the mass of soil originally split on the No. 10 sieve, and multiplying the result by 100. To obtain the mass passing the No. 10 sieve, subtract the mass retained on the No. 10 sieve from the original mass.

12.2 To secure the total mass of soil passing the No. 4 (4.75-mm) sieve, add to the mass of the material passing the No. 10 sieve the mass of the fraction passing the No. 4 sieve and retained on the No. 10 sieve. To secure the total mass of soil passing the ⅜-in. (9.5-mm) sieve, add to the total mass of soil passing the No. 4 sieve, the mass of the fraction passing the ⅜-in. sieve and retained on the No. 4 sieve. For the remaining sieves, continue the calculations in the same manner.

12.3 To determine the total percentage passing for each sieve, divide the total mass passing (see 12.2) by the total mass of sample and multiply the result by 100.

13. Hygroscopic Moisture Correction Factor

13.1 The hydroscopic moisture correction factor is the ratio between the mass of the oven-dried sample and the air-dry mass before drying. It is a number less than one, except when there is no hygroscopic moisture.

14. Percentages of Soil in Suspension

14.1 Calculate the oven-dry mass of soil used in the hydrometer analysis by multiplying the air-dry mass by the hygroscopic moisture correction factor.

14.2 Calculate the mass of a total sample represented by the mass of soil used in the hydrometer test, by dividing the oven-dry mass used by the percentage passing the No. 10 (2.00-mm) sieve, and multiplying the result by 100. This value is the weight W in the equation for percentage remaining in suspension.

TABLE 1 Values of Correction Factor, α, for Different Specific Gravities of Soil Particles[A]

Specific Gravity	Correction Factor[A]
2.95	0.94
2.90	0.95
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.03
2.45	1.05

[A] For use in equation for percentage of soil remaining in suspension when using Hydrometer 152H.

14.3 The percentage of soil remaining in suspension at the level at which the hydrometer is measuring the density of the suspension may be calculated as follows (Note 13): For hydrometer 151H:

$$P = [(100\,000/W) \times G/(G - G_1)](R - G_1)$$

NOTE 13—The bracketed portion of the equation for hydrometer 151H is constant for a series of readings and may be calculated first and then multiplied by the portion in the parentheses.

For hydrometer 152H:

$$P = (Ra/W) \times 100$$

where:

a = correction faction to be applied to the reading of hydrometer 152H. (Values shown on the scale are computed using a specific gravity of 2.65. Correction factors are given in Table 1),
P = percentage of soil remaining in suspension at the level at which the hydrometer measures the density of the suspension,
R = hydrometer reading with composite correction applied (Section 7),
W = oven-dry mass of soil in a total test sample represented by mass of soil dispersed (see 14.2), g,
G = specific gravity of the soil particles, and
G_1 = specific gravity of the liquid in which soil particles are suspended. Use numerical value of one in both instances in the equation. In the first instance any possible variation produces no significant effect, and in the second instance, the composite correction for R is based on a value of one for G_1.

15. Diameter of Soil Particles

15.1 The diameter of a particle corresponding to the percentage indicated by a given hydrometer reading shall be calculated according to Stokes' law (Note 14), on the basis that a particle of this diameter was at the surface of the suspension at the beginning of sedimentation and had settled to the level at which the hydrometer is measuring the density of the suspension. According to Stokes' law:

$$D = \sqrt{[30n/980(G - G_1)] \times L/T}$$

where:

D = diameter of particle, mm,

n = coefficient of viscosity of the suspending medium (in this case water) in poises (varies with changes in temperature of the suspending medium),

L = distance from the surface of the suspension to the level at which the density of the suspension is being measured, cm. (For a given hydrometer and sedimentation cylinder, values vary according to the hydrometer readings. This distance is known as effective depth (Table 2)),

T = interval of time from beginning of sedimentation to the taking of the reading, min,

G = specific gravity of soil particles, and

G_1 = specific gravity (relative density) of suspending medium (value may be used as 1.000 for all practical purposes).

NOTE 14—Since Stokes' law considers the terminal velocity of a single sphere falling in an infinity of liquid, the sizes calculated represent the diameter of spheres that would fall at the same rate as the soil particles.

15.2 For convenience in calculations the above equation may be written as follows:

$$D = K\sqrt{L/T}$$

where:

K = constant depending on the temperature of the suspension and the specific gravity of the soil particles. Values of K for a range of temperatures and specific gravities are given in Table 3. The value of K does not change for a series of readings constituting a test, while values of L and T do vary.

15.3 Values of D may be computed with sufficient accuracy, using an ordinary 10-in. slide rule.

NOTE 15—The value of L is divided by T using the A- and B-scales, the square root being indicated on the D-scale. Without ascertaining the value of the square root it may be multiplied by K, using either the C- or CI-scale.

16. Sieve Analysis Values for Portion Finer than No. 10 (2.00-mm) Sieve

16.1 Calculation of percentages passing the various sieves used in sieving the portion of the sample from the hydrometer test involves several steps. The first step is to calculate the mass of the fraction that would have been retained on the No. 10 sieve had it not been removed. This mass is equal to the total percentage retained on the No. 10 sieve (100 minus total percentage passing) times the mass of the total sample represented by the mass of soil used (as calculated in 14.2), and the result divided by 100.

16.2 Calculate next the total mass passing the No. 200 sieve. Add together the fractional masses retained on all the sieves, including the No. 10 sieve, and subtract this sum from the mass of the total sample (as calculated in 14.2).

16.3 Calculate next the total masses passing each of the other sieves, in a manner similar to that given in 12.2.

16.4 Calculate last the total percentages passing by dividing the total mass passing (as calculated in 16.3) by the total mass of sample (as calculated in 14.2), and multiply the result by 100.

17. Graph

17.1 When the hydrometer analysis is performed, a graph of the test results shall be made, plotting the diameters of the particles on a logarithmic scale as the abscissa and the percentages smaller than the corresponding diameters to an

TABLE 2 Values of Effective Depth Based on Hydrometer and Sedimentation Cylinder of Specified Sizes[A]

Hydrometer 151H		Hydrometer 152H			
Actual Hydrometer Reading	Effective Depth, L, cm	Actual Hydrometer Reading	Effective Depth, L, cm	Actual Hydrometer Reading	Effective Depth, L, cm
1.000	16.3	0	16.3	31	11.2
1.001	16.0	1	16.1	32	11.1
1.002	15.8	2	16.0	33	10.9
1.003	15.5	3	15.8	34	10.7
1.004	15.2	4	15.6	35	10.6
1.005	15.0	5	15.5		
1.006	14.7	6	15.3	36	10.4
1.007	14.4	7	15.2	37	10.2
1.008	14.2	8	15.0	38	10.1
1.009	13.9	9	14.8	39	9.9
1.010	13.7	10	14.7	40	9.7
1.011	13.4	11	14.5	41	9.6
1.012	13.1	12	14.3	42	9.4
1.013	12.9	13	14.2	43	9.2
1.014	12.6	14	14.0	44	9.1
1.015	12.3	15	13.8	45	8.9
1.016	12.1	16	13.7	46	8.8
1.017	11.8	17	13.5	47	8.6
1.018	11.5	18	13.3	48	8.4
1.019	11.3	19	13.2	49	8.3
1.020	11.0	20	13.0	50	8.1
1.021	10.7	21	12.9	51	7.9
1.022	10.5	22	12.7	52	7.8
1.023	10.2	23	12.5	53	7.6
1.024	10.0	24	12.4	54	7.4
1.025	9.7	25	12.2	55	7.3
1.026	9.4	26	12.0	56	7.1
1.027	9.2	27	11.9	57	7.0
1.028	8.9	28	11.7	58	6.8
1.029	8.6	29	11.5	59	6.6
1.030	8.4	30	11.4	60	6.5
1.031	8.1				
1.032	7.8				
1.033	7.6				
1.034	7.3				
1.035	7.0				
1.036	6.8				
1.037	6.5				
1.038	6.2				

[A] Values of effective depth are calculated from the equation:

$$L = L_1 + \tfrac{1}{2}\,[L_2 - (V_B/A)]$$

where:

L = effective depth, cm,

L_1 = distance along the stem of the hydrometer from the top of the bulb to the mark for a hydrometer reading, cm,

L_2 = overall length of the hydrometer bulb, cm,

V_B = volume of hydrometer bulb, cm^3, and

A = cross-sectional area of sedimentation cylinder, cm^2

Values used in calculating the values in Table 2 are as follows:

For both hydrometers, 151H and 152H:

L_2 = 14.0 cm

V_B = 67.0 cm^3

A = 27.8 cm^2

For hydrometer 151H:

L_1 = 10.5 cm for a reading of 1.000

= 2.3 cm for a reading of 1.031

For hydrometer 152H:

L_1 = 10.5 cm for a reading of 0 g/litre

= 2.3 cm for a reading of 50 g/litre

TABLE 3 Values of K for Use in Equation for Computing Diameter of Particle in Hydrometer Analysis

Temperature, °C	Specific Gravity of Soil Particles								
	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.01510	0.01505	0.01481	0.01457	0.01435	0.01414	0.01394	0.01374	0.01356
17	0.01511	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.01321
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.1323	0.01305
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.01289
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.01258
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.01243
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.01229
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.01201
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01255	0.01208	0.01191	0.01175
29	0.01312	0.01290	0.01269	0.01249	0.01230	0.01212	0.01195	0.01178	0.01162
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.01149

arithmetic scale as the ordinate. When the hydrometer analysis is not made on a portion of the soil, the preparation of the graph is optional, since values may be secured directly from tabulated data.

18. Report

18.1 The report shall include the following:

18.1.1 Maximum size of particles,

18.1.2 Percentage passing (or retained on) each sieve, which may be tabulated or presented by plotting on a graph (Note 16),

18.1.3 Description of sand and gravel particles:

18.1.3.1 Shape—rounded or angular,

18.1.3.2 Hardness—hard and durable, soft, or weathered and friable,

18.1.4 Specific gravity, if unusually high or low,

18.1.5 Any difficulty in dispersing the fraction passing the No. 10 (2.00-mm) sieve, indicating any change in type and amount of dispersing agent, and

18.1.6 The dispersion device used and the length of the dispersion period.

NOTE 16—This tabulation of graph represents the gradation of the sample tested. If particles larger than those contained in the sample were removed before testing, the report shall so state giving the amount and maximum size.

18.2 For materials tested for compliance with definite specifications, the fractions called for in such specifications shall be reported. The fractions smaller than the No. 10 sieve shall be read from the graph.

18.3 For materials for which compliance with definite specifications is not indicated and when the soil is composed almost entirely of particles passing the No. 4 (4.75-mm) sieve, the results read from the graph may be reported as follows:

(1) Gravel, passing 3-in. and retained on No. 4 sieve .... %
(2) Sand, passing No. 4 sieve and retained on No. 200 sieve .... %
  (a) Coarse sand, passing No. 4 sieve and retained on No. 10 sieve .... %
  (b) Medium sand, passing No. 10 sieve and retained on No. 40 sieve .... %
  (c) Fine sand, passing No. 40 sieve and retained on No. 200 sieve .... %
(3) Silt size, 0.074 to 0.005 mm .... %
(4) Clay size, smaller than 0.005 mm .... %
  Colloids, smaller than 0.001 mm .... %

18.4 For materials for which compliance with definite specifications is not indicated and when the soil contains material retained on the No. 4 sieve sufficient to require a sieve analysis on that portion, the results may be reported as follows (Note 17):

SIEVE ANALYSIS

Sieve Size	Percentage Passing
3-in.	....
2-in.	....
1½-in.	....
1-in.	....
¾-in.	....
⅜-in.	....
No. 4 (4.75-mm)	....
No. 10 (2.00-mm)	....
No. 40 (425-μm)	....
No. 200 (75-μm)	....

HYDROMETER ANALYSIS

0.074 mm	....
0.005 mm	....
0.001 mm	....

NOTE 17—No. 8 (2.36-mm) and No. 50 (300-μm) sieves may be substituted for No. 10 and No. 40 sieves.

19. Keywords

19.1 grain-size; hydrometer analysis; hygroscopic moisture; particle-size; sieve analysis

Standard Test Method for Specific Gravity of Soils[1]

This standard is issued under the fixed designation D 854; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the determination of the specific gravity of soils that pass the 4.75-mm (No. 4) sieve, by means of a pycnometer. When the soil contains particles larger than the 4.75-mm sieve, Test Method C 127 shall be used for the material retained on the 4.75-mm sieve and this test method shall be used for the material passing the 4.75-mm sieve.

1.1.1 Two procedures for performing the specific gravity are provided as follows:

1.1.1.1 *Method A*—Procedure for Oven-Dry Specimens, described in 9.1.

1.1.1.2 *Method B*—Procedure for Moist Specimens, described in 9.2. The procedure to be used shall be specified by the requesting authority. For specimens of organic soils and highly plastic, fine-grained soils, Procedure B shall be the preferred method.

1.2 When the specific gravity value is to be used in calculations in connection with the hydrometer portion of Test Method D 422, it is intended that the specific gravity test be made on that portion of the sample which passes the 2.00-mm (No. 10) sieve.

1.3 The values stated in acceptable metric units are to be regarded as standard.

1.4 *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 127 Test Method for Specific Gravity and Absorption of Coarse Aggregate[2]

C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials[2]

D 422 Test Method for Particle-Size Analysis of Soils[3]

D 653 Terminology Relating to Soil, Rock, and Contained Fluids[3]

D 2487 Test Method for Classification of Soils for Engineering Purposes[3]

D 4753 Specification for Evaluating, Selecting, and Specifying Balances and Scales for Use in Soil and Rock Testing[3]

E 1 Specification for ASTM Thermometers[4]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[5]

E 12 Terminology Relating to Density and Specific Gravity of Solids, Liquids, and Gases[6]

2.2 *AASHTO Standards:*[7]

AASHTO Test Method T100

3. Terminology

3.1 All definitions are in accordance with Terminology D 653 and E 12.

3.2 *Description of Term Specific to This Standard:*

3.2.1 *specific gravity*—the ratio of the mass of a unit volume of a material at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

4. Significance and Use

4.1 The specific gravity of a soil is used in calculating the phase relationships of soils (that is, the relative volumes of solids to water and air in a given volume of soil).

4.2 The term solid particles is typically assumed to mean naturally occurring mineral particles that are not readily soluble in water. Therefore, the specific gravity of materials containing extraneous matter (such as cement, lime, and the like), water-soluble matter (such as sodium chloride), and soils containing matter with a specific gravity less than one, typically require special treatment or a qualified definition of their specific gravity.

5. Apparatus

5.1 *Pycnometer*—The pycnometer shall be one of the following:

5.1.1 *Volumetric Flask*, having a capacity of at least 100 mL.

5.1.2 *Stoppered Bottle*, having a capacity of at least 50 mL. The stopper shall be of the same material, and shall permit the emission of air and surplus water when it is put in place.

NOTE 1—Flask sizes of greater than the specified minimum capacity are recommended. Larger flasks are capable of holding larger specimens and tend to produce better statistical results.

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.03 on Texture, Plasticity and Density Characteristics of Soils.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.08.

[4] *Annual Book of ASTM Standards*, Vol 14.03.

[5] *Annual Book of ASTM Standards*, Vol 14.02.

[6] *Annual Book of ASTM Standards*, Vol 15.05.

[7] Available from American Association of State Highway and Transportation Officials, 444 N Capital St., NW, Washington, DC 20001.

5.2 *Balance*—Meeting the requirements of Specification D 4753 and readable, without estimation, to at least 0.1 % of the specimen mass.

5.3 *Drying Oven*—Thermostatically-controlled oven, capable of maintaining a uniform temperature of 110 ± 5°C (230 ± 9°F) throughout the drying chamber.

5.4 *Thermometer*, capable of measuring the temperature range within which the test is being performed, graduated in a 0.5°C (1.0°F) division scale and meeting the requirements of Specification E 1.

5.5 *Desiccator*—A desiccator cabinet or large desiccator jar of suitable size containing silica gel or anhydrous calcium sulfate.[8]

NOTE 2—It is preferable to use a desiccant that changes color to indicate when it needs reconstitution.

5.6 *Entrapped Air Removal Apparatus*—To remove entrapped air, use one of the following:

5.6.1 *Hot Plate or Bunsen Burner*, capable of maintaining a temperature adequate to boil water.

5.6.2 *Vacuum System*, a vacuum pump or water aspirator, capable of producing a partial vacuum of 100 min or less absolute pressure.

NOTE 3—A partial vacuum of 100 mm Hg absolute pressure is approximately equivalent to a 660 mm (26 in.) Hg reading on vacuum gauge at sea level.

5.7 *Miscellaneous Equipment*, specimen dishes and insulated gloves.

6. Reagents and Materials

6.1 *Purity of Water*—Where distilled water is referred to in this test method, either distilled or demineralized water may be used.

7. Test Specimen

7.1 The test specimen may be oven-dried or moistened soil and shall be representative of the total sample. In either case the specimen shall be large enough that its minimum mass in the oven-dried state is in accordance with the following:

Maximum Particle Size (100 % passing)	Standard Sieve Size	Minimum Mass of Test Specimen, g
2 mm	No. 10	20
4.75 mm	No. 4	100

8. Calibration of Pycnometer

8.1 Determine and record the mass of a clean, dry pycnometer, M_f.

8.2 Fill the pycnometer with distilled water to the calibration mark. Visually inspect the pycnometer and its contents to ensure that there are no air bubbles in the distilled water. Determine and record the mass of the pycnometer and water, M_a.

8.3 Insert a thermometer in the water, and determine and record its temperature, T_a, to the nearest 0.5°C (1.0°F).

8.4 From the mass, M_a, determined at the observed temperature, T_a, prepare a table of values of mass, M_a, for a series of temperatures that are likely to prevail when the mass of the pycnometer, soil, and water, M_b, is determined later. These values of M_a can be determined experimentally or may be calculated as follows:

$$M_a \text{ (at } T_x) = [(\text{density of water at } T_x/\text{density of water at } T_a) \times (M_a \text{ (at } T_a) - M_f)] + M_f$$

where:
M_a = mass of pycnometer and water, g,
M_f = mass of pycnometer, g,
T_a = observed temperature of water, °C, and
T_x = any other desired temperature, °C.

NOTE 4—This test method provides a procedure that is more convenient for laboratories making many determinations with the same pycnometer. It is equally applicable to a single determination. Bringing the pycnometer and contents to some designated temperature when masses M_a and M_b are taken, requires considerable time. It is important that masses M_a and M_b be based on water at the same temperature. Values for the density of water at temperatures from 16.0 to 30.0°C are given in Table 1.

9. Procedure

9.1 *Test Method A—Procedure For Oven-Dried Specimens:*

9.1.1 Dry the specimen to a constant mass in an oven maintained at 110 ± 5°C (230 ± 9°F) (See Note 5) and cool it in a desiccator.

NOTE 5—Drying of certain soils at 110°C (230°F) may bring about loss of water of composition or hydration, and in such cases drying may be done in reduced air pressure or at a lower temperature.

9.1.2 Determine and record the mass of a clean, dry, calibrated pycnometer, M_f. Select a pycnometer of sufficient capacity that the volume filled to the mark will be at least 50 percent greater than the space required to accommodate the test specimen. Place the specimen in the pycnometer.

TABLE 1 Density of Water and Correction Factor *K* for Various Temperatures

Temperature, °C	Density of Water (g/mL)	Correction Factor *K*
16.0	0.99897	1.0007
16.5	0.99889	1.0007
17.0	0.99880	1.0006
17.5	0.99871	1.0005
18.0	0.99862	1.0004
18.5	0.99853	1.0003
19.0	0.99843	1.0002
19.5	0.99833	1.0001
20.0	0.99823	1.0000
20.5	0.99812	0.9999
21.0	0.99802	0.9998
21.5	0.99791	0.9997
22.0	0.99780	0.9996
22.5	0.99768	0.9995
23.0	0.99757	0.9993
23.5	0.99745	0.9992
24.0	0.99732	0.9991
24.5	0.99720	0.9990
25.0	0.99707	0.9988
25.5	0.99694	0.9987
26.0	0.99681	0.9986
26.5	0.99668	0.9984
27.0	0.99654	0.9983
27.5	0.99640	0.9982
28.0	0.99626	0.9980
28.5	0.99612	0,9979
29.0	0.99597	0.9977
29.5	0.99582	0.9976
30.0	0.99567	0.9974

[8] Anhydrous calcium sulfate is sold under the trade name Drierite.

TABLE 2 Table of Precision Estimates[A]

Material and Type Index	Standard Deviation[B] Passing No. 4 (4.75 mm)	Standard Deviation[B] Passing No. 10 (2.00 mm)	Acceptable Range of Two Results[C] Passing No. 4 (4.75 mm)	Acceptable Range of Two Results[C] Passing No. 10 (2.00 mm)
Single-operator precision:				
Cohesive soils	0.021	0.019	0.06	0.06
Noncohesive soils	B	B	B	B
Multilaboratory precision:				
Cohesive soils	0.056	0.041	0.16	0.12
Noncohesive soils	B	B	B	B

[A] The figures given in Columns 2 and 3 are the standard deviations that have been found to be appropriate for the materials described in Column 1. The figures given in Columns 4 and 5 are the limits that should not be exceeded by the difference between the two properly conducted tests.

[B] These numbers represent, respectively, the (1S) and (D2S) limits as described in Practice C 670.

[C] Criteria for assigning standard deviation values for noncohesive soils are not available at the present time.

Determine the mass of the specimen and pycnometer, and subtract the mass of the pycnometer, M_f, from this value to determine the mass of the oven-dry specimen, M_o.

9.1.3 Fill the pycnometer with distilled water to a level slightly above that required to cover the soil and soak the specimen for at least 12 h.

NOTE 6—For some soils containing a significant fraction of organic matter, kerosine is a better wetting agent than water and may be used in place of distilled water for oven-dried specimens. If kerosine is used, the entrapped air should only be removed by use of an aspirator. Kerosine is a flammable liquid that must be used with extreme caution.

NOTE 7—Adding distilled water to just cover the soil makes it easier to control boil-over during removal of entrapped air.

9.1.4 Remove the entrapped air by one of the following methods:

9.1.4.1 Boil the specimen gently for at least 10 min while agitating the pycnometer occasionally to assist in the removal of air. Then cool the heated specimen to room temperature.

9.1.4.2 Subject the contents to a vacuum (air pressure not exceeding 100 mm Hg) for at least 30 min (Note 8) either by connecting the pycnometer directly to an aspirator or vacuum pump or by use of a bell jar. While the vacuum is being applied, gently agitate the pycnometer periodically to assist in the removal of air. Some soils boil violently when subjected to reduced air pressure. It will be necessary in those cases to reduce the air pressure at a slower rate or to use a larger flask.

NOTE 8—Specimens with a high plasticity at the natural water content may require 6 to 8 h to remove entrapped air. Specimens with a low plasticity at the natural water content may require 4 to 6 h to remove entrapped air. Oven-dried specimens may require 2 to 4 h to remove entrapped air.

9.1.5 Fill the pycnometer to just below the calibration mark with distilled water at room temperature. Add the distilled water slowly and carefully to avoid the entrapment of air bubbles in the specimen (Note 9). Allow the pycnometer to obtain a uniform water temperature (Note 10).

NOTE 9—To avoid the entrapment of air bubbles, the distilled water can be introduced through a piece of small-diameter flexible tubing with its outlet end kept just below the surface of the distilled water in the pycnometer.

NOTE 10—To obtain a uniform water temperature the pycnometer may be allowed to sit overnight or be placed in a constant temperature bath.

9.1.6 Fill the pycnometer with distilled water at the same temperature to the mark, clean the outside, and dry with a clean, dry cloth. Determine and record the mass of the pycnometer filled with soil and water, M_b.

9.1.7 Insert a thermometer into the water, and determine and record its temperature, T_b, to the nearest 0.5°C (1.0°F).

9.2 *Test Method B—Procedure For Moist Specimens:*

9.2.1 Place the specimen in a calibrated pycnometer.

9.2.1.1 Disperse specimens of clay soils in distilled water before they are placed in the pycnometer, by use of the dispersing equipment specified in Test Method D 422. The minimum volume of slurry that can be prepared by this dispersing equipment is such that a 500-mL (or larger) flask is needed as a pycnometer.

9.2.2 Proceed as described in Sections 9.1.4 and 9.1.7.

9.2.3 Remove the specimen from the pycnometer, and dry it to a constant mass in an oven maintained at 110 ± 5°C (230 ± 9°F) (See Note 5). Cool the specimen in a desiccator.

9.2.4 Determine and record the mass of the oven-dried soil, M_o.

10. Calculation

10.1 Calculate the specific gravity of the soil, G, to the nearest 0.01, based on water at a temperature (T_b) as follows:

$$G \text{ at } T_b = M_o/[M_o + (M_a - M_b]$$

where:

M_o = mass of sample of oven-dry soil, g,
M_a = mass of pycnometer filled with water at temperature T_b (Note 11), g,
M_b = mass of pycnometer filled with water and soil at temperature T_x, g,
T_b = temperature of the contents of the pycnometer when mass M_b was determined, °C.

NOTE 11—This value can be obtained from the table of values of M_a, prepared in accordance with 6.2, for the temperatures prevailing when mass M_b was determined, °C.

NOTE 12—The equation shown in 9.1 is for computing the specific gravity of the soil tested in water. When kerosine is used, the Eq must be adjusted by multiplying the result by the specific gravity of kerosine at T_b and dividing it by the density of water at T_b.

10.2 Calculate the weighted average specific gravity for soils containing particles both larger and smaller than the 4.75-mm sieve using the following equation:

$$G_{avg} = \frac{1}{\dfrac{R_1}{100G_1} + \dfrac{P_1}{100G_2}}$$

where:

G_{avg} = weighted average specific gravity of soils composed of particles larger and smaller than the 4.75-mm sieve,
R_1 = percent of soil particles retained on 4.75-mm sieve,
P_1 = percent of soil particles passing the 4.75-mm sieve,
G_1 = apparent specific gravity of soil particles retained on the 4.75-mm sieve as determined by Test Method C 127, and

G_2 = specific gravity of soil particles passing the 4.75-mm sieve as determined by this test method.

10.3 Unless otherwise required, specific gravity (G) values reported shall be based on water at 20°C. Calculate the value based on water at 20°C from the value based on water at the observed temperature T_b, as follows:

$$G \text{ at } 20°C = K \times (G \text{ at } T_b)$$

where:

K = a number found by dividing the density of water at temperature T_b by the density of water at 20°C. Values for the range of temperatures are given in Table 1.

10.4 In some cases, it is desired to report the specific gravity value based on water at a different temperature. In these cases, the specific gravity value, based on any temperature T_x, may be calculated as follows:

$$G \text{ at } T_x = \frac{G \text{ at } 20°C}{K}$$

11. Report

11.1 The report (data sheet) shall include the following:

11.1.1 Identification of the sample (material) being tested, such as boring number, sample number, test number, etc.

11.1.2 Specific gravity at 20°C to the nearest 0.01. Test procedure used (A or B).

11.1.3 Maximum particle size of the test specimen.

11.1.4 Specific gravity to the nearest 0.01 at a specified temperature other than 20°C, if applicable.

11.1.5 Type of fluid used, if other than distilled water.

11.1.6 When any portion of the original sample of soil is eliminated in the preparation of the test specimen, the portion on which the test has been made shall be reported.

12. Precision and Bias

12.1 *Precision*—Criteria for judging the acceptability of specific gravity test results obtained by this test method on material passing the 4.75-mm sieve are given as follows:

12.2 *Statement of Precision*—Criteria for judging the acceptability of specific gravity test results obtained by this test method on material passing the 4.75 (No. 4) or 2.00 mm (No. 10) sieve are given in Table 2. The estimates of precision for material passing the 2.00 mm sieve are based on results from the AASHTO Materials Reference Laboratory (AMRL) Proficiency Sample Program, of testing conducted on material passing the 2.00 (No. 10) sieve by this test method and AASHTO Test Method T100.

12.3 *Bias*—There is no acceptable reference value for this test method; therefore, bias cannot be determined.

13. Keywords

13.1 soil; specific gravity

Suction Measurements – Filter Paper and Chilled Mirror Psychrometer

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Abstract

In this paper, two soil suction measurement techniques are compared: the filter paper method and a chilled mirror psychrometer technique. With the filter paper method, both total and matric suction measurements are possible; however, with the chilled mirror psychrometer technique only total suction measurements can be made. Since the chilled mirror psychrometer can only infer total suction, the comparison is based on this component of suction. Soil suction is one of the stress state variables describing the behavior of unsaturated soils. Soil suction has found its applications in all branches of unsaturated soil mechanics, from volume change predictions of expansive soils to slope stability problems. For the filter paper method a calibration curve was developed using Schleicher and Schuell No. 589-WH filter papers and NaCl salt solutions. Initially dry filter papers were equilibrated with the vapor pressures developed above the salt solutions in closed containers at 25°C temperature. A water bath was employed for this purpose to attain a constant temperature environment, in which temperature fluctuations did not exceed ±0.2°C. A total suction (or osmotic suction) versus relative humidity relationship was obtained with the chilled mirror psychrometer employing the same salt solutions used in the filter paper calibration. From the calibration data, the capabilities and limitations of the two methods were analyzed.

Introduction

In recent years, there have been tremendous improvements in theoretical approaches and experimental studies of the behavior of unsaturated soils. After soil suction was proven to be a stress state variable in unsaturated soil mechanics, its measurement for engineering applications has become very important. Soil suction is one of the most important parameters describing the moisture stress condition of unsaturated soils. For many cases, soils are unsaturated and behave quite differently from that predicted by saturated soil mechanics theory. Soil suction and positive pore water pressure are two similar important parameters in regard to describing the behavior of unsaturated and saturated soils, respectively (Houston et al. 1994). The initial and final soil suction profiles within the active zone provide valuable information for many engineering applications. These profiles can be obtained with reasonable accuracy with reliable soil suction measurement techniques.

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There are many different methods of measuring soil suction, some of which have been in practice for many years. However, each of these methods has shortcomings with regard to one or more aspects, such as the range of application, cost, reliability, and practicality. Among suction measurement methods, the filter paper and psychrometer techniques are the two most commonly used. The filter paper method and the psychrometer technique can be used confidently if the basic principles of the methods are understood and a well-maintained laboratory procedure is followed. There are several different types of psychrometers available for suction measurements and in this study the new dewpoint chilled mirror psychrometer [or its product name WP4 Dewpoint PotentiaMeter, developed by Decagon Devices, Inc. (Decagon Devices, Inc. 1999)] is described.

With the filter paper method, both total and matric suction measurements are possible; however, with the chilled mirror psychrometer only total suction measurements can be made. The working principle behind the filter paper method is simply that the filter paper will come to a moisture equilibrium with the soil either through vapor flow or liquid flow. At equilibrium, the suction value of the filter paper and the soil will be the same. If the filter paper is allowed to absorb water through vapor flow (no contact between the filter paper and soil), then only total suction is measured. However, if the filter paper is allowed to absorb water through fluid flow (contact between the filter paper and soil), then only matric suction is measured. The chilled mirror psychrometer, on the other hand, can only infer total suction by measuring the relative humidity in a small chamber in which the free energy of the relative humidity in the chamber is in thermodynamic equilibrium with the free energy of the soil water.

Soil Suction

Soil suction can simply be described as a measure of the ability of a soil to attract and hold water. In geotechnical engineering practice, soil suction is composed of two components: matric and osmotic suction. Matric suction comes from the capillarity, texture, and surface adsorptive forces of the soil. Osmotic suction arises from the dissolved salts contained in the soil water. The sum of the matric and osmotic suction is called the total suction, which is given as:

$$h_t = h_m + h_\pi \tag{1}$$

where h_t is the total suction, h_m is the matric suction, and h_π is the osmotic suction. Soil suction can also be defined in terms of the free energy state of soil water using the principles of thermodynamics. Depending on the field of application, suction has several different names such as water potential, free energy of water, chemical potential of water. If it is assumed that the water vapor pressures are in a range in which the ideal gas law is valid, then the free energy of soil water can be calculated using water vapor pressure measurements (Edlefsen and Anderson 1943). Free energy always has the same value in water and its vapor phase when they are in thermodynamic equilibrium with each other. In other words, the free energy of the water vapor above a water surface is equal to the free energy of the water itself. The thermodynamic relationship, or Kelvin's equation, describing total suction (or free energy of water) can be written as:

$$h_t = \frac{RT}{V}\ln\left(\frac{P}{P_o}\right) \tag{2}$$

where h_t is the total suction, R is the universal gas constant, T is the absolute temperature, V is the molar volume of water, P/P_o is the relative humidity, P is the partial pressure of pore water vapor, and P_o is the saturation pressure of water vapor over a flat surface of pure water at the same temperature.

Soil suction has usually been represented in *pF* units (Schofield 1935) (i.e., suction in $pF = log_{10}$ *(suction in cm of water)*). Soil suction is also currently being represented in *log kPa* unit system (Fredlund and Rahardjo 1993) (i.e., suction in *log kPa* = log_{10} *(suction in kPa)*). The relationship between these two systems of units is approximately $log\ kPa = pF - 1$.

Filter Paper Method

The filter paper method is an indirect laboratory test method for soil suction measurements. This method has several advantages over other suction measurement devices. It is an inexpensive and relatively simple laboratory test method and also the only method from which both total and matric suction can be inferred. It is also the only method that can cover the whole suction range. However, about one week is required to reach a satisfying suction equilibrium condition. In the filter paper method, the soil specimen and filter paper are brought to equilibrium either in a contact (matric suction measurement) or not in a contact (total suction measurement) manner in a constant temperature environment. After equilibrium is established between the filter paper and soil, the water content of the filter paper disc is measured. Then, by using an appropriate filter paper water content versus suction calibration curve, the corresponding suction value is estimated.

It is very important that all the items related to the filter paper testing are cleaned carefully before commencing the filter paper suction measurements. It is strongly suggested that latex gloves and tweezers are used to handle the materials in nearly all steps of the experiment. A detailed description of the experimental procedure for the filter paper calibration testing and soil suction measurements can be found in Bulut et al. (2001). The filter paper method can be a very reliable soil suction measurement technique if the basic principles of the method are understood and strictly-practiced laboratory procedures are carefully followed, such as prescribed by Lee (1991), ASTM D 5298-94 (1994), and Bulut et al. (2001).

Calibration of Filter Papers

In this study, a wetting filter paper calibration curve was constructed using NaCl salt solutions and Schleicher & Schuell No. 589-WH filter papers. It is important to note that calibration curves are unique for each type of filter paper and that only calibration curves for a particular filter paper can be used to infer soil suction from these papers. The wetting filter paper calibration curve using salt solutions is based upon the thermodynamic relationship between osmotic suction and the water activity or relative humidity resulting from a specific concentration of salt in solution. For the wetting calibration curve developed in this study, sodium chloride solutions were prepared to cover the practical range of suction interest. Distilled water was used to achieve the saturation condition for zero total suction measurements. Then, two Schleicher and Schuell No. 589-WH filter papers were simply placed in a non-contact manner above salt solutions of various molality and distilled water and sealed in an airtight container. After sealing, the whole apparatus was put in a constant temperature environment of 25°C for equilibrium. Temperature fluctuations were kept as low as possible during a three-week equilibrium period. A water bath was employed for this purpose and measured temperature fluctuations did not exceed ± 0.2°C. After the equilibration time the filter paper water content was measured as quickly and accurately as possible. Finally, the total suction is plotted versus filter paper water content to obtain the wetting calibration curve, which is given in Fig. 1. From the figure, the sensitivity of the filter paper water content and total suction relationship can be seen clearly at very low suction values.

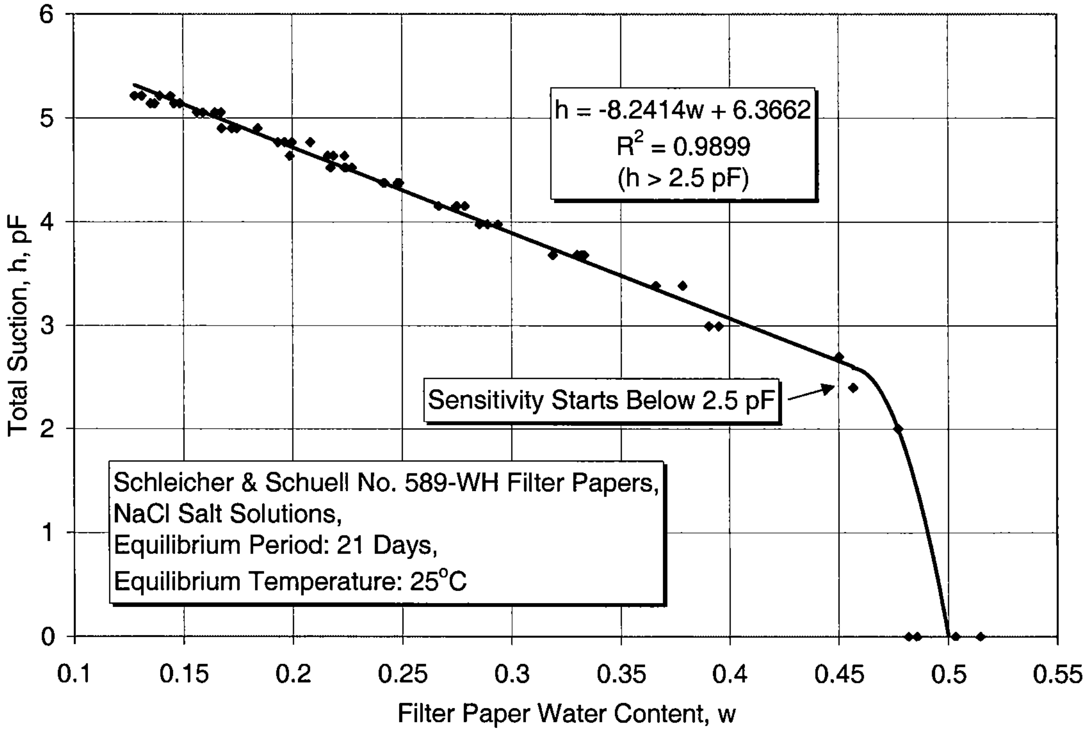

Fig. 1. Total Suction Calibration Curve.

The dramatic decrease in total suction at high water contents depends on the natural logarithmic function in Kelvin's equation (i.e., Eq. 2) and the use of the base ten logarithmic scale for total suction (Bulut et al. 2001). The sensitivity of suction with very small changes in the filter paper water content at low suction levels can also be described with the relative humidity and osmotic suction relationship of a salt solution. Table 1 gives water activity and osmotic suction relationship for sodium sulfate solutions at low suction levels. As it is seen from Table 1, within the suction sensitivity range, the change in relative humidity is only at fourth decimal points up to a suction level of about 3 pF. In other words, very small, minor changes in relative humidity result in very large changes in suction.

Chilled Mirror Psychrometer Technique

The WP4 instrument also makes use of the water activity or relative humidity measurements to infer suction. In that sense, the principles behind using the wetting filter calibration curve for total suction measurements and using the WP4 device are very similar. The chilled mirror sensors in the WP4 instrument are fundamental in their method of operation. A small polished metal mirror is cooled by a solid state Peltier thermoelectric heat pump until it reaches the dewpoint of the air within the enclosed chamber. The dewpoint of an air volume sample is the temperature at which the water vapor in the air condenses. When this temperature has been reached, condensation will begin to form on the mirror surface. The presence of condensation is detected by the reflection of an infrared light off the surface of the mirror.

Table 1. Relative Humidity and Osmotic Suction Properties of Na_2SO_4 at 25°C.

Molality (m)	Relative Humidity (P/P_o)	Osmotic Suction h_π (kPa)	Osmotic Suction h_π (log kPa)	Osmotic Suction h_π (pF)	
0.001	0.999948	7.134	0.853	1.8~~53~~ 62	+ 1.009
0.002	0.999898	13.993	1.146	2.1~~46~~ 54	+ 1.008
0.003	0.999848	20.853	1.319	2.3~~19~~ 27	+ 1.008
0.004	0.999799	27.576	1.441	2.4~~41~~ 49	+ 1.008
0.005	0.999751	34.163	1.534	2.5~~34~~ 42	+ 1.008
0.006	0.999703	40.749	1.610	2.6~~10~~ 18	+ 1.008
0.007	0.999656	47.199	1.674	2.6~~74~~ 82	+ 1.008
0.008	0.999609	53.649	1.730	2.7~~30~~ 38	+ 1.008
0.009	0.999562	60.099	1.779	2.7~~79~~ 87	+ 1.008
0.010	0.999516	66.412	1.822	2.8~~22~~ 31	+ 1.009
0.020	0.999063	128.600	2.109	3.1~~09~~ 18	+ 1.009
0.030	0.998626	188.618	2.276	3.2~~76~~ 84	+ 1.008

Reference: Goldberg, R. N. 1981.

The capacity of an air sample to absorb water vapor depends on temperature. A hotter air can hold more water vapor. An air sample holding the maximum amount of water for a given temperature is said to be saturated. An unsaturated air sample will eventually become saturated as the temperature falls. Beyond that, excess water vapor will condense to form dew or frost. At dewpoint, the partial water vapor pressure in air equals the water vapor saturation pressure. Since the correlation between water vapor saturation pressure and temperature is known, the relative humidity can be calculated from the measured temperature and the dewpoint. The WP4 instrument makes use of the dewpoint measurements to calculate the relative humidity in a closed chamber. Then, Eq. 2 is used to calculate total suction. A detailed description of the WP4 chilled mirror device and its working principle can be found in Decagon Devices, Inc. (1999).

WP4 Suction Characteristic Curve

A characteristic curve was obtained for the WP4 device using the same NaCl salt solutions that were used in the calibration of filter papers as described in previous sections. The osmotic suction values of the sodium chloride salt solutions at various molalities are shown in Table 2. The osmotic suctions in Table 2 are calculated using the following equation (Bulut et al. 2001):

$$h_\pi = -\nu RTm\phi \tag{3}$$

where h_π is the osmotic suction, ν is the number of ions from one molecule of salt (i.e., ν = 2 for NaCl and KCl), R is the universal gas constant, T is the absolute temperature, m is the molality, and ϕ is the osmotic coefficient.

Table 2. Osmotic Properties of NaCl at 25°C.

Molality (m)	Osmotic Coefficient (ϕ)	Osmotic Suction h_π (kPa)	Osmotic Suction h_π (pF)
0.000	1.00000	0.00	0.00
0.002	0.98402	9.76	2.00
0.005	0.97604	24.20	2.39
0.010	0.96804	47.99	2.69
0.020	0.95832	95.02	2.99
0.050	0.94357	233.90	3.38
0.100	0.93250	462.32	3.67
0.200	0.92387	916.08	3.97
0.300	0.92123	1370.19	4.15
0.500	0.92224	2286.15	4.37
0.700	0.92691	3216.82	4.52
0.900	0.93350	4165.31	4.63
1.200	0.94567	5626.15	4.76
1.600	0.96487	7653.84	4.89
2.200	0.99818	10887.35	5.05
2.600	1.02263	13182.03	5.13
3.000	1.04848	15594.52	5.20

Note: At the same 25°C temperature, the osmotic coefficient and osmotic suction for 0.5 Molality KCl: $\phi = 0.900$, $h_\pi = 2231.02$ kPa = 3.3485 log kPa
References: Hamer and Wu 1972, Bulut et al. 2001.

The relationship between the sodium chloride salt solution concentrations and obtained osmotic suction values are depicted in Fig. 2. In order to see and interpret the sensitivity portion of the characteristic curve more clearly, Fig. 3 is developed from Fig. 2 by magnifying the lower portion of Fig. 2 between salt solution molality of 0.0 and 0.5. As it is seen from Fig. 3, once suction falls below about 4 pF the scatter in suction data increases. The WP4 chilled mirror psychrometer is very reliable for suction measurements above about 4 pF; however, below about 4 pF suction measurements caution should be exercised in interpreting the data. Suction measurements below 3 pF, as also suggested by Wacker (2002), should be considered as error.

The WP4 is a nice, compact and very easy to use total suction measuring device. Its response time is very quick. It measures total suction between about 5 to 7 minutes.

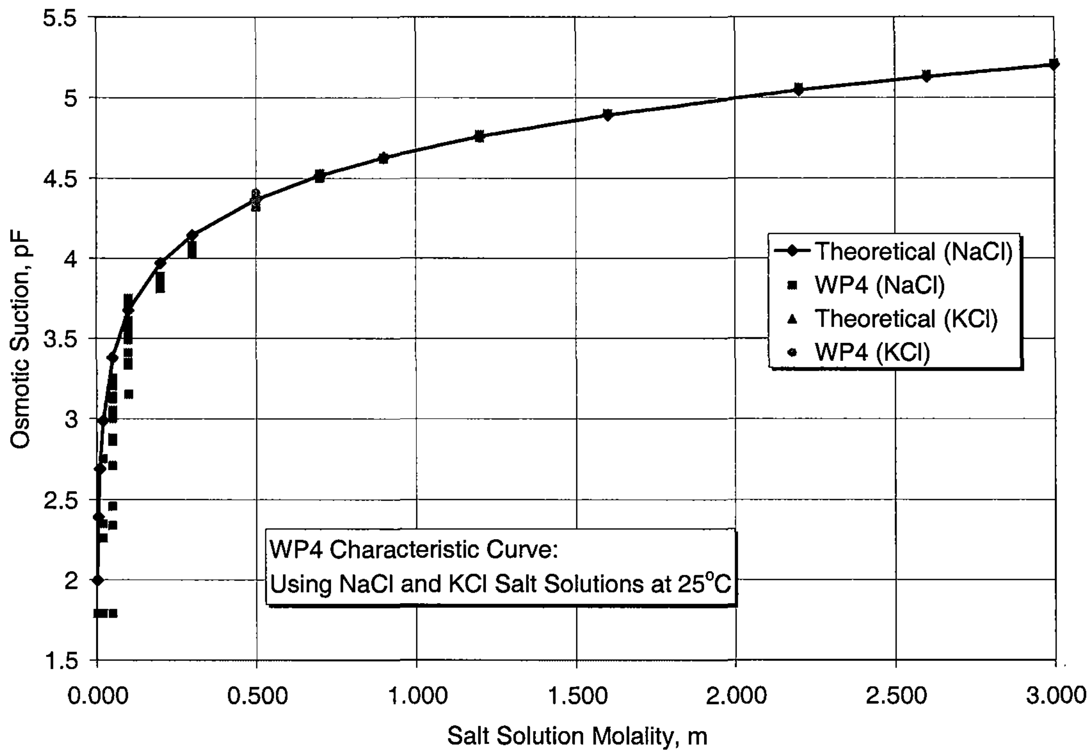

Fig. 2. WP4 Characteristic Curve 1a.

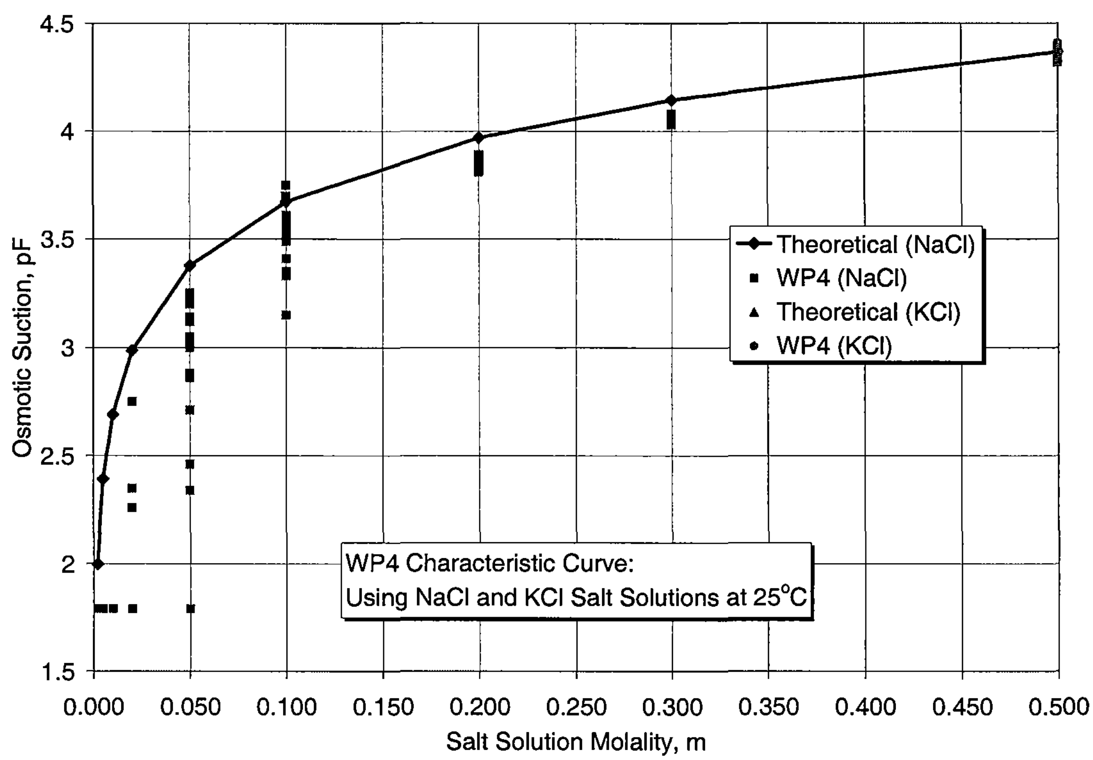

Fig. 3. WP4 Characteristic Curve 1b.

Soil Suction Measurements

Soil suction measurements were performed on the same soil samples using the filter paper method and the WP4 chilled mirror instrument. The results are summarized in Table 3 for the soil samples that were tested at high water contents (or low suction range) and in Table 4 for the soil samples that were tested at low water contents (or high suction range). As expected, based on the filter paper calibration curve and the WP4 characteristic relationship, there is scatter in data at low suction ranges. However, when compared with the WP4 results, the filter paper measurements are more consistent (Table 3). The two methods result in a good correlation of total suction measurements for the dry soil samples (Table 4).

Table 3. Soil Total Suction Measurements at Low Suction Range with the Filter Paper Method and WP4 Device.

Total Suction (pF) [log kPa ≅ pF – 1]					
Soil Sample No.	Filter Paper	WP4	Soil Sample No.	Filter Paper	WP4
1	3.36	3.57	5	3.34	3.40
1*	3.22	3.00	5*	3.44	3.07
2	3.38	2.86	6	3.56	3.49
2*	3.28	3.46	6*	3.42	3.25
3	3.30	3.25	7	3.43	3.00
3*	3.37	3.32	7*	3.56	3.31
4	3.41	3.29	8	3.54	1.79
4*	3.30	3.56	8*	3.48	3.49

*Suction sample from the same 1-foot boring soil sample.

Table 4. Soil Total Suction Measurements at Low Suction Range with the Filter Paper Method and WP4 Device.

Total Suction (pF)		
Soil Sample No.	Filter Paper	WP4
A	5.47	5.41
B	5.10	5.02
C	5.16	5.23
D	5.42	5.32
E	5.20	5.23

Discussion

As it can be seen from the filter paper calibration curve and the characteristic relationship for the WP4 device, at high water contents (i.e., in low suction range) suction becomes very sensitive to very small changes in relative humidity (or degree of saturation or filter paper water content). However, the degree of error associated with the WP4 psychrometer is higher than with the filter paper method. Although, the theoretical relationship (i.e., Kelvin's equation) between the two methods are the same and depends on water vapor pressure measurements it appears that the filter papers are somehow suppressing the error at low suction values down to about 2.5 pF, and then there is almost a sudden drop in suction with a very small change in filter paper water content. Therefore, it is necessary to mention that it is this narrow range of relative humidity that most total suction inferring devices are affected by minor temperature fluctuations. The reason for the scattering of data at high degrees of saturation is likely because at very low suction values at even very small temperature fluctuations there are alternating condensations and evaporations during equilibrium.

During total suction testing, temperature fluctuations result in alternating condensations and evaporations; therefore, the fluctuations need to be minimal and certainly less than ± 1°C. Good laboratory protocol procedures become of paramount importance in soil suction measurements. The filter paper method and the chilled mirror dewpoint technique are the two commonly employed suction measuring methods in current geotechnical engineering practice. Both techniques can be very reliable and dependable if the basic working principles of the methods are understood and a well-maintained laboratory protocol is strictly followed.

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Clay, to about 65 kPa for the 30/70% London Clay/ sand mixture, which had a liquid limit of 24%.

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The Transistor Psychrometer

A New Instrument for Measuring Soil Suction

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Abstract

The transistor psychrometer is the latest instrument available for the measurement of soil moisture suction. It has been developed over a number of years and is very similar in operation to the thermistor psychrometer which it effectively replaces. Improvements in performance have been made and the latest instrument can measure a much wider range of soil suctions in a much shorter time. Much of this improvement is due to extensive work on calibration procedures which have determined many of the characteristics of the instrument. It is also due to advances in micro-chip technology which allow amplification within the probe of the signal generated by the temperature depression of a water drop attached to a transistor.

Perhaps the greatest improvement, and the one that sets this instrument apart from any others measuring soil suction, is that the analogue output can be read by a standard millivoltmeter or logged by any millivolt data logger. The latter allows the storage, reduction and manipulation of the data to be carried out by a PC. Plotting of the output can be achieved in real time, with a logger designed for the instrument and a colour dot matrix printer. These advances have enabled the psychrometer to take its place in the modern soil mechanics laboratory.

Introduction

Development of the transistor psychrometer has taken place in several stages over the past 10 years. It has now progressed to the stage where it is being manufactured under an agreement with CSIRO in Australia and is being used in a number of laboratories around the world.

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The initial concept and working models of transistor probes were made by Paul Peter of CSIRO in the mid-1980s. This followed earlier work using thermistors as temperature sensors to measure the temperature depression of the electronic equivalent of a wet bulb thermometer. When that research ceased, the manufacturing rights were granted to John Woodburn of Soil Mechanics Instrumentation (SMI) and that organisation carried out further work and development of the probe to the stage where prototype instruments were produced. Dr. Jim Holden and his research group at Vic Roads (an Australian State Road Authority) then used one of the prototype instruments to develop standard operating procedures which gave a high degree of reproducibility in the results.

Work on the new type of psychrometer began when it became apparent that with microchip technology, a probe using transistors as heat sensors could be used for measuring relative humidity. As now developed the probe of the psychrometer consists of a "wet "and "dry" transistor with the temperature depression of the wet transistor measured and amplified within the probe. The analogue output for most soil testing has been adjusted to lie within the range 6 to 500 millivolts representing soil suctions between pF 3.0 and about pF 5.0. For conventional use the instrument is calibrated to pF 5.0 but may be calibrated to higher suctions using special procedures.

The accuracy of the instrument is dependent on the degree of ambient temperature control during the period of the test. For this reason the probes are inserted into a thermally insulated bath. This ensures that the probes and specimens remain at a near constant temperature during the period of any one test. As the probes are affected by room air temperature changes, greater accuracy and reproducibility of results is obtained within a room controlled to about $\pm$ 0.5°C. The degree of accuracy and reproducibility for soil suctions above pF 3.5 with this degree of temperature control and special operating procedures is close to $\pm$ 0.02 pF. Using standard operating procedures an accuracy of $\pm$ 0.05 pF is easily obtained.

When testing soil samples the time allowed for a test is the same as that allowed for calibration of the probes. At the present time this has been fixed at one hour. However indications are that this time for testing can be reduced without loss of accuracy. For most engineering purposes, an accuracy of $\pm$ 0.05 pF is all that is required. This order of accuracy allows room air temperature changes as high as $\pm$ 1.5°C to occur during the period of the test when testing soils above pF 4.0.

Current Techniques for Soil Suction Measurement

A number of techniques are available for soil suction measurement in the laboratory and these have been recently reviewed (Nelson and Miller, 1992). There are problems with many of the techniques due to lack of range, length of time required for equilibrium and their ability to measure only matrix suction.

Soil suction measurement is now recognised as one of the series of tests available to the geotechnical engineer practising in the field of expansive soils and Standards are available in at least two countries. The techniques currently in use have been available for many years (between 20 and 40) and have not really advanced in that time. This contrasts with present day laboratory procedures used for triaxial, swelling pressure and oedometer testing which use computer driven equipment for accurate monitoring of the test, the reduction of the results and the storage of data.

Psychrometric Technique

This technique is widely used in Australia where there is a Standard (AS 1289.2.2.1-1992) governing the test procedure. The most commonly used instrument is the Wescor Dew Point Microvoltmeter which is used with the C-51 sample chamber. The equipment is expensive in Australia, prone to damage in inexperienced hands and has an upper limit of about pF 4.5. This is often too low for testing soils sampled at the end of a drying cycle in semi-arid and arid areas. The sample chamber can suffer from problems with contamination of the thermocouple and subsequent corrosion unless care and periodic cleaning is carried out.

Filter Paper Technique

This technique has been adopted widely in the past few years and an American Standard (ASTM, 1990) is currently at the review stage. The technique has been available for many years but has recently undergone a revival with the advent of cheaper, more accurate balances and extensive work by a number of researchers. It is doubtful that the cost of a test is no more than that of a moisture content test as reported by McKeen (1992) because in addition to the moisture content determination involving measurements to 0.0001gm., it is a time consuming test. Monitoring of the filter paper weight over a period of at least a week may be required to ensure equilibrium has been reached and great care must be taken to ensure no moisture loss occurs prior to weighing. The technique is useful in that it has the ability to measure both matrix and total suction although with the latter a stable temperature environment must be provided as moisture transfer has to occur in the vapour phase. Consequently even longer times may be required for equilibrium to occur and great care must be taken to ensure that condensation inside the chamber does not affect the filter paper.

Development of the Transistor Psychrometer

Suction/Relative Humidity Relationships

A psychrometer is an instrument which measures relative vapour pressure or relative humidity. Relative humidity is related to soil moisture suction in accordance with the relationship:

$$s = \mathrm{Log}[(1.284657 \times 10^6 + 4.703\ t) \times \mathrm{Ln}(H/100)]$$

where
s = suction in pF
t = temperature in °C
H = relative humidity specified as a percentage

This equation provides an indication of the change in suction with temperature and relative humidity (Table 1). When plotted for 20°C it gives the well known curve shown in Figure 1(a). As the most relevant part of this curve to engineers and scientists interested in the flow of moisture in soils lies above 95% relative humidity, an enlargement of the upper part is shown -Figure 1(b). What this enlargement shows, and what is often not appreciated is that any instrument using psychrometric techniques must be able to measure relative humidities up to 99.9%. It must also be able to differentiate between relative humidity changes of about 0.02% at pF 3.0 (ie a suction change of 0.1 pF). This represents a temperature depression of 0.002 °C for the wetted sensor in a pair used in a similar way to a wet and dry bulb thermometer.

SUCTION vs RELATIVE HUMIDITY AND TEMPERATURE SUCTION DISPLAYED IN pF						
RELATIVE HUMIDITY (%)	TEMPERATURE (DEGREES CENTIGRADE)					
	15	20	25	30	35	40
99.90	3.132	3.139	3.147	3.154	3.161	3.168
99.50	3.832	3.839	3.847	3.854	3.861	3.868
99.00	4.134	4.141	4.149	4.156	4.163	4.170
98.50	4.311	4.319	4.326	4.333	4.340	4.347
98.00	4.437	4.445	4.452	4.459	4.466	4.473
97.50	4.535	4.543	4.550	4.557	4.564	4.571
97.00	4.615	4.623	4.630	4.638	4.645	4.652
96.50	4.684	4.691	4.698	4.706	4.713	4.720
96.00	4.743	4.750	4.757	4.765	4.772	4.779
95.50	4.795	4.802	4.810	4.817	4.824	4.831
95.00	4.842	4.849	4.857	4.864	4.871	4.878
94.50	4.884	4.892	4.899	4.906	4.914	4.921
94.00	4.923	4.931	4.938	4.945	4.952	4.959
93.50	4.959	4.967	4.974	4.981	4.988	4.995
93.00	4.993	5.000	5.007	5.015	5.022	5.029
92.50	5.024	5.031	5.038	5.046	5.053	5.060
92.00	5.053	5.060	5.068	5.075	5.082	5.089
91.50	5.080	5.088	5.095	5.102	5.110	5.116
91.00	5.106	5.114	5.121	5.128	5.135	5.142
90.50	5.131	5.138	5.146	5.153	5.160	5.167
90.00	5.154	5.162	5.169	5.177	5.184	5.191
89.50	5.177	5.184	5.192	5.199	5.206	5.213
89.00	5.198	5.206	5.213	5.220	5.227	5.234
88.50	5.219	5.226	5.234	5.241	5.248	5.255
88.00	5.238	5.246	5.253	5.260	5.268	5.275

Table 1-Suction vs Relative Humidity and Temperature

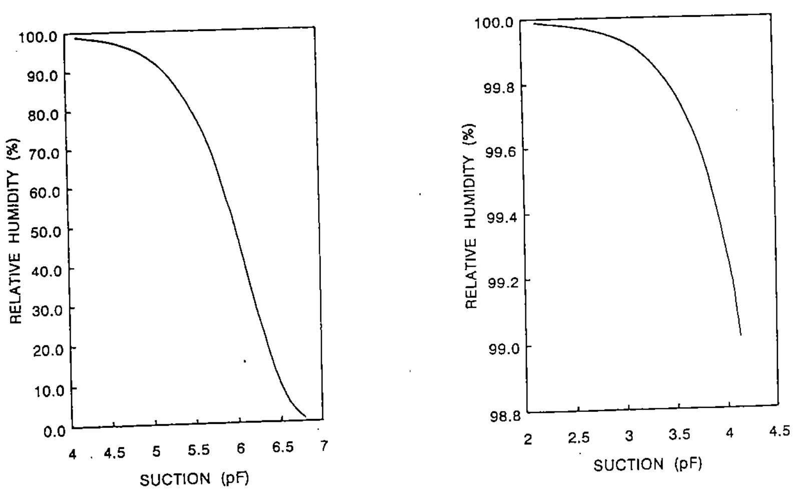

Figure 1-(a)Suction vs Relative Humidity at 20°C (b) Enlarged

There are a number of electrical components which are temperature sensitive and which can be used to measure temperature changes of this order. Some of these components register the very small temperature changes in micro-volts (thermocouples and thermistors) while others can utilise a higher voltage source and register outputs in the millivolt range (transistors and the latest range of temperature sensors).

Transistors as the Sensing Device

The CSIRO has been actively engaged in the investigation of soil suction techniques for many years and early work on psychrometers was carried out in the 1960's using thermistors. With those sensors it was found that for reasonable accuracy and reproducibility of results the test had to be carried out in a room controlled to ± 0.5 °C with a constant temperature bath controlled to ± 0.001 °C (Richards, 1965). It was also found that there was a need to accurately control the water drop size on the "wet bulb" sensor.

The idea of using transistors instead of thermistors as the temperature sensors in the psychrometer probes was first conceived by the Soil Engineering Group of the CSIRO, Division of Soils in 1984. This was partly influenced by the fact that matched pairs of thermistors were expensive, each probe needed its own power supply (1.35V mercury cell), the glass sheathing enclosing the temperature sensitive element was fragile and therefore more easily damaged and they also deteriorated with time. On the other hand transistors and integrated circuits (operational-amplifiers) were relatively cheap, robust and reliable. Advances in technology had also improved the characteristics of op-amps at that time to the extent that it was both economical and feasible to use individual op-amps in each probe.

In the final development of the transistor psychrometer the two transistors (temperature sensors), the operational amplifier and other associated components were mounted on a printed circuit board designed to fit into the shaft of the probe (Figure 2). This ensured that with the probes placed in the thermally insulated bath, the components in the probe were buffered from temperature changes occurring in the laboratory environment (Figure 3). The transistors and other components that make up each probe are carefully chosen and matched to circuit requirements. Silicon NPN type transistors are used which have a nominal base-emitter temperature coefficient of 2 millivolts/°C. This signal is amplified approximately 1000 fold with the aid of a high input impedance operational amplifier, giving an output from the probe of 2 volts/°C. Output voltages of this order can be readily monitored with a number of devices ranging from digital voltmeters and pen recorders to data loggers.

Operating Procedures and Accuracy

Prototypes of the transistor psychrometers have now been in use in the Vic Roads and CSIRO laboratories for several years. During this time a number of techniques for calibrating the psychrometer and testing soil specimens have been examined. Specific procedures and conditions have now been established which provide a high order of accuracy (i.e.up to ± 0.02 pF) in suctions over about pF 3.5. However, it has been found that an adequate degree of accuracy for most engineering purposes can be achieved using less stringent procedures and conditions.

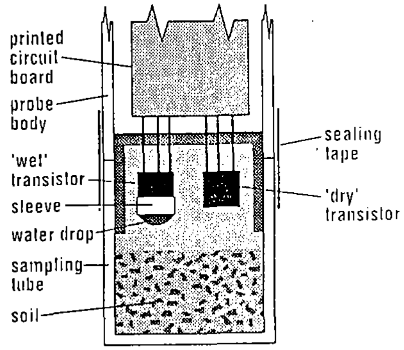

Figure 2-The Probe Tip Containing the Sensing Elements (Transistors)

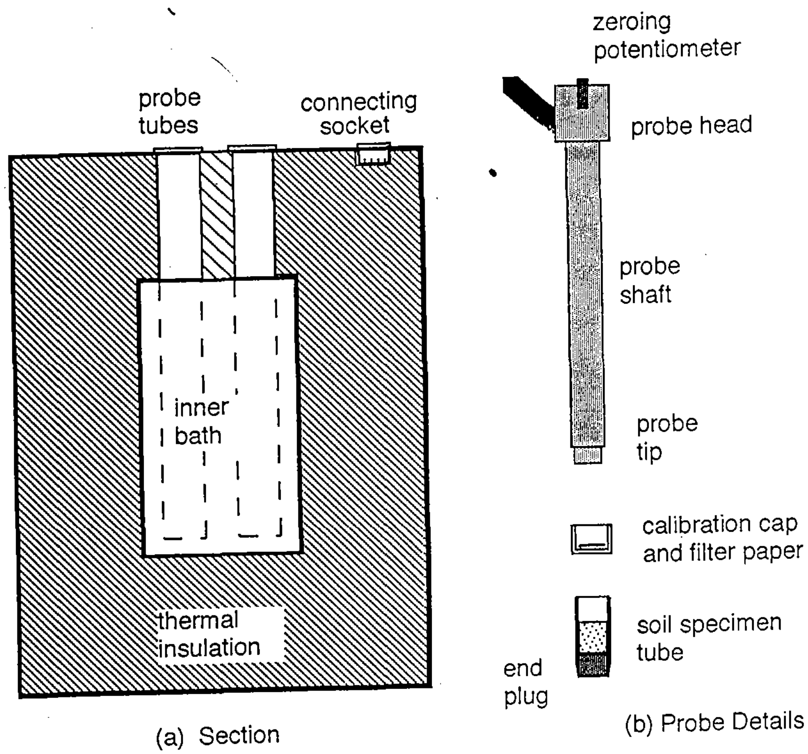

Figure 3-The Probe and Thermally Insulated Container

Calibration

Calibration of the transistor psychrometer is required because the output from the instrument is in millivolts which must be converted to a suction value if the instrument is to be used for the suction testing of soil specimens. The procedure, described in detail by Dimos (1991), is similar to the calibration of other psychrometric instruments and involves the use of standard salt solutions prepared to give equivalent relative humidities between pF 3.0 and pF 5.0. Before calibrating, the probes are zeroed with a pF 2.0 solution. A typical calibration line for a probe is shown in Figure 4.

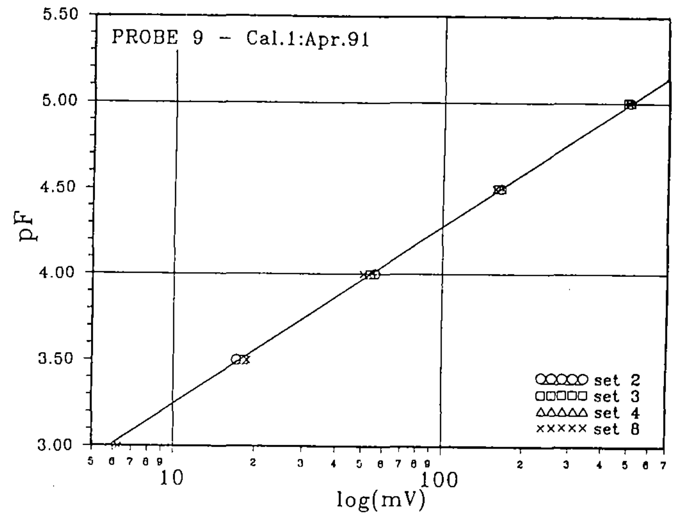

Figure 4-Calibration Line of Transistor Psychrometer Probe

The calibration line is affected to some extent by changes in the room or bath temperature, the drop shape and the size of the gap between the drop and soil surface, although these factors may only be critical when very high orders of accuracy are being achieved. Other factors are also thought to influence the calibration and it is for this reason that standard calibration procedures have been established.

Prior to calibrating and any other form of suction testing the probes must be zeroed or located on the calibration line using standard salt solutions. Early work on calibration techniques used distilled water for this purpose (H=100%) but after much work and finally dispensing with free liquids because of contamination problems, impregnated filter papers were accepted for both zeroing and calibration. When the testing program allows zeroing to be carried out overnight, filter papers impregnated with pF2.0 solution (H=99.99%) are used. Even this very high relative humidity gives a small positive output from the probe giving a more definite "zero" reading

Calibration of the probes is carried out using the PVC caps (Figure 3) which hold the filter paper discs impregnated with standard salt solutions. Before calibrating the probes, each tip must be cleaned with distilled water and inspected to ensure that the sleeve on the 'wet' transistor is at the specified height to hold the water drop (Figure 2).

Each calibration cap is prepared by placing a two-layer disc of filter paper, 15mm diameter in the base. The filter paper is then saturated using 3 drops of pF 2.0 salt solution and the cap sealed with a rubber stopper until all have been prepared. The caps are then placed on the end of each probe in turn and carefully inserted into the thermally insulated bath, after placing a standard drop of distilled water on the "wet" transistor. At this low suction the probes require many hours to reach equilibrium and are generally allowed to run overnight. In the morning the probes are zeroed prior to beginning the calibration which follows the same procedure outlined above, but using pF 3.0, 3.5, 4.0, 4.5 and 5.0 solutions in turn. For each stage the data is collected for one hour. This has been found an adequate time for equilibrium to occur and to obtain a result (Dimos,1991).

For each calibration test using a particular concentration of salt solution a graph of millivolts *versus* time is obtained. The equilibrium value obtained after one hour is the value used for the calibration line. At the completion of a set of tests using the range of standard solutions, a calibration line of pF *versus* log millivolts can be produced for each probe. This entire procedure can be conducted in a normal working day but should be repeated at least two times initially to ensure that the calibrations have been carried out correctly. When a number of sets of results have been obtained, an average calibration line can be determined for each probe.

Check calibrations should also be conducted at regular intervals once the psychrometer is in use with the maximum recommended period between them being 3 months. A calibration check should also be carried out immediately before soil testing if a very high degree of accuracy is required. A calibration line is supplied for each probe with the equipment and the calibrations obtained soon after receiving the equipment should be compared with these.

Preparation of soil specimens

Soil samples are usually obtained in sample tubes during auger drilling in the field. In the laboratory a hydraulic jack is used to push the sample out of the tube after removing the seals. When preparing the specimens care must be taken to minimise exposure of the soil to the atmosphere.

About 5mm is shaved off the end of the sample to ensure that the specimens tested are not contaminated or dried out. A 10mm high sample ring (with the same diameter as the sample tube) is placed on top of the sample tube and the sample jacked up until it protrudes about 1mm above the sample ring. At this point the ring is released and the sample jacked again until there is about 5mm showing below the ring. A sample slice including the ring is then cut off and trimmed to ring height.

At least two specimens should be obtained from each slice to allow for comparison and averaging of the results. Each specimen sampling tube (Figure 3) is pushed into the sample slice either by hand or using a small press. The position of the specimen in the specimen tube is then adjusted accurately using a specimen spacer. End plugs which have also been specially made for each of the probes are inserted in the bottom end of the tube to keep the specimen in position. Rubber stoppers are inserted in the other end of the specimen tube to eliminate any possible moisture loss. When all specimens are prepared they are immediately placed in the constant temperature room or laboratory and allowed to stand for at least half an hour to reach temperature equilibrium and to humidify the atmosphere within the specimen tube.

Testing of soil specimens

Before testing the specimens the probes must be stabilised overnight using pF 2.0 salt solution. They are then zeroed in the morning prior to placing the soil specimens. After removal of the probe and the pF 2.0 solution cap, the soil specimens are placed on the end of each probe in turn and the probes returned to the thermally insulated bath. Monitoring of the output then continues for at least one hour or until the graph of millivolts *vs* time reaches constant output. This gives a millivolt value for each probe. From this the soil suction in the specimen is obtained using the average calibration line for the probe.

If more than one set of specimens are to be tested in one day then *it is important that wet soil samples are tested before the dry samples to avoid a hysteresis effect.* When testing is complete the probes are dried, a new water drop added and stabilised overnight on pF 2.0 solution ready for testing the next day.

If results to ± 0.02 pF are required then further conditioning of the probes is required prior to soil testing by running the psychrometer with a solution of pF 3.0 for one hour. If the soil samples are likely to have a soil suction above pF 3.5, the probes should also be conditioned for a further hour with pF 3.5 solution. This conditioning is necessary in order to remove or measure the effects of drift prior to carrying out the test. This drift can be compensated at the reduction stage of the results or eliminated by comparing the millivolt value obtained during the conditioning stage with that obtained from the calibration line and re-setting the output accordingly.

Time for Equilibrium

When calibrating, the value after one hour is recorded as it is usual that a constant, or near constant reading has been achieved in that time. When testing soil specimens the reading is also taken after one hour although a longer time may sometimes be required for equilibration.

Temperature Control

The output from the probes varies with the temperature of the probes and these temperature changes are manifested as a drift in the output record. For this reason there must be a good degree of ambient temperature control during the period of the test. The purpose of the thermally insulated bath is to hold the body of the probes at a near constant temperature during the hour that the test is run. However because the head of the probe is outside the bath the probes can still be affected by room air temperature changes which produce temperature gradients along the probe shaft. It has been found that the greatest accuracy and reproducibility of results is achieved with the room temperature controlled to ±0.5°C.

The latest version of the transistor psychrometer is portable, contains 8 probes and has an insulated lid which can be closed after inserting the probes. This assists in keeping the heads of the probes at a constant temperature during the period of the test.

Accuracy

With good laboratory temperature control and the current operating procedures the transistor psychrometer is capable of measuring the total suction of a soil in the range of at least pF3.0 to pF5.0 with an accuracy of about ± 0.02 pF above pF3.5. This accuracy is greater than that required for most engineering applications. It is much greater than that specified by the Australian Standard AS 1289.2.2.1 - 1992 which states that results below pF 3.6 should be given to ± 0.1 pF and above pF 3.6 to ± 0.05 pF. To provide a check on the accuracy of the results and to enable an average to be calculated, specimens are usually tested in pairs or preferably in groups of three for greater accuracy.

Output

At least 48 specimens can be tested using the 12 probe psychrometer in a normal working day. This includes any initial conditioning of the probes with salt solutions at pF 3.0 and 3.5 and then performing 4 runs with soil samples. Without the conditioning procedures, even more samples can be tested although towards the end of the day the water drop may be depleted if most of the soils are at high suctions.

Test Monitoring, Reduction and Storage of Results

There are a number of means of obtaining and reducing the results produced by the transistor psychrometer and three options are shown schematically in Figure 5 and outlined below:

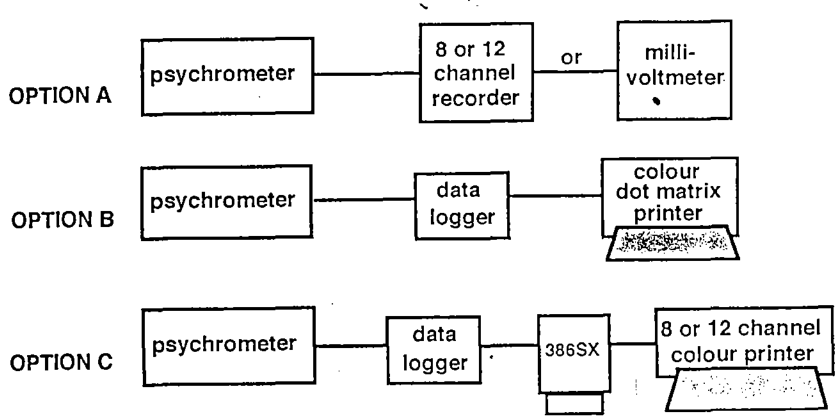

Figure 5-Psychrometer Data Recording and Manipulation.

Option 1. This system is the most simple. The results are obtained as a direct analogue output from a millivoltmeter or a millivolt recorder. The results obtained after 1 hour are recorded and the soil suction reduced from the set of calibration lines held in the laboratory.

Option 2. This is the system most commonly used. As supplied by SMI it consists of the psychrometer and data logger which has been programmed to use one of the new range of dot matrix colour printers. In the portable version the data logger is contained in the same thermally insulated case as the 8 probes and provided with printer and RS232 ports. With both systems the user has the option of a numerical or graphical output of the millivolt reading. The logger is programmed to show results from -23 to +1000mV, i.e. up to about pF 5.5. The results can be converted to suction values using the calibration lines obtained for each probe. They can also be accessed by a 386SL Notebook computer using appropriate software.

Option 3. This is the most sophisticated of the three options. It requires a software package which collects the data, presents the results on the screen during testing and allows the user to manipulate them. They can also be printed out in a graphical format with the important values given numerically once the test is complete. One such package has been written specifically for the Vic Roads psychrometer (VICroads 1992) and provides facilities to collect and store data, enable past data to be retrieved and to display the data collected in a number of formats. The procedure involved in the use of the computer package is described in detail by Dimos (1991).

Discussion

It has now been established that the transistor psychrometer is an accurate device for the measurement of the total water potential of soils. Work on the instrument is continuing with tests now being carried out on a more portable 8-channel model. It is believed that this will be suitable for field laboratory applications where high accuracy below about pF 3.75 is not required. One of the potential uses of this portable instrument lies in the control of the moisture condition of road fills and subgrades at the time of placement and compaction. These materials are often of high reactivity to moisture changes and should be placed at their equilibrium suction value. This would help to reduce the pavement cracking which often occurs with roads built on these soils.

Another possible use of a more portable instrument is in the determination of suction profiles without the need for actual sampling. Indications are that the technology now developed can be placed within a penetrometer and suction values obtained by staging the probe at various levels as it is forced into the ground. This would allow a suction profile to be measured to a depth of at least 6 meters in a relatively short time.

Conclusions

During the past three years much knowledge has been gained in the operation, use and calibration procedures required for the transistor psychrometer. It is now regarded as a reliable instrument for the measurement of soil suctions to better than the order of accuracy required for engineering applications and the current Standards. The instrument should be readily accepted in the modern soils laboratory because the process of measurement and reduction of the results can be carried out using data loggers and PCs with either currently available or customised software.
There is the potential to extend the use of the instrument to such applications as the placement and compaction of expansive clays at their equilibrium suction value and the determination of suction profiles without the need for actual sampling. Work is continuing on the evaluation of these applications.

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The views expressed are those of the authors and do not necessarily represent those of their parent organisations.

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MODELLING THE BEHAVIOUR OF COMPACTED SOILS

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Abstract

Predictions based on a recently developed model for unsaturated soil behaviour have been compared with some results of two experimental programs on compacted soils recently published in the literature. In both programs compacted specimens were subjected to loading and wetting sequences. It is shown that experimental observations are consistently reproduced by the model. This success has led to a simplified framework for compacted soil behaviour in which model parameters represent soil type and compaction method whereas the as compacted dry density and moulding water content are conveniently described by initial preconsolidation stress and water suction.

Introduction

The concepts behind compaction were firmly etablished in the papers published by Proctor (1933) and have not experienced significant revisions. Two variables, dry density and compaction water content, are universally used to describe the compaction state of a given soil. A third component, microstructure, is also widely recognized as an important piece of information to explain the behaviour of compacted soil despite the fact that it lacks a simple quantitative descriptor to be used in practice. Field experience and an extensive laboratory research carried out in many parts of the world indicates that compacted soils may either compress (collapse) or swell when saturation increases. The first type of behaviour is inconsistent with the tenets of a single effective stress. On the other hand swelling upon wetting is strongly dependent on the particular sequence of loading and wetting imposed to the specimen. These conceptual difficulties explain the widespread use of empirical approaches and descriptive case-oriented studies when the behaviour of compacted soils is analyzed.

Two important concepts to understand the behaviour of compacted soils are the negative pressure (or suction in more general terms) of the pore water and the relationship between water and microstructure. The significance of these concepts will be described in the next section. They are a key part in recent efforts of the authors to provide a consistent framework to describe the behaviour of partially saturated soils (Alonso, Gens and Hight, 1987; Alonso, Gens and Josa, 1990; Josa, Balmaceda, Gens and Alonso, 1992; Gens and Alonso, 1992). Compacted soils belong to this class of soils. In fact, most of the experimental basic research on unsaturated soils has been carried

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Other post-graduate experience includes approximately 30 credit hours at the University of Texas at Austin in the areas of soil structure interaction and geotechnical engineering.

BUSINESS EXPERIENCE

1959 through 1962 - United States Air Force, primarily as a base civil engineering officer.
1962 through 1964 - Frank G. Bryant & Associates, a testing laboratory in Austin.
1964 to present – Owner and Senior Consultant in MLAW Consultants and Engineers. Structural Engineer of Record on hundreds of commercial buildings, apartments, custom homes, retaining walls and over 30,000 single family residences. Civil Engineer of Record for hundreds and development projects.
1985 to present - Owner, Senior Consultant in MLA Labs, Inc. Geotechnical Engineer of Record for over 10,000 site investigations for buildings, bridges, subdivisions, pavements, utilities, industrial facilities and utility plants.
1996 to present - Owner, Chairman of Geostructural Tool Kit, Inc., a software development and applications consulting company serving engineers in the areas of soil-structure interaction and unsaturated soil mechanics.

PUBLICATIONS

Author or co-author 18 technical papers dealing with expansive clay, foundations, pavements and forensic engineering.

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American Society of Civil Engineers - Fellow
American Society of Civil Engineers, National Standards Committee—Design of Residential Structures on Expansive Soils - Member.
American Society of Civil Engineers, National - Member Technical Council on Forensic Engineering
Texas Board of Professional Engineers - Chairman of Sub-Committee - Initial Design of Residential Foundations
Texas Section – American Society of Civil Engineers, Residential Foundation Investigation and Design & Residential Foundation Evaluation & Repair Sub-Committees member, January 2000 – Present.
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GEOTECHNICAL INVESTIGATIONS AND REPORTS

DO NOT TRIVIALIZE GEOTECHNICAL STUDIES

- WITHOUT A CAREFULLY EXECUTED GEOTECHNICAL INVESTIGATION, ALL FOUNDATION DESIGN WORK IS EITHER OVER-DESIGNED OR UNDER-DESIGNED AND GENERALLY ONLY HIGH PRICED DRAFTING BASED ON GUESSWORK. MUCH OF THE HARD THINKING AND JUDGEMENT NEEDED FOR GOOD DESIGN IS SQUARELY ON THE GEOTECHNICAL ENGINEER.

EXPANSIVE OR NON-EXPANSIVE SITE OR ZONE?

3.2.1 Expansive Soils Sites

Sites for which expansive soil design is applicable should satisfy 3.2.1.1 through 3.2.1.3, or 3.2.1.4. Tests showing compliance with 3.2.1.1 through 3.2.1.3 are not required if the test prescribed in 3.2.1.4 is conducted. This is consistent with the expansive soil classification found in the *International Building Code* (IBC) 2003 Section 1802.3.2[50].

3.2.1.1 - Plasticity Index (*PI*) of 15 or greater, determined in accordance with ASTM D 4318.

3.2.1.2 - More than 10 percent of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D 422.

3.2.1.3 - More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.

3.2.1.4 - Expansion index (*EI*) greater than 20 determined in accordance with ASTM D 4829.

FIELD INVESTIGATION

3.3 Minimum Field Investigation Program

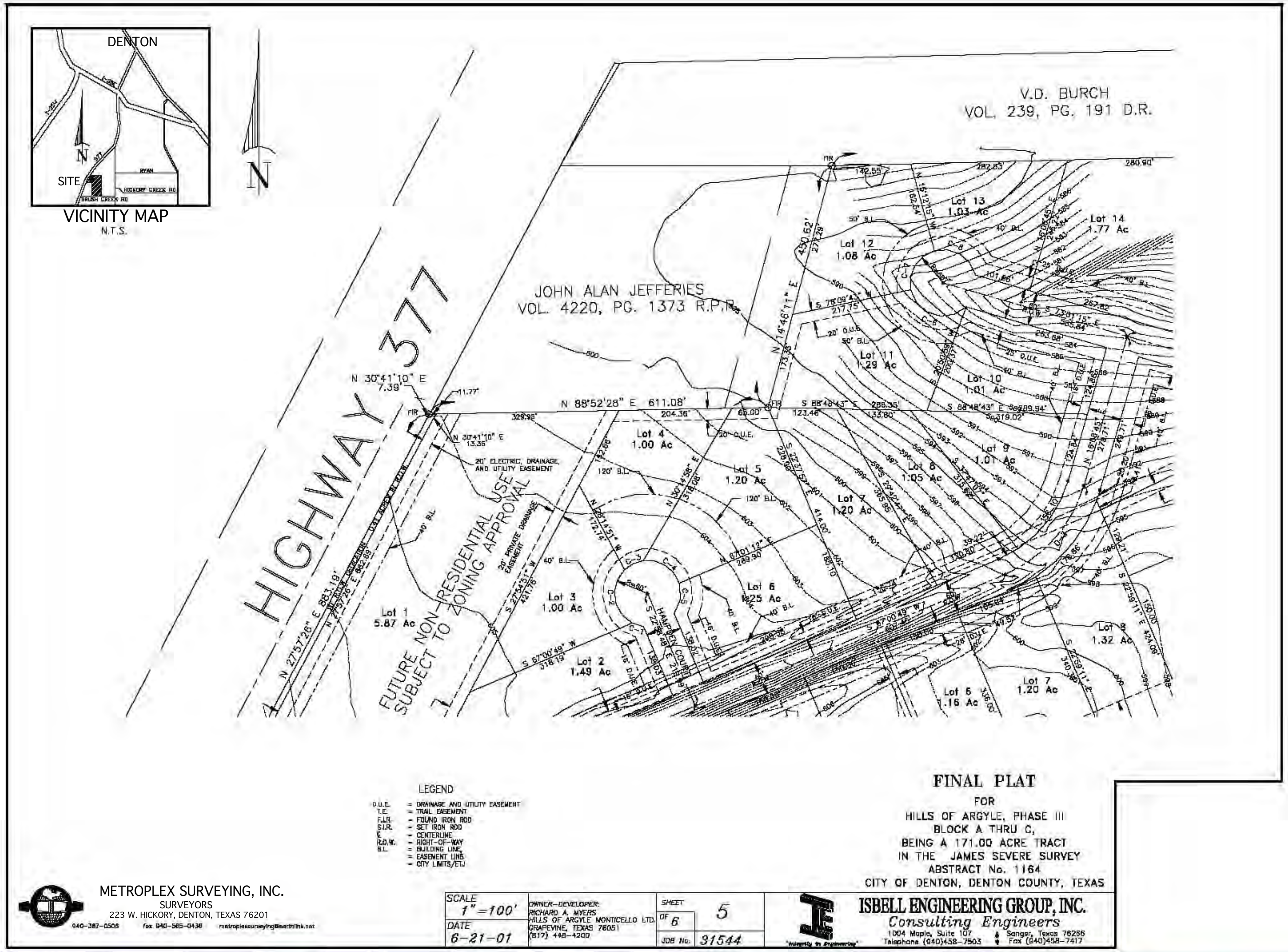
The geotechnical engineer should develop the proposed exploration program. A minimum exploration program for subdivisions should cover the geographic and topographic limits of the structural areas, and should evaluate expected differences in geology in sufficient detail to provide information and guidance for secondary investigations, if any.

Borings should generally extend through any known fill or potentially compressible materials even if greater depths are required. If grading information is available, the exploration program should be designed to consider the effects of significant cuts so that additional boring depths may be required in areas of these cuts.

[redacted] The geotechnical exploration program should take into account site conditions and subsurface anomalies, such as vegetation, depth of fill, drainage, seepage areas, slopes, fence lines, old roads or trails, man-made constructions, the time of year regarding seasonal weather cycles, perched water seepage and other conditions that may affect foundation performance. A representative series of aerial photographs are recommended to identify site conditions before subdivision grading is initiated.

Special Project Requirements

Plan and Area Topography



In general, for expected uniform subsurface conditions, borings should be placed at 200-foot centers across a subdivision, or as determined by the geotechnical engineer of record based upon local practice. Non-uniform subsurface conditions may require additional borings.

A single lot investigated in isolation should generally include two borings. Borings should generally be a minimum of 20 feet in depth unless confirmed extensive non-swelling rock strata are encountered at a lesser depth. The borings depth can be reduced to 15 ft in any area with uniform soil deposits and a positive Thornthwaite Moisture Index of +15 or greater, such as the Texas Gulf Coast region. [redacted]

Borings should generally extend through any known fill or potentially compressible materials even if greater depths are required. If grading information is available, the exploration program should be designed to consider the effects of significant cuts so that additional boring depths may be required in areas of these cuts.

Special Project Requirements

All borings should be sampled at a minimum interval of one per two ft of boring in the upper 10 ft and at 5-ft intervals below that. In clayey soil conditions, relatively undisturbed tube samples should be obtained. In granular soils, samples using Standard Penetration Tests should be obtained. Borings should be sampled and logged in the field by personnel properly trained in geotechnical testing procedures and all borings should be sampled such that a geotechnical engineer may examine and confirm the field logs in the laboratory.

Exploration may either be by drill rig or by test pit. Sites, which are obviously rock with outcrops showing or easily discoverable by shallow test pits, may be investigated and reported without resorting to drilled borings, and terminate at depths sufficient to identify rock.

Field logs should generally note inclusions, such as roots, organics, fill, calcareous nodules, gravel and man-made materials. If encountered, the depth to water should be logged. If the geology or site conditions indicate, overnight water levels should be recorded prior to backfilling boreholes. Additional measurements should be taken at the direction of the geotechnical engineer. The impact of perched water seepage should be considered if the geology of the site dictates.

LABORATORY TESTING

3.4 Suggested Laboratory Testing Program

The geotechnical engineer should develop the laboratory testing program. Sufficient laboratory testing should be performed to identify significant strata and soil properties found in the borings across the site. Such tests may include:

a. Dry Density

b. Moisture Content

c. Atterberg Limits[7]

d. Estimates of Cohesive Strength using Pocket Penetrometer or Torvane

e. Confined or Unconfined Compressive Strength tests

f. Swell and/or Shrinkage Tests

g. Hydrometer Testing to obtain percentage of smaller than 2 microns sizes

h. Sieve Size Percentage through #200

i. Soil Suction Tests

j. Consolidation - Swell Pressure Testing

k. Expansion Index Test

GEOTECHNICAL REPORTS

3.5 Geotechnical Report

3.5.1 Report Contents

Geotechnical reports should contain, as a minimum:

a. Name and address of firm preparing the report
b. Detailed location (with limits) of site being investigated
c. Purpose and scope, and limitations of services
d. Project description, including design assumptions

e. Investigative procedures

f. Laboratory testing procedures

g. Laboratory testing results (to include swell test)

h. Logs of borings and plan showing boring locations

i. Site characterization

j. Foundation design information and recommendations

k. Percentage of water-soluble sulfates and chlorides in soil, by weight, if required by local governing authorities

l. Professional Engineer's seal

m. Aerial photos before site grading

3.5.2 Site Characterization

The geotechnical engineer should characterize the site for design purposes. The report should comment on site conditions and subsurface anomalies that may affect the foundation design and performance, such as:

a. Topography including drainage features and slopes
b. Trees and other vegetation
c. Seeps
d. Stock tanks
e. Fence lines or other linear features

f. Geotechnical conditions, including estimated swell-shrink or consolidation type settlement

g. Active or inactive surface faults, if applicable

h. Subsurface water conditions

i. Areas of fill detected at the time of the investigation

j. Other man made features

Site Information

3.5.3 Foundation Design Information and Recommendations

Reports should contain the applicable design information and recommendations for each lot or structural area in the project. Site characterization should be provided to determine if expansive soils, compressible soils or inactive ground conditions control foundation design, for example:

A. Expansive Soils Conditions

1. The estimated depth of the moisture active zone.
2. Suction profiles, in-situ and recommended design envelopes
3. Post-Tensioning Institute (PTI) parameters including the following:

a. e_m and y_m for edge lift and center lift modes. The e_m and y_m design recommendations should take into account the added effect of trees and other environmental effects including subsurface anomalies such as perched water. The determinations should be based on design suction profile change and laboratory determined values of suction-compression index. See Figure 3.5 for a graphical display of the e_m and y_m concepts.

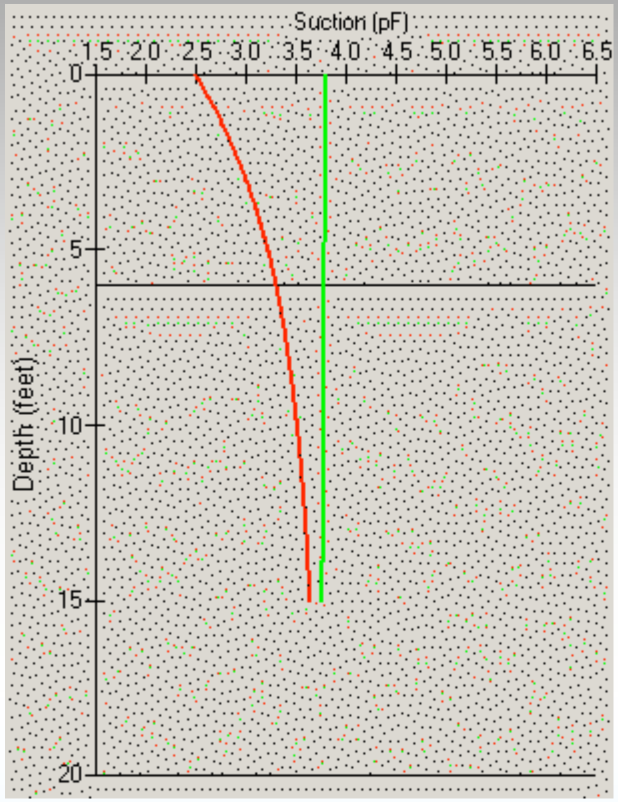
b. Bearing capacity of the soil to be applied at the bottom of stiffener ribs or across the entire width of uniform thickness foundations.

c. e_m and y_m should be reported for design conditions for suction profiles varying from equilibrium, and for probable extreme suction conditions.

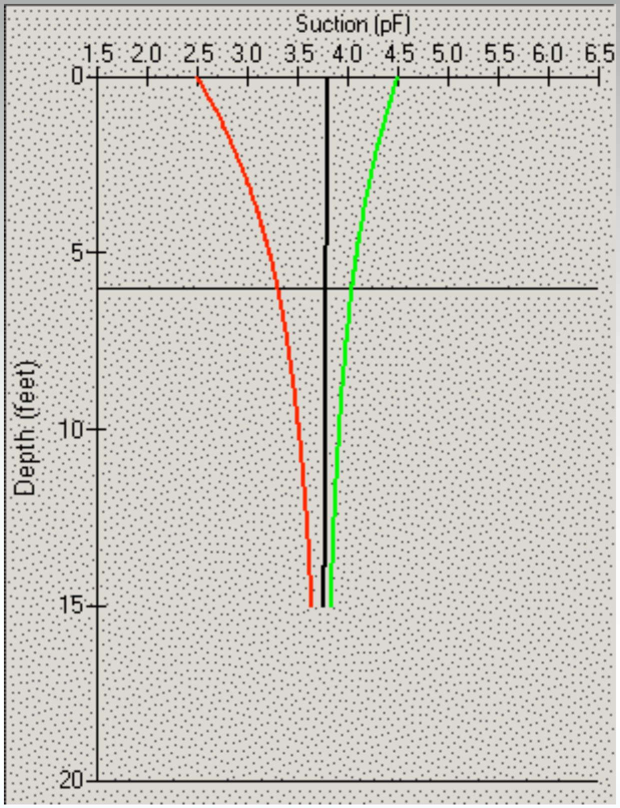
5. Soil treatment method(s) to reduce the soil movement potential and the corresponding reduction in predicted movement.
6. Methods for dealing with trees and other site environmental concerns that may affect the foundation design.
7. Moisture control procedures to help reduce soil movement.
8. Surface drainage recommendations to help reduce soil movement.

SUCTION PROFILES

Post Equilibrium Suction Profile



Post Construction Suction Profile

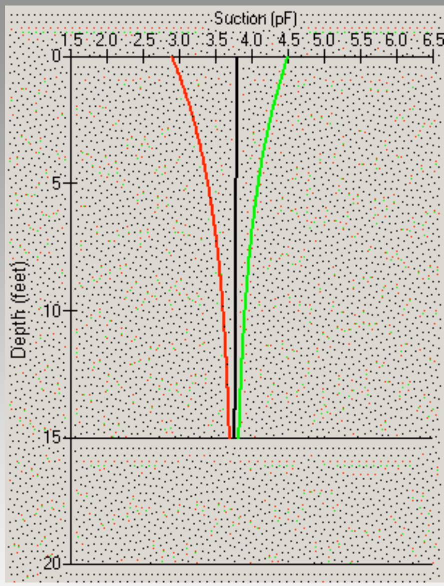


POST-EQUILIBRIUM VS POST-CONSTRUCTION SOIL STATES

POST-EQUILIBRIUM IMPLIES A LONG TIME PERIOD AFTER WHICH SOIL MOISTURE (OR SUCTION) IN THE CENTER OF A SLAB STABILIZES AND ONLY THE EDGES MOVE UP OR DOWN IN RESPONSE TO WETTING OR DRYING. **(ORIGINAL PTI ASSUMPTION)**

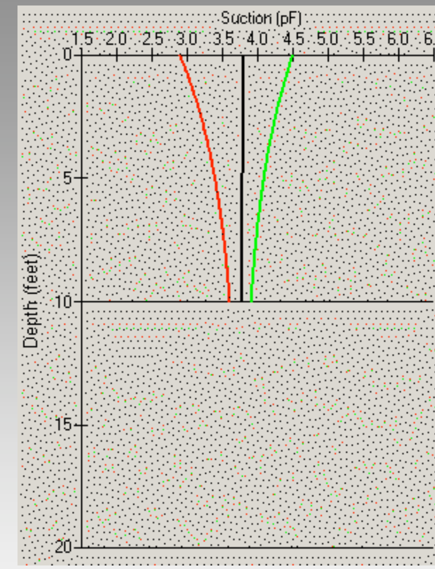
POST-CONSTRUCTION SOIL MOISTURE STATES (SUCTION) CAN EXIST DURING CONSTRUCTION AT EXTREME WET OR DRY VALUES AND CHANGE TO THE OPPOSITE WITHIN THE FIRST SEVERAL YEARS OF A SLAB'S LIFE. THIS RESULTS IN MUCH MORE EXTREME EDGE LIFT OR EDGE SHRINK AND MAY CONTROL PERFORMANCE.

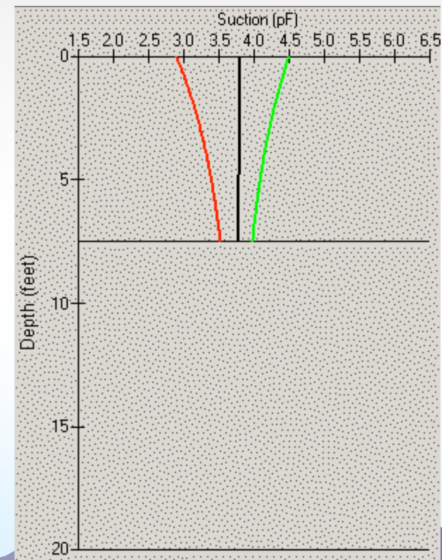
DESIGN SUCTION ENVELOPES REFLECTING VARIOUS DEPTHS OF NON-ACTIVE ZONE ON $Y_{M\ SWELL}$ (POST-CONST.)



Non-Active Zone at 15 Feet

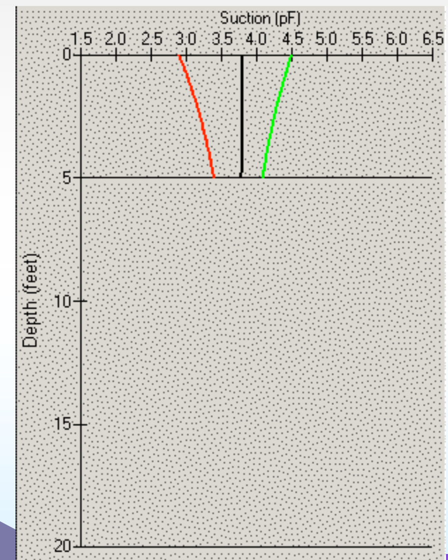
Non-Active Zone at 10 Feet





Non-Active Zone at 7.5 Feet

Non-Active Zone at 5 Feet



GEOTECHNICAL REPORTS EMPHASIS ITEMS

1. Tube samples at 2' to 10' depth and at 5' below that, in borings at 200 ft spacing or at special interest areas. Fill in between with auger borings and samples.

2. For each soil layer, run A.L., %fc, F_f.

3. Identify fau lts.

4. Identify wooded areas to be cleared. Routinely use aerial photos.

5. Clearly indicate "Non -Expansive" areas for Type II Recommendations. Site preparation i s important.

6. Select and report Design Suction Envelopes for each subdivision zone or commercia l site. Use field suction profiles to verify envelopes. Tree areas or shallow water tables may require rectangular envelope. As a general rule use "Post -Construction" profiles within I_m -20 to +10. For controlled moisture conditioned sites, "Post -Equilibrium" may be used.

7. Long term f ield suction data base is needed to refine typical Design Suction Envelopes for various localities and conditions. Until better info is available, use pF 2.9 to pF 4.5.

8. For expansive clay sites, include table of revised e_m based on vertical barrier depths. Use table in PTI 3rd Ed. Revised y_m for barriers can be modeled using VOLFLO.

9. Two methods for estimating swell in all expansive clay reports. Note that PVR is NOT Ym.

10. Run VOLFLO and include outputs of runs in appen dix.

DO NOT TRIVIALIZE GEOTECHNICAL STUDIES

- WITHOUT A CAREFULLY EXECUTED GEOTECHNICAL INVESTIGATION, ALL FOUNDATION DESIGN WORK IS EITHER OVER-DESIGNED OR UNDER-DESIGNED AND GENERALLY ONLY HIGH PRICED DRAFTING BASED ON GUESSWORK. MUCH OF THE HARD THINKING AND JUDGEMENT NEEDED FOR GOOD DESIGN IS SQUARELY ON THE GEOTECHNICAL ENGINEER.

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Master of Science in Civil Engineering (1998)
University of Texas at Austin, Department of Structural Engineering
Bachelor of Science in Architectural Engineering (1992)
University of Texas at Austin
Attendance at numerous seminars and technical continuing education courses in the areas of geotechnical, structural and foundation engineering

Experience
1996 - Present – ***Owner, Director, Officer and Chief Software Programmer***, Geostructural Tool Kit, Inc.
1994 - Present - ***Engineer***, MLAW Consultants and Engineers
1993 - 1994 ***Teaching Assistant***, University of Texas at Austin, Structural Engineering Department.
1993 - 1994 ***Hydrologic Technician***, United States Geological Survey, Water Resource Division.
1984 - 1986 ***Draftsman / Construction Assistant***, Doyle Watson Homes.

Publications
"Support Parameters for Slabs on Ground on Expansive Clay with Vegetation Considerations," Conference Proceedings, American Society of Civil Engineers, Houston, Texas, October 2001 with Kirby Meyer
"Case Studies of Fifty Slabs On Ground on Expansive Clay and Implications for Future Design," Conference Proceedings, Texas Section American Society of Civil Engineers, Arlington, Terxas, Spring 2002 with Kirby Meyer

Affiliations, Professional & Community Activities
American Concrete Institute - Member
American Society of Civil Engineers - Member
National Society of Professional Engineers - Member
Post Tensioning Institute – Slab-on-Ground Committee, Structural Sub--Chair
Texas Exes – Life Member
Triumphant Lutheran Church (offices held – Technology Committee – Chair)

1:00 PM – 2:30 PM
READ & MEYER

GEOTECHNICAL PROCEDURE

Application of Geotechnical and Structural Procedures for Expansive Soil Using PTI 3rd Edition Manual

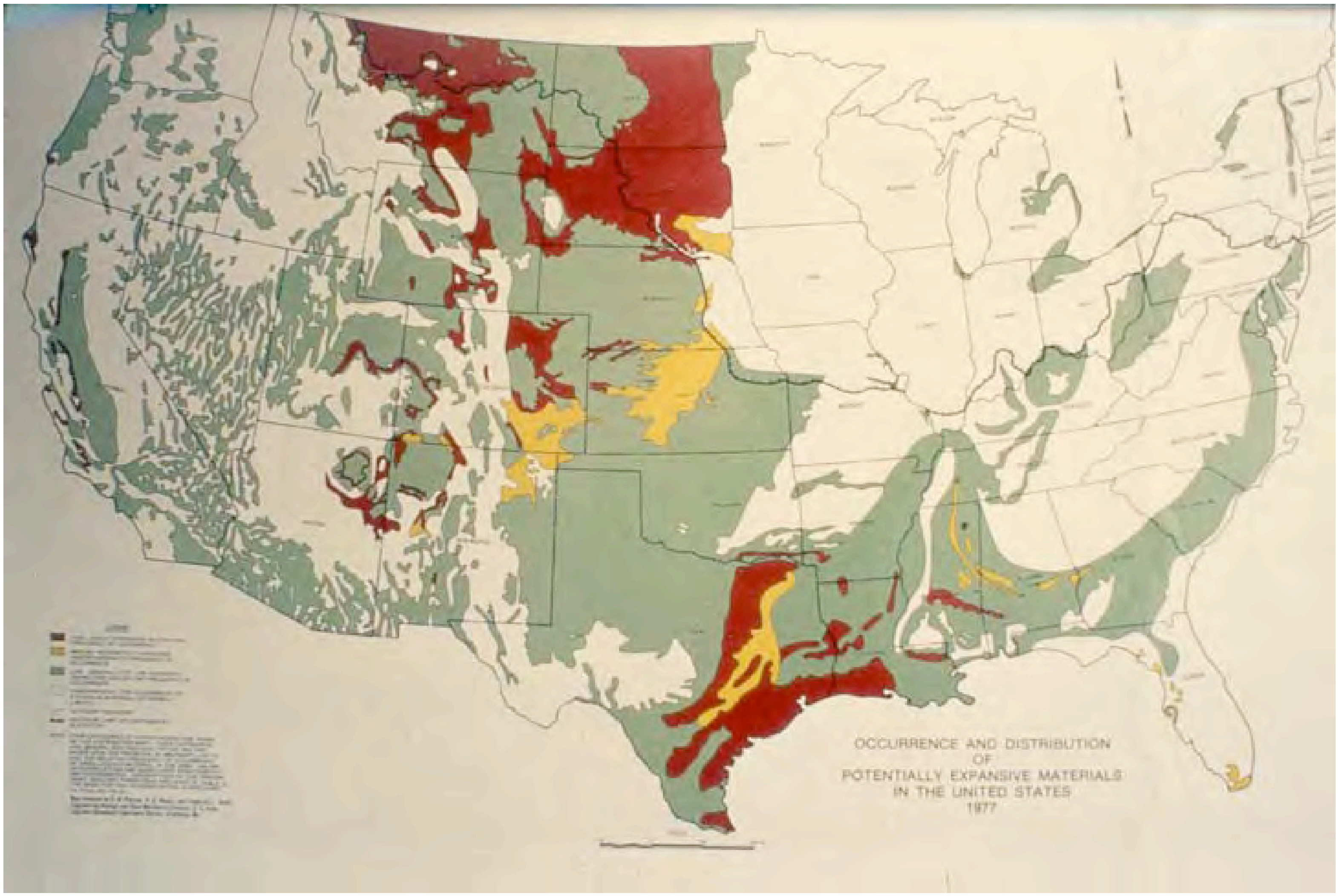
September 2005

Houston, TX

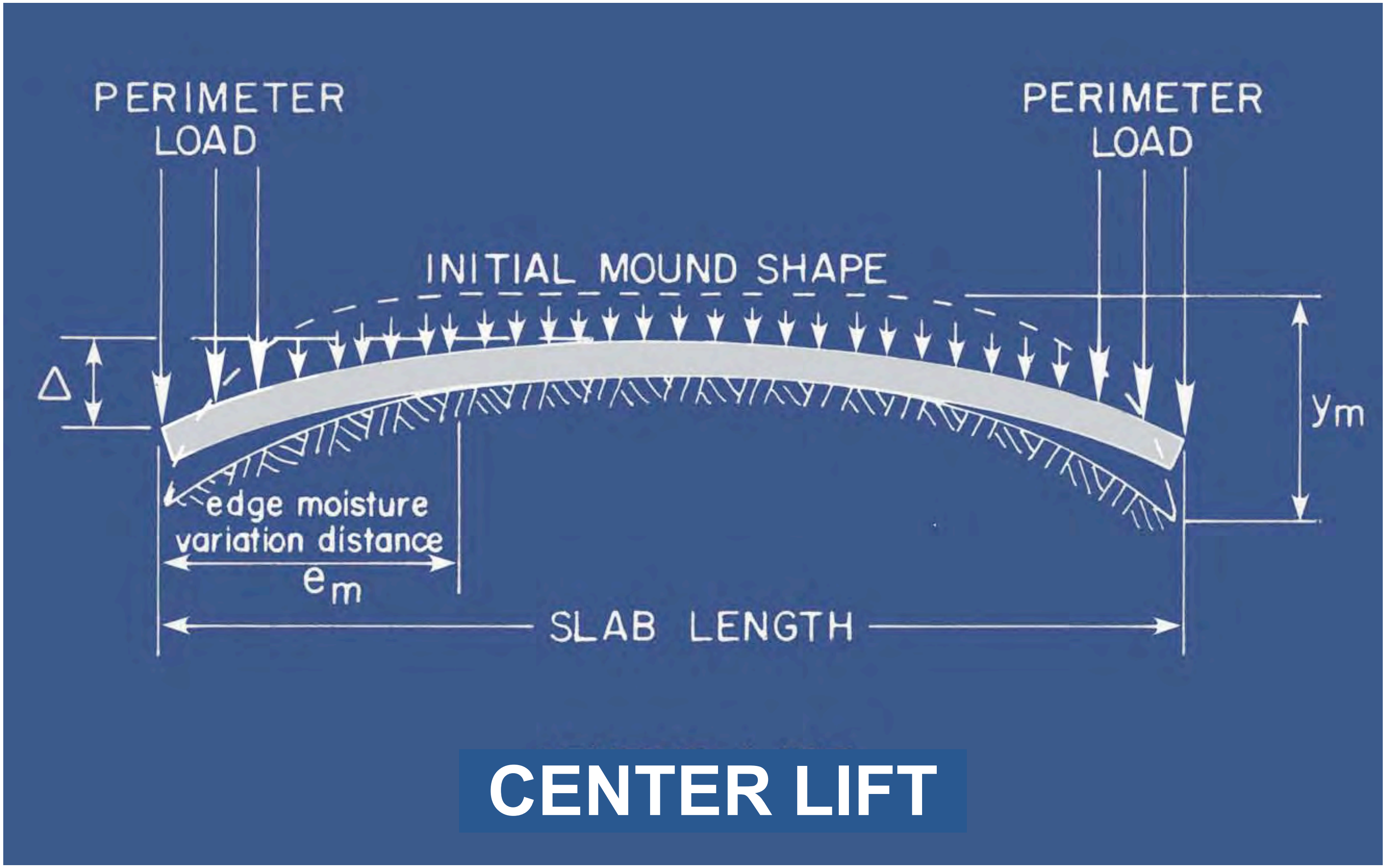
Expansive Soil?

- PTI 3.2.1 - Expansive Soil Design is applicable if:
  - All three of the following are true
    - PI >= 15
    - Passing #200 Sieve > 10%
    - Finer than 5 micron > 10%
  - Or EI > 20
- PTI 4.1 (Commentary) – The design method for slabs on expansive soils is applicable for:
  - PI >=15 in the upper five feet

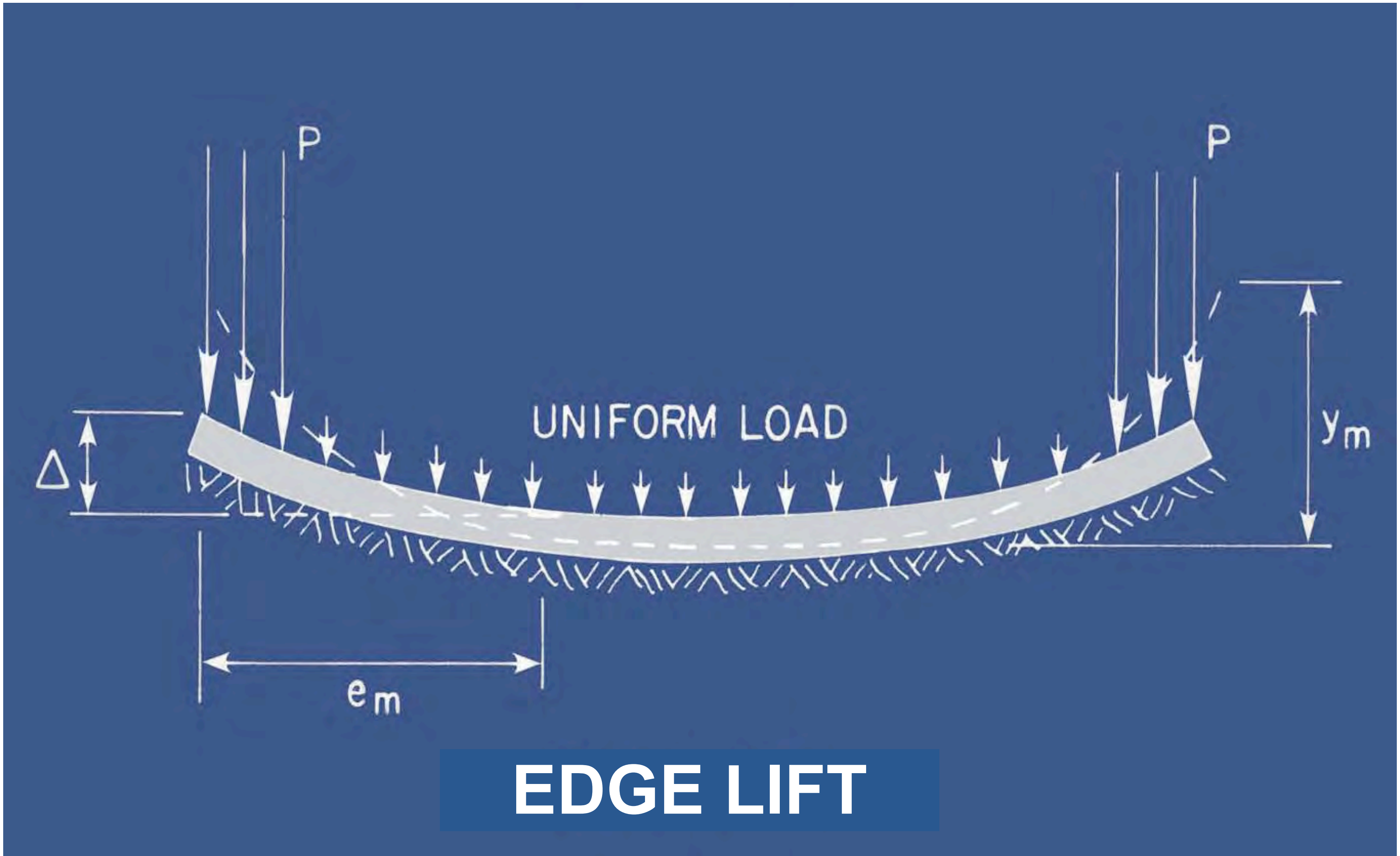
Distribution of Expansive Soils



Soil Structure Interaction



Soil Structure Interaction



Edge Moisture Variation Distance, e_m

- Edge Moisture Variation Distance - e_m represents the distance measured inwards from the edge of a shallow foundation within which moisture will change due to wetting or drying influences around the perimeter of the foundation.

- e_m is based on both climatic and soil properties.

Differential Soil Movement, y_m

- Differential Soil Movement - y_m represents the change in soil surface elevation at two locations separated by a distance e_m.
- y_m can be determined using the Stress Change Factor (SCF) method or computer methods such as VOLFLO.

Differential Soil Movement, y_m

- y_m is NOT the expected differential deflection of the foundation. y_m should always be greater than the actual differential deflection of the foundation due to foundation stiffness.
- y_m would only equal the differential deflection for a “perfectly flexible” foundation with no externally applied loads.
- y_m is NOT the same as Potential Vertical Rise (PVR) . PVR is a commonly used swell predictor used in Texas.

REQUIRED Variables for determining e_m and y_m

- Liquid Limit (LL) **- 90**
- Plastic Limit (PL) **- 29**
- % Passing #200 Sieve **- 89**
- % Finer than 2 micron **- 29**
- Geographic Location **- Houston, TX**
- Depth to Constant Suction **- 9 feet**
- Fabric Factor (Ff) **- 1.0**

Note: With the exception of the Fabric Factor, the required inputs were all required in the 2nd Edition procedure.

Depth to Constant Suction

- The Depth to Constant Suction can be estimated by several different methods:
  - Published analytical procedures
  - The depth at which the suction changes less than 0.027 pF (difficult to measure to this accuracy)
  - 2 feet deeper than the deepest root
  - Depth of “moisture active zone” (difficult to determine, can vary on different sites)
- While the Depth to Constant Suction is commonly assumed to be 9 feet it can be significantly deeper.

Additional Variables for determining e_m and y_m

- % Passing #10 Sieve
- Dry Unit Weight **(at natural water content)**
- Wet Total Unit Weight **(at approx. 2.5 pF)**

Example of e_m Determination

Steps to determine e_m

- Step 1 – Calculate the Plasticity Index (PI)
- Step 2 – Calculate the % fine clay (%fc)
- Step 3 – Determine Zone from Mineral Classification chart
- Step 4 – Calculate the Activity Ratio (PI/%fc)
- Step 5 – Calculate LL / %fc
- Step 6 – Determine Suction Compression Index (γ_o) from Zone Charts
- Step 7 – Calculate Suction Compression Index (γ_h)
- Step 8 – Modify γ_h for shrinking and swelling

Steps to determine e_m

- Step 9 – Calculate S_s
- Step 10 – Calculate Unsaturated Diffusion Coefficient (α)
- Step 11 – Calculate Modified Unsaturated Diffusion Coefficient (α')
- Step 12 – Calculate Weighted Modified Unsaturated Diffusion Coefficient (weighted α')
- Step 13 – Determine Thornthwaite Moisture Index (I_m)
- Step 14 – Determine e_m

e_m Step 1 & 2
Calculate PI and %fc

- Step 1 – Calculate Plasticity Index (PI)

  PI = LL – PL

  PI = 90 – 29 = 61

- Step 2 – Calculate the % Fine Clay (%fc)

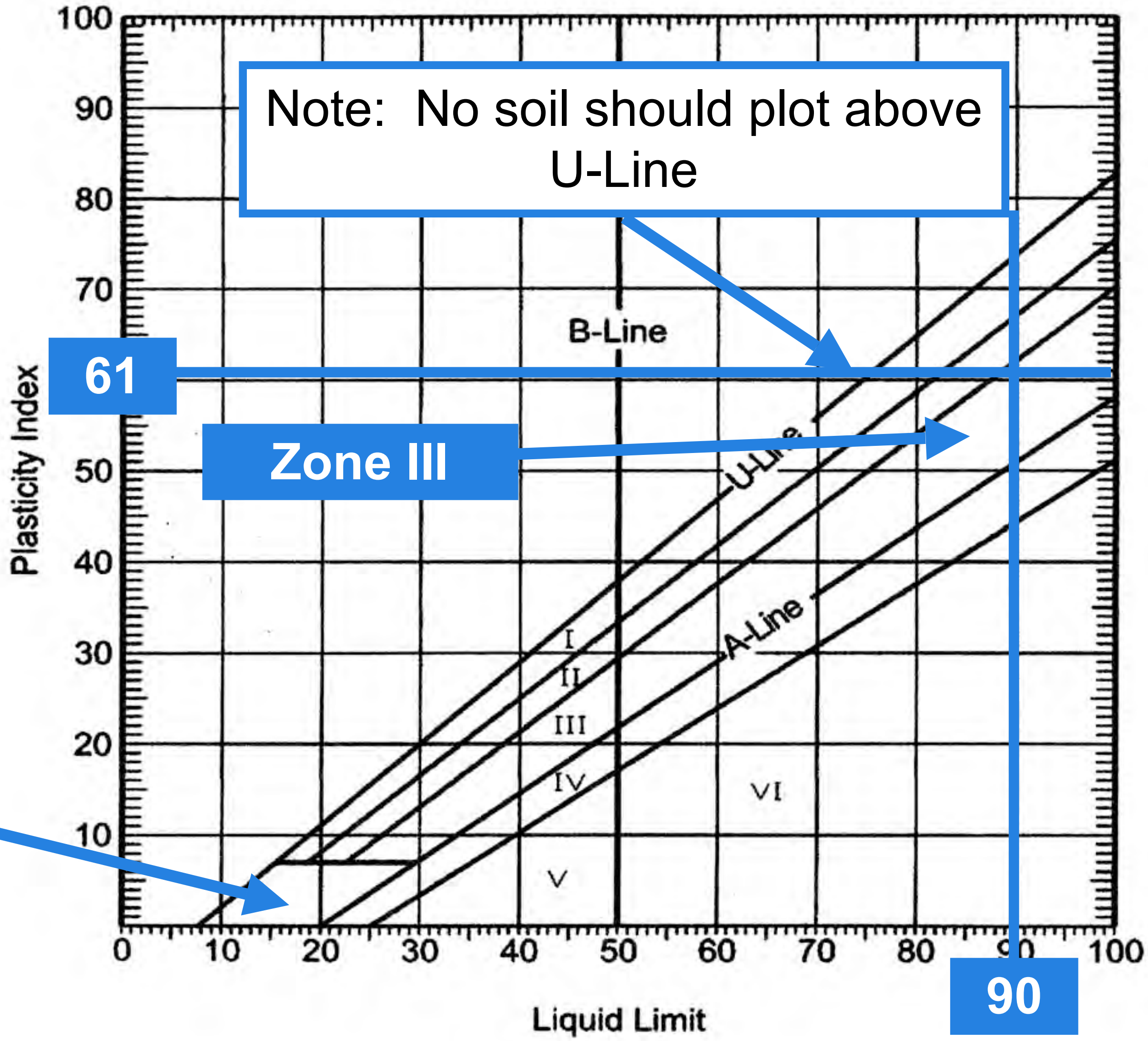
  $$\%fc = \frac{\%\ \text{finer than 2 micron}}{\%\ \text{passing \#200 sieve}}$$

  %fc = (65 / 89) * 100 = 73%

Note: Percent Fine Clay is not the same as Percent Clay as published in other sources

e_m Step 3

Determine Zone from Mineral Classification Chart



Referred to as Zone 7 in VOLFLO. There is no Zone Chart for this area. PTI 3.6.2 says to use $\gamma_o = 0.01$

e_m Step 4 & 5
Calculate PI/%fc and LL/%fc

- Step 4 – Calculate Activity Ratio (PI / %fc)

  PI / %fc

  PI / %fc = 61 / 73 = 0.84

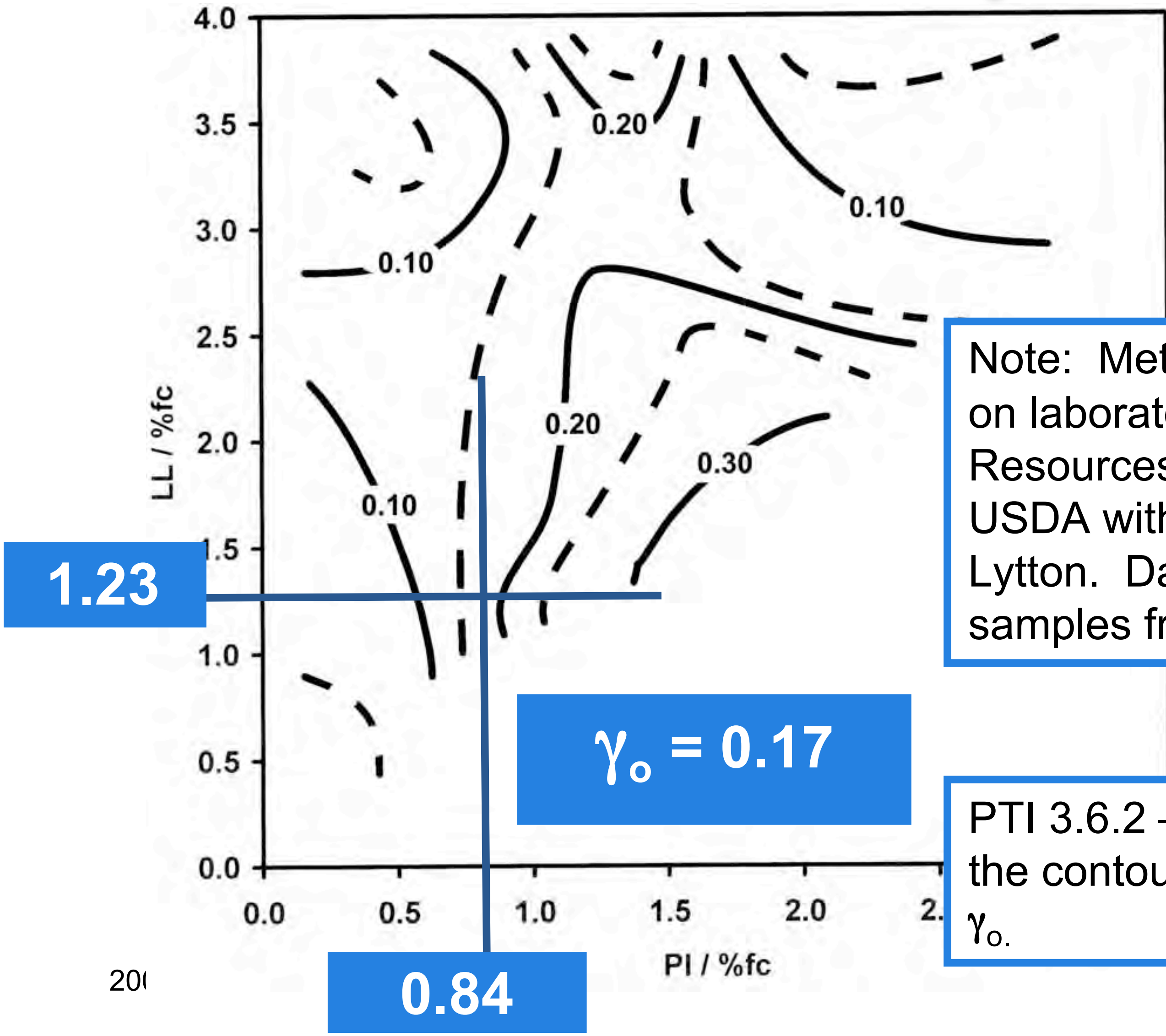
- Step 5 – Calculate LL / %fc

  LL / %fc

  LL / %fc = 90 / 73 = 1.23

e_m Step 6

Determine Suction Compression Index (γ_o)



Zone III

Note: Method for determining γ_o based on laboratory data from the National Resources Conservation Service, USDA with analysis by Covar and Lytton. Data included over 7000 samples from across the United States.

PTI 3.6.2 – Beyond extreme values of the contours, use the nearest values for γ_o.

e_m Step 7
Calculate Suction Compression Index (γ_h)

- γ_o is the suction compression index for a soil with 100 % fine clay (all particles smaller than 2 micron)

- γ_h is the suction compression index adjusted for the actual percentage of fine clay

$$\gamma_h = \gamma_o \,(\%fc)/100$$

$$\gamma_h = 0.17(73)/100 = 0.124$$

e_m Step 7

Calculate Suction Compression Index (γ_h)

Coarse-Grained Soil Correction

$$F = \frac{100}{1 + \left(\frac{J}{100 - J}\right)\left(\frac{(\gamma_t - wet)}{\gamma_w (G_s)_{coarse}}\right)}$$

$$(\gamma_h)_{corr} = \gamma_h \left[\frac{100}{F\left(\frac{\gamma_t - wet}{\gamma_d - dry}\right) + (100 - F)}\right]$$

Note: Should not be applied "unless a significant amount is retained by #10 sieve." Error exists in Eq 3-11. Correct equation shown above.

e_m Step 8

Modify γ_h for Shrinking and Swelling

- γ_o and γ_h determined with zone charts represent mean values.
- γ_h needs to be corrected for shrinking and swelling.

$$\gamma_{h\ shrinking} = \gamma_h e^{-\gamma_h}$$

$$\gamma_{h\ shrinking} = 0.124e^{-0.124} = 0.110$$

$$\gamma_{h\ swelling} = \gamma_h e^{\gamma_h}$$

$$\gamma_{h\ swelling} = 0.124e^{0.124} = 0.140$$

Note: Correction is different than in Technical Note #12

Alternate Procedures for Determining $\gamma_{h\ Swelling}$

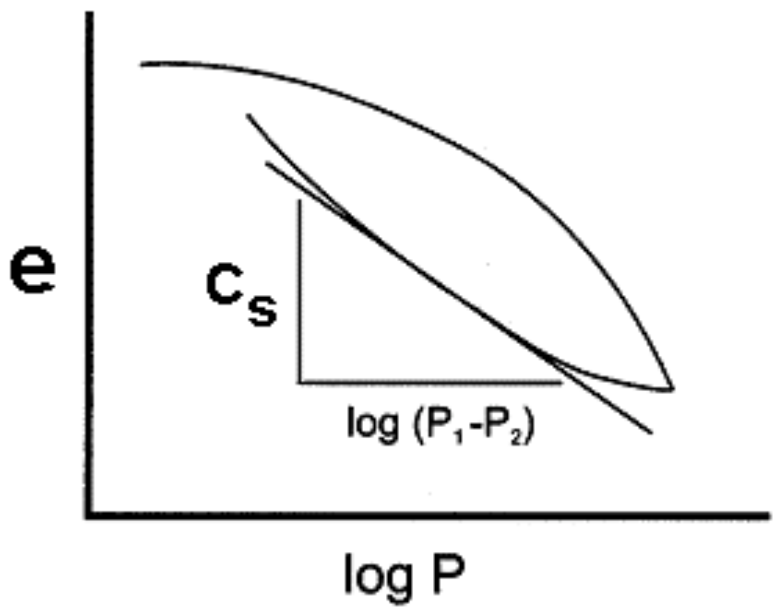
- Expansion Index Procedure: use ASTM D 4829 to determine EI:

$$\gamma_{h\ swell} = EI/1700$$

$$EI = \frac{1000x\left(final\ thickness - initial\ thickness\right)}{initial\ thickness}$$

Alternate Procedures for Determining $\gamma_{h\ \mathrm{Swelling}}$

- Consolidation - Swell Pressure Test Procedure: use ASTM D-4546 Method C



$$\gamma_{h\ \mathrm{swell}} = \frac{(0.7)(C_s)}{(1+e_2)}$$

Alternate Procedures for Determining $\gamma_{h\ \text{Swelling}}$

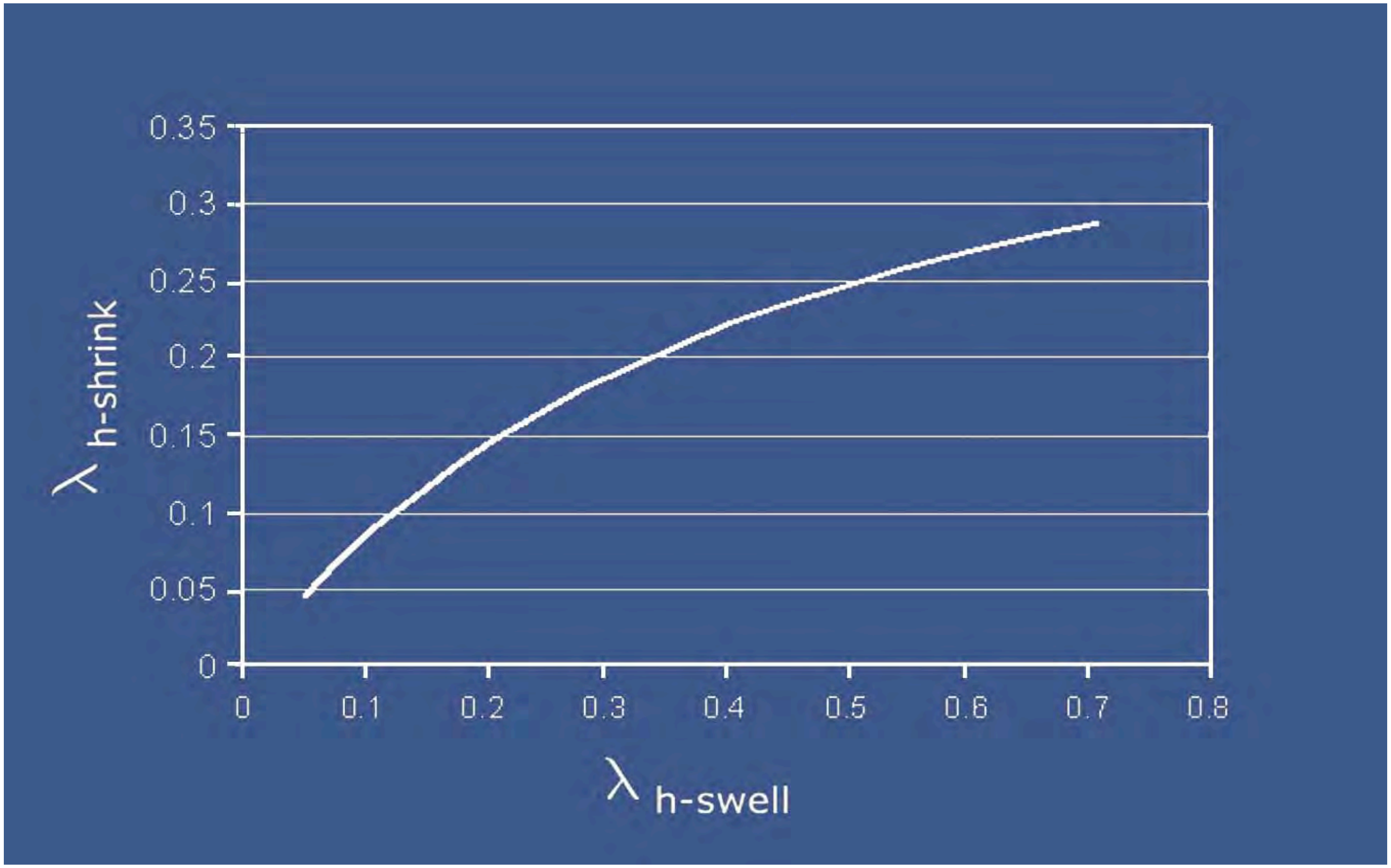
- Overburden Pressure Swell Test

$$\gamma_{\text{h swell}} = \frac{\frac{\Delta H}{H}}{1.7 - Log_{10}P}$$

$\frac{\Delta H}{H}$ is decimal change of specimen height divided by the initial height

P Overburden Pressure

Alternate Procedures for Determining $\gamma_{h\ Swelling}$



e_m Step 9
Calculate S_s

- S_s is the slope of the suction vs. gravimetric water content curve.

- Can be determined from soil-water characteristic curve or be estimated with the following equation.

S_s =-20.29 + 0.1555 LL – 0.117 PI + 0.0684 (%-#200)

$S_s = -20.29 + 0.1555(90) - 0.117(61) + 0.0684(89)$

$S_s = -7.3$

e_m Step 10

Calculate Unsaturated Diffusion Coefficient

- The Unsaturated Diffusion Coefficient (a) for shrinking and swelling can be estimated with the following equations (based on field observations)

$$\alpha_{shrinking} = 0.0029 - 0.000162(S_s) - 0.0122(\gamma_{h\ shrinking})$$

$$\alpha_{shrinking} = 0.0029 - 0.000162(-7.3) - 0.0122(0.110)$$

$$\alpha_{shrinking} = 0.00274$$

$$\alpha_{swelling} = 0.0029 - 0.000162(S_s) - 0.0122(\gamma_{h\ swelling})$$

$$\alpha_{swelling} = 0.0029 - 0.000162(-7.3) - 0.0122(0.140)$$

$$\alpha_{swelling} = 0.00237$$

e_m Step 11

Calculate Modified Unsaturated Diffusion Coefficient

$$\alpha' = \alpha\,(F_f)$$

Table 3.1 - Soil Fabric Factor

Condition	F_f
Soil profiles contain few roots, layers, fractures or joints (No more than 1 per vertical foot)	1.0
Soil profiles contain some roots, layers, fractures or joints (2 to 4 per vertical foot)	1.3
Soil profiles contain many roots, layers, fractures or joints (5 or more per vertical foot)	1.4

Reason for Fabric Factor

Reason for Fabric Factor

e_m Step 11

Calculate Modified Unsaturated Diffusion Coefficient

$$\alpha'_{shrinking} = \alpha_{shrinking}\,(F_f)$$

$$\alpha'_{shrinking} = 0.00274\,(1.0) = 0.00274$$

$$\alpha'_{swelling} = \alpha_{swelling}\,(F_f)$$

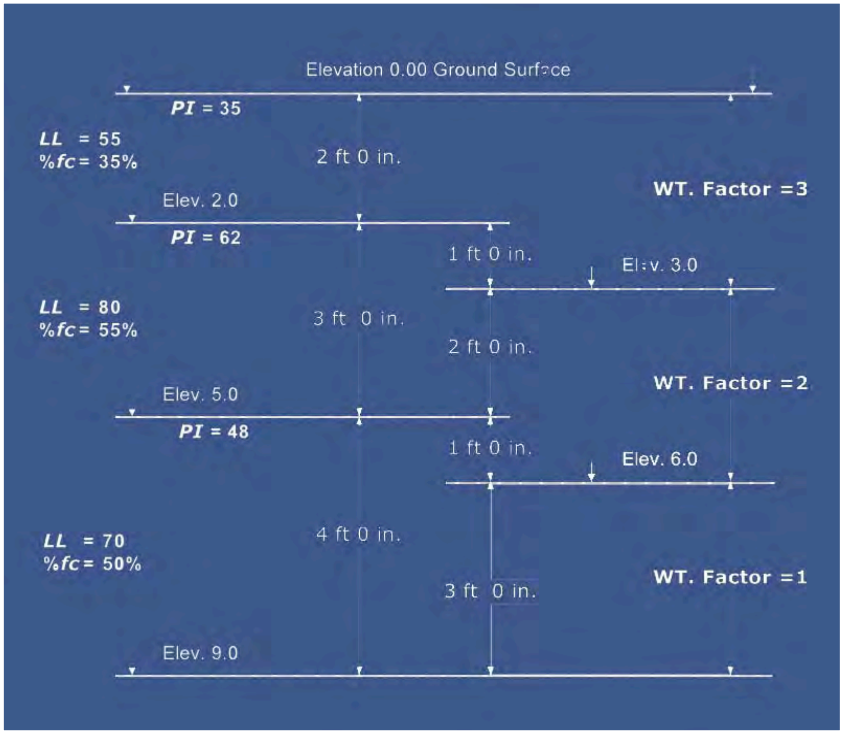
$$\alpha'_{swelling} = 0.00237\,(1.0) = 0.00237$$

e_m Step 12

Calculate Weighted Modified Unsaturated Diffusion Coefficient

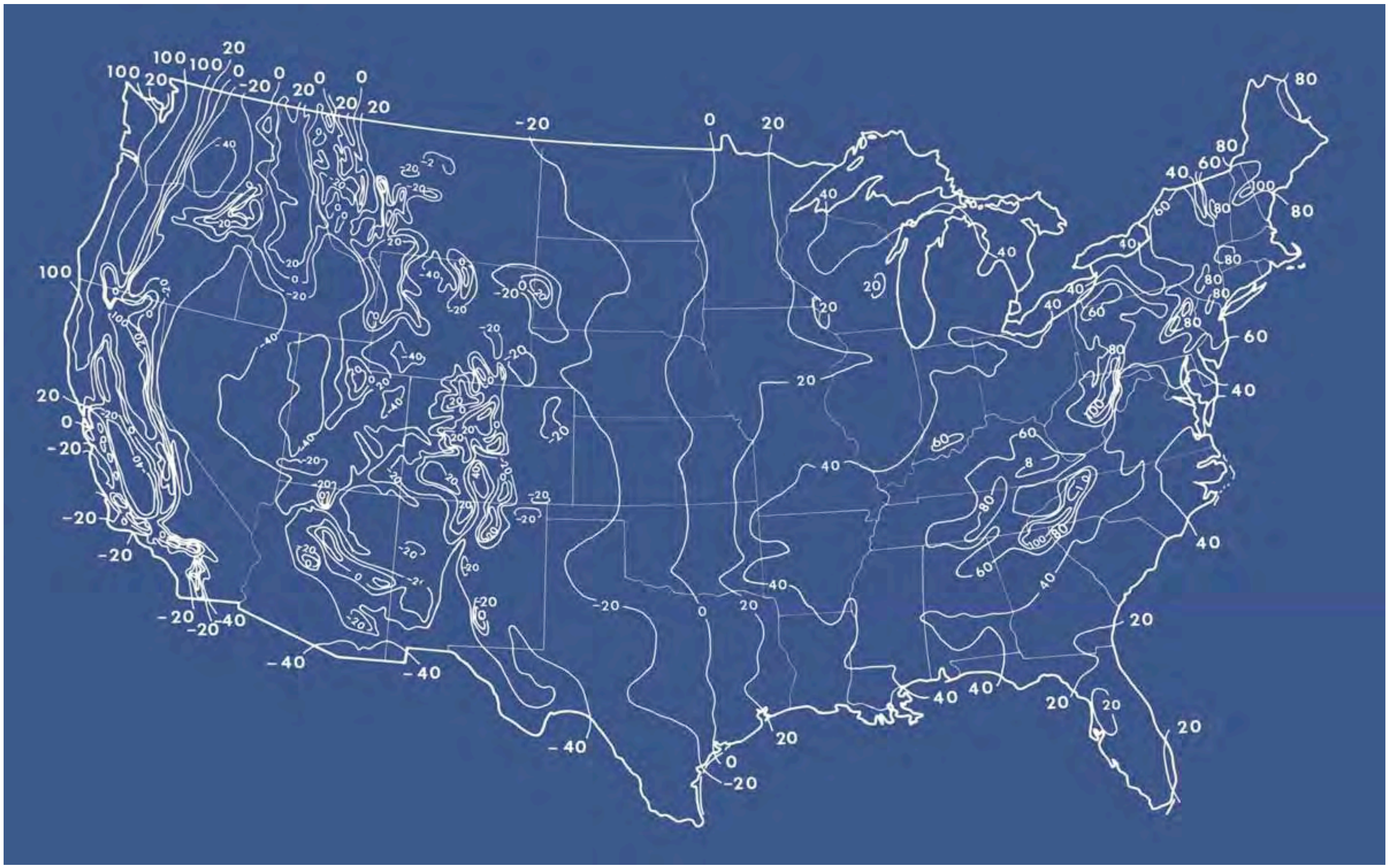
For layered soil profiles $(\alpha')_{weighted}$ to be calculated per the following equation:

$$(\alpha')_{weighted} = (\Sigma F_i \times D_i \times \alpha_i) / (\Sigma F_i \times D_i)$$



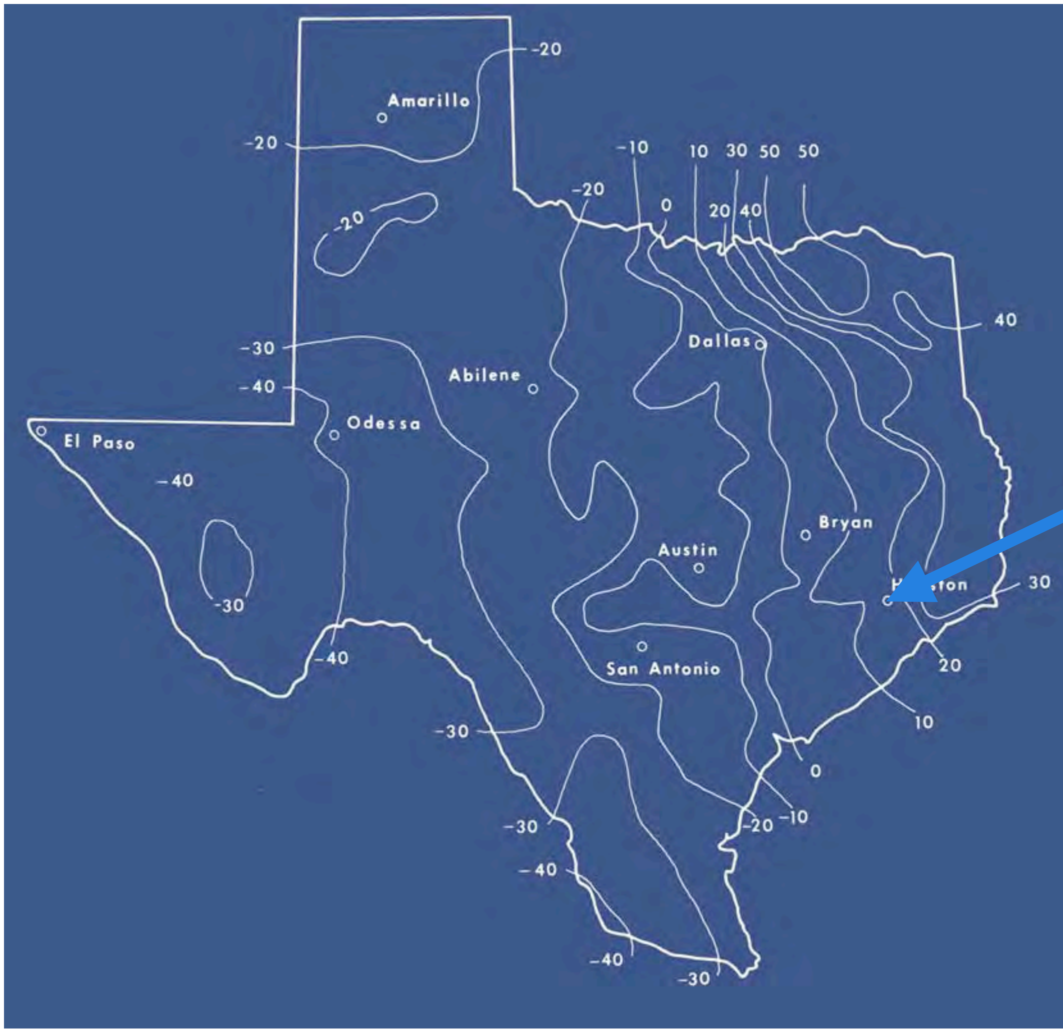
e_m Step 13

Determine Thornthwaite Moisture Index (I_m)



e_m Step 13

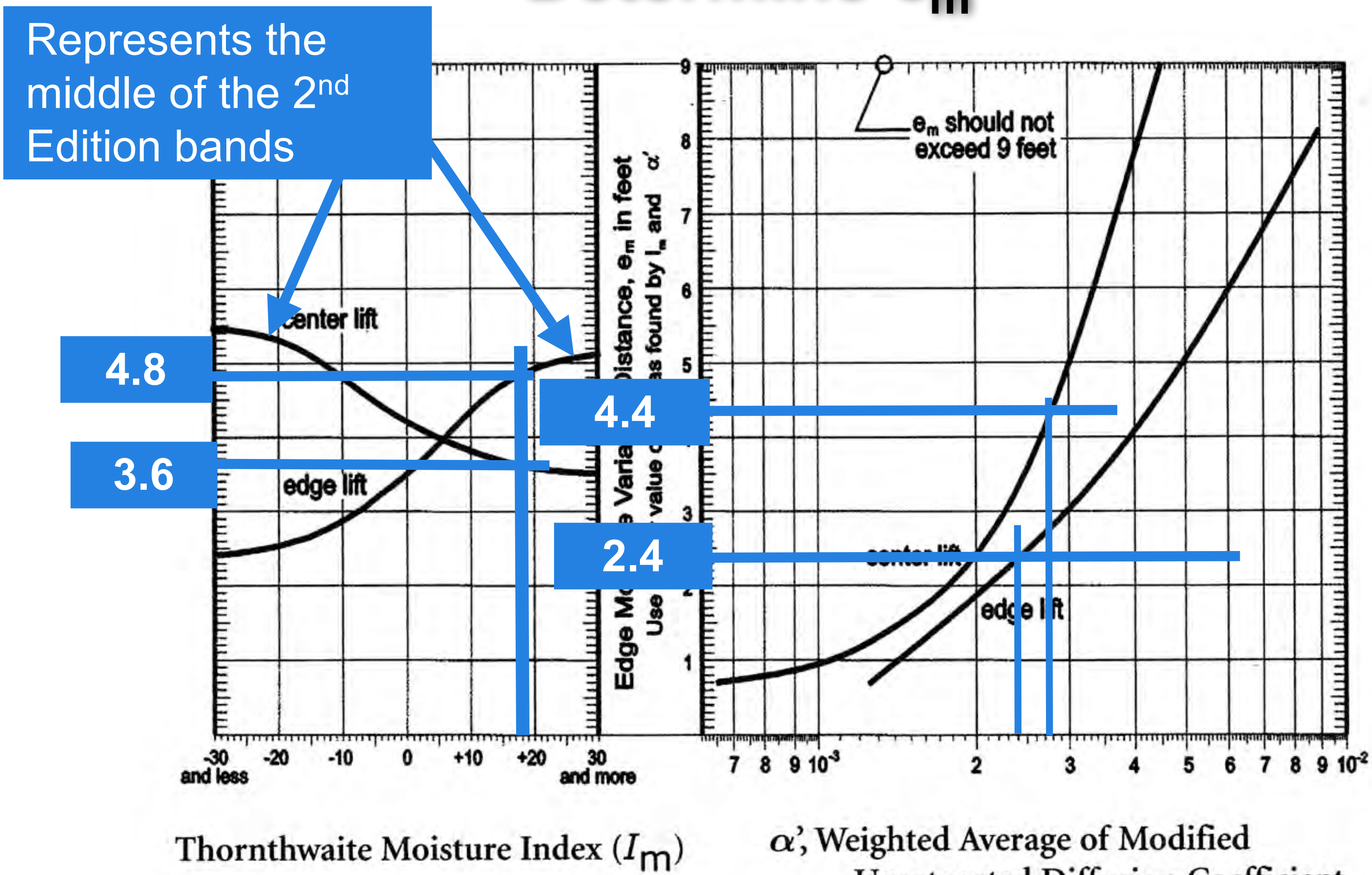
Determine Thornthwaite Moisture Index (I_m)



$I_m = 18$

e_m Step 14

Determine e_m



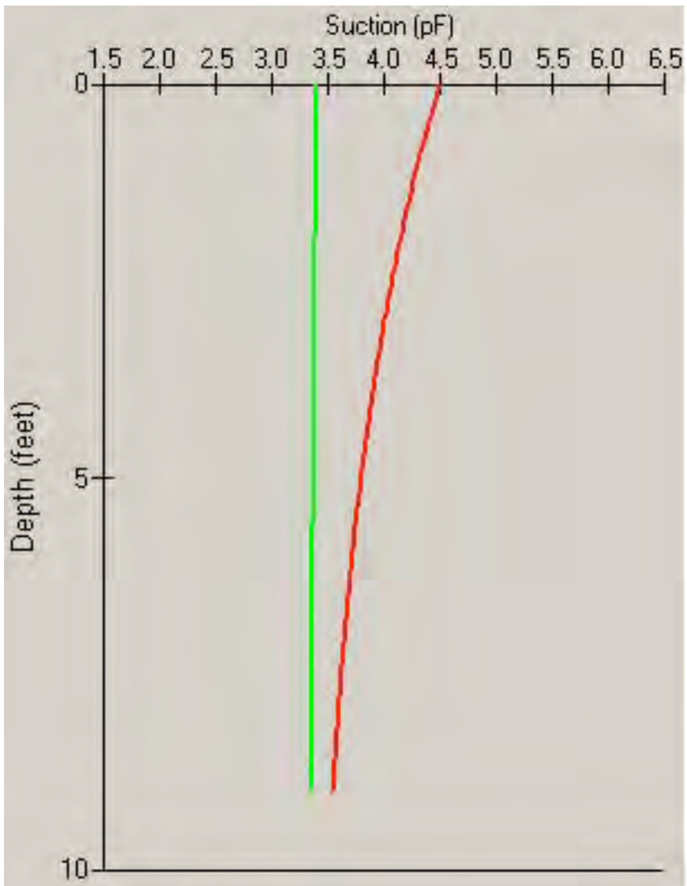
Example of y_m Calculation

Steps to determine y_m

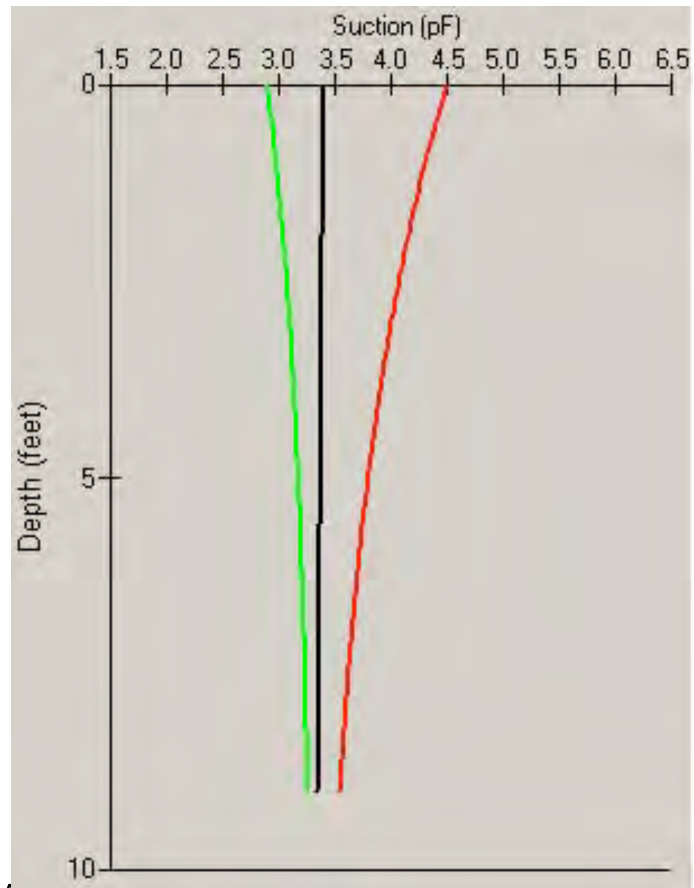
- Step 1 – Determine if Post-Equilibrium or Post-Construction Suction Profiles control
- Step 2 - Determine Equilibrium Suction
- Step 3 – Determine Dry Suction Envelope
- Step 4 – Determine Wet Suction Envelope
- Step 5 – Determine Stress Change Factors
- Step 6 – Calculate Weighted Suction Compression Index
- Step 7 – Calculate y_m

y_m Step 1
Post-Equilibrium or Post-Construction

SHRINKING



Post–Equilibrium

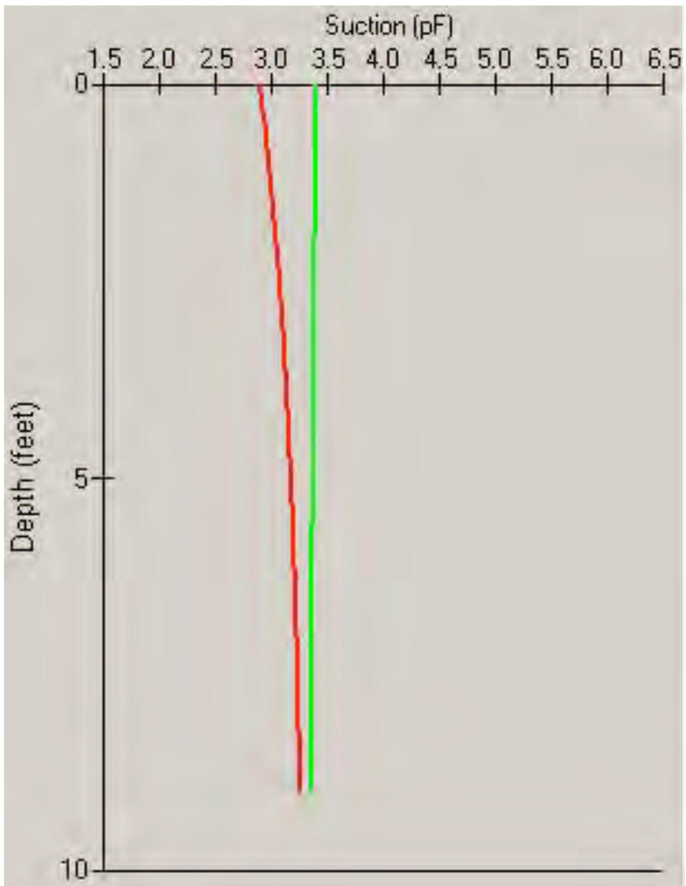


Post–Construction

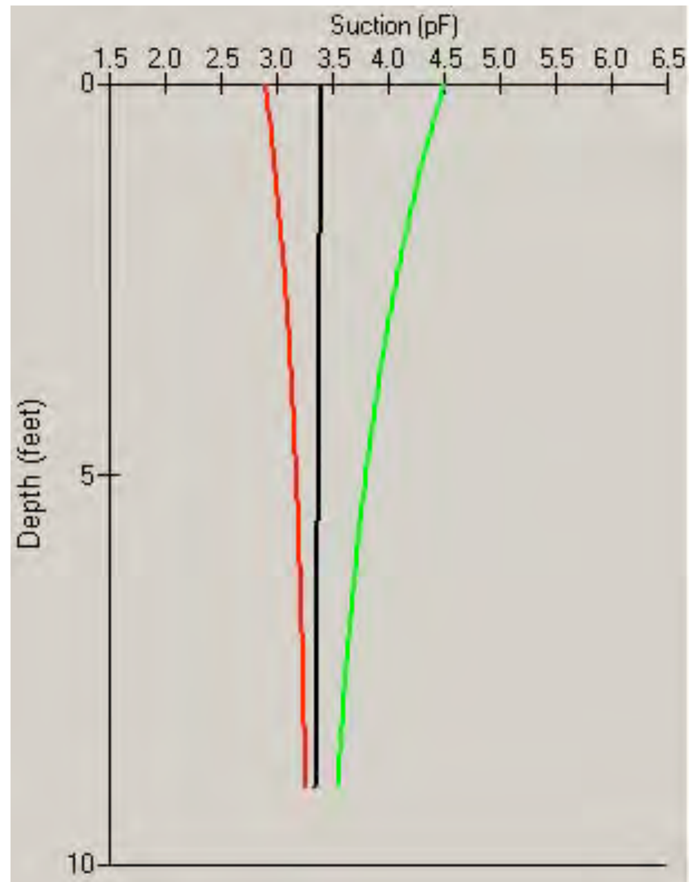
y_m Step 1

Post-Equilibrium or Post-Construction

SWELLING



Post–Equilibrium



Post–Construction

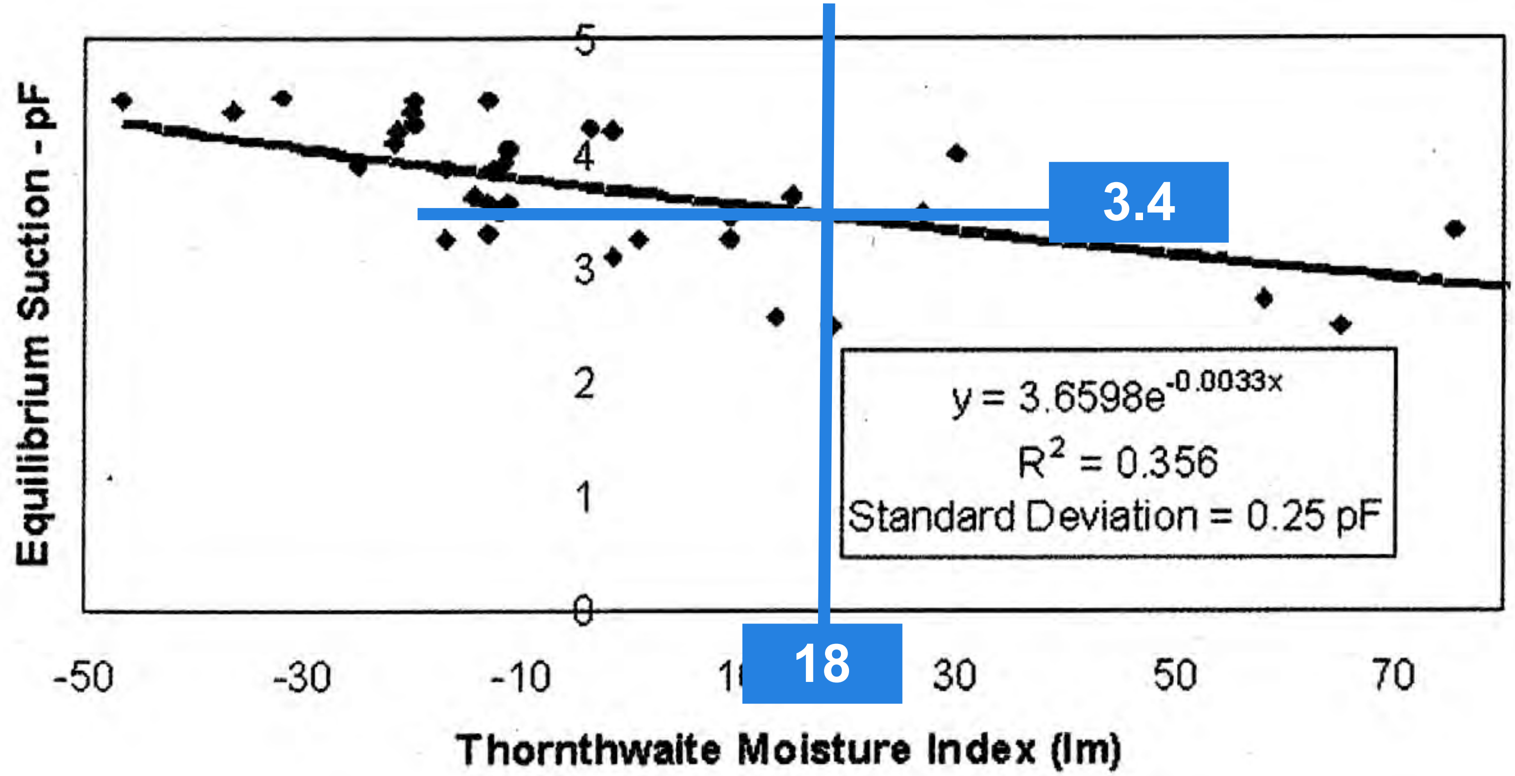
y_m Step 1
Post-Equilibrium or Post-Construction

- For Post-Equilibrium suction envelopes use Stress Change Factor (SCF) procedure included in 3rd Edition manual or computer method such as VOLFLO.

- For Post-Construction suction envelopes use computer method such as VOLFLO.

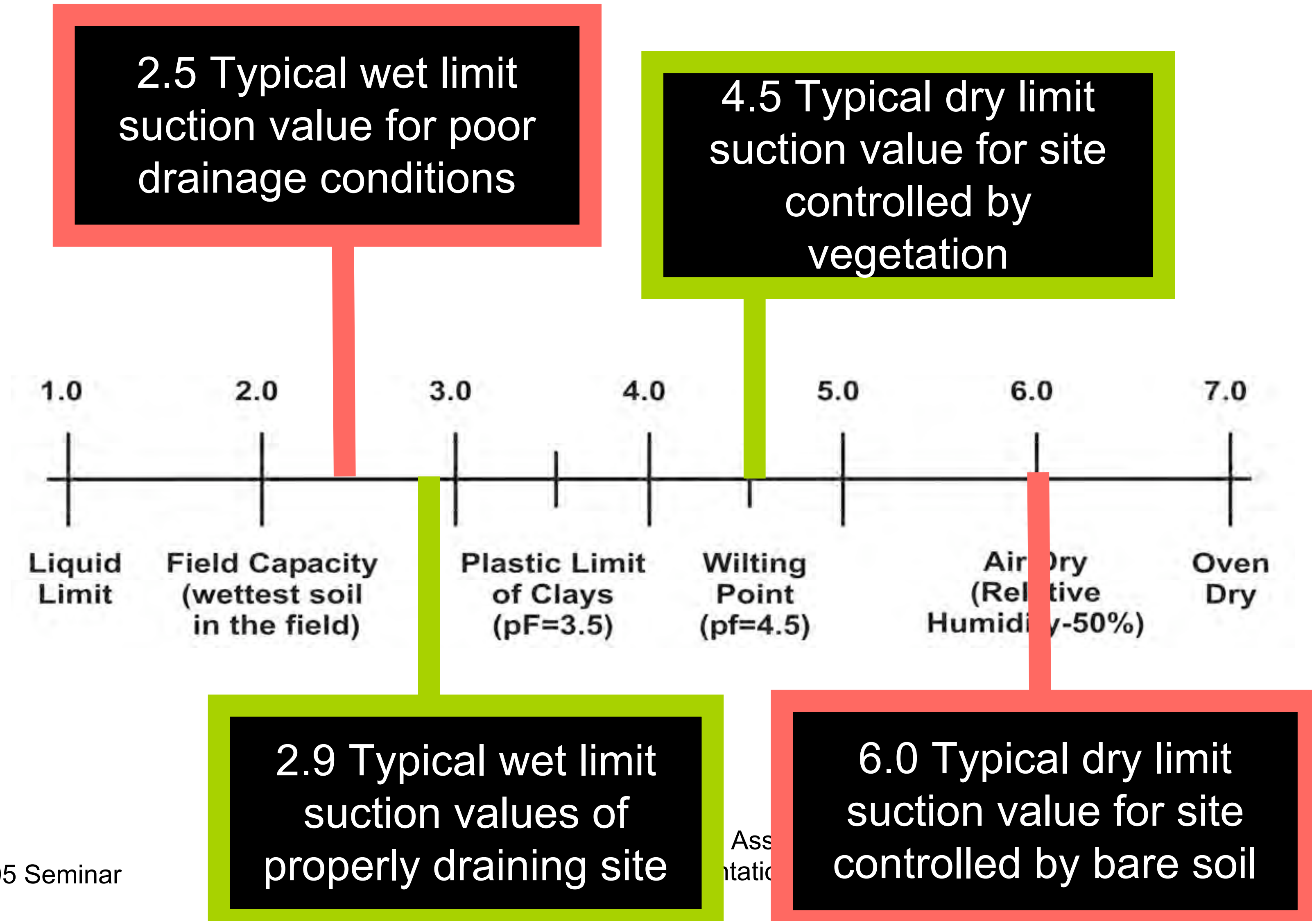
y_m Step 2
Determine Equilibrium Suction

Equilibrium Suction may be estimated from correlation below in the absence of local observations:



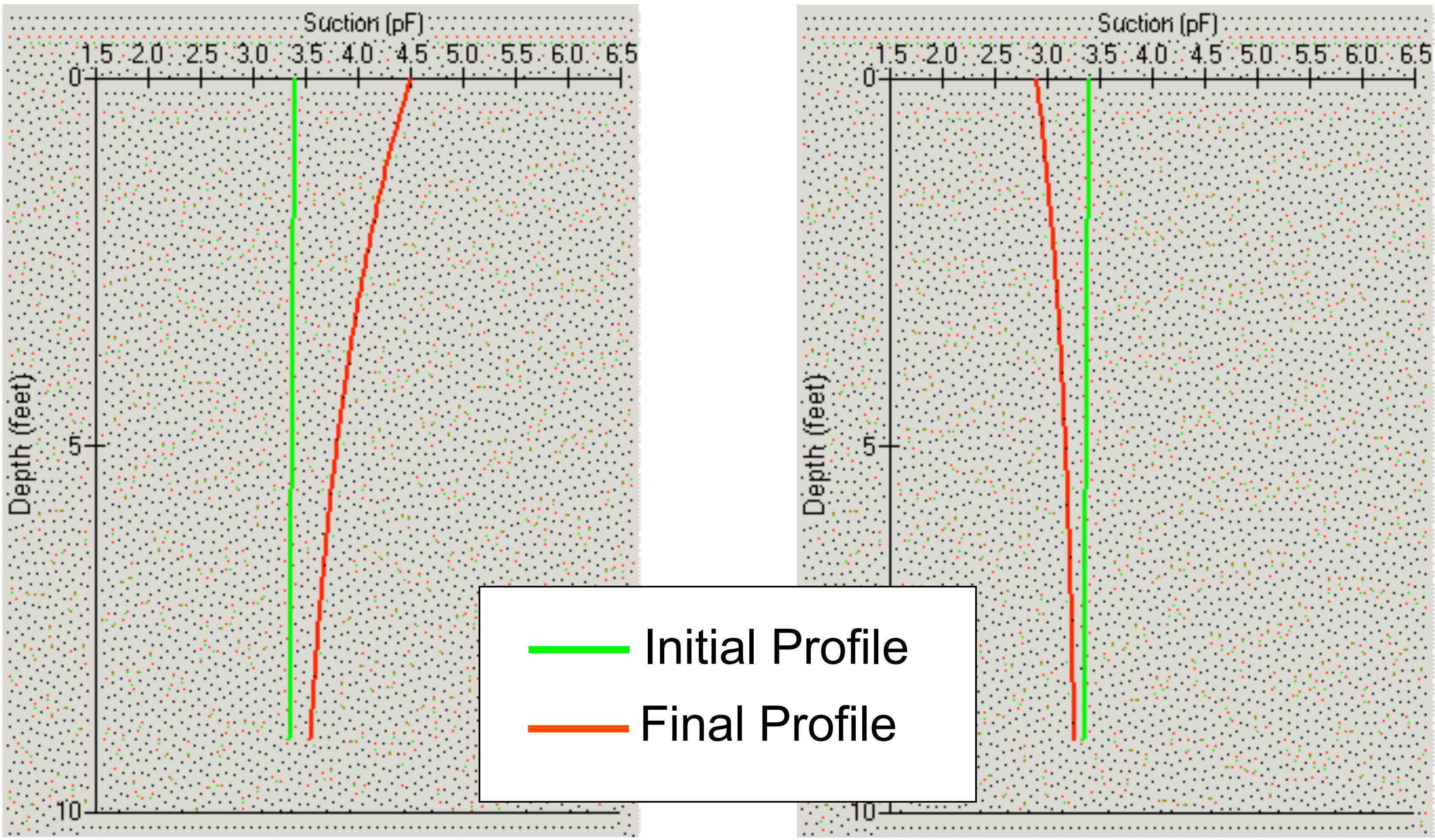
Note: Also referred to as constant suction or measured suction at depth. This figure has changed from 2nd Edition.

y_m Steps 3 and 4
Determine Dry and Wet Suction Envelopes



y_m Steps 3 and 4

Determine Dry and Wet Suction Envelopes



Shrinking **Swelling**

y_m Step 5
Determine Stress Change Factors

Table 3.2 Stress Change Factor (*SCF*) for Use in determining y_m

Measured Suction (*pF*) at Depth z_m	Final Controlling Suction At Surface, *pF*						
	2.5	2.7	3.0	3.5	4.0	4.2	4.5
2.7	+3.2	0	-4.1	-13.6	-25.7	-31.3	-40.0
3.0	+9.6	+5.1	0	-7.5	-18.2	-23.1	-31.3
3.3	+17.7	+12.1	+5.1	-2.6	-11.5	-15.8	-23.1
3.6	+27.1	+20.7	+12.1	+1.6	-5.7	-9.4	-15.8
3.9	+38.1	+30.8	+20.7	+7.3	-1.3	-4.1	-9.4
4.2	+50.4	+42.1	+30.8	+14.8	+3.2	0	-4.1
4.5	+63.6	+54.7	+42.1	+23.9	+9.6	+5.1	0

$SCF_{shrinking}$= -20.7

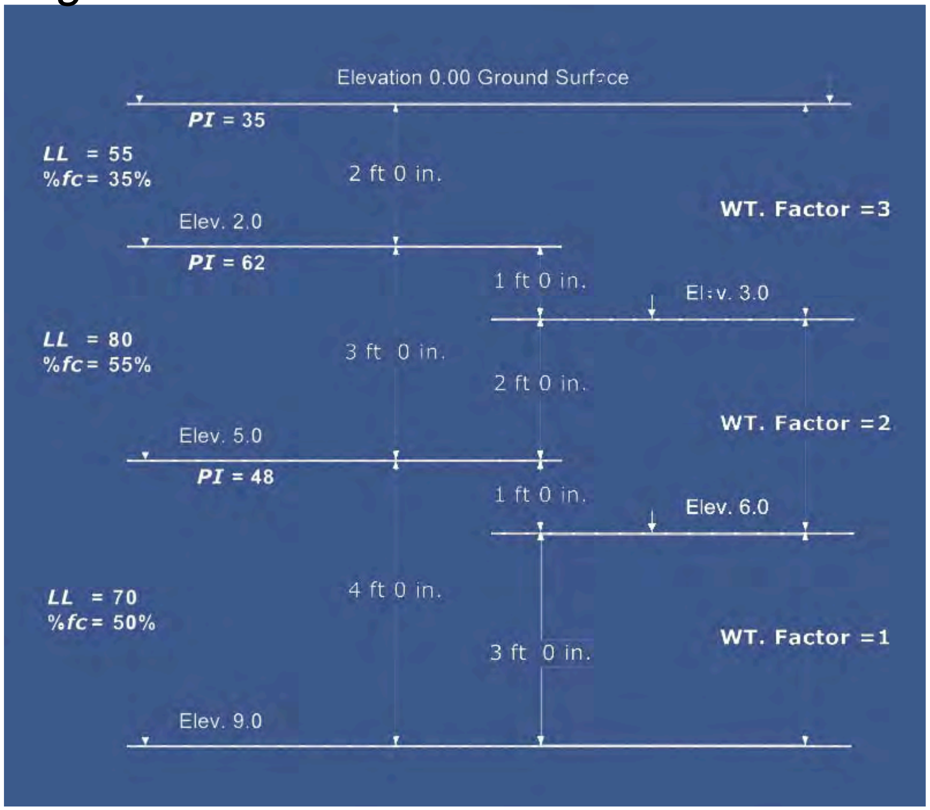
$SCF_{swelling}$= 9.9

y_m Step 6

Calculate Weighted Suction Compression Index

For layered soil profiles - $(\gamma_h)_{weighted}$ to be calculated per the following equation:

$$(\gamma_h)_{weighted} = (\Sigma F_i \times D_i \times \gamma_{hi}) / (\Sigma F_i \times D_i)$$



y_m Step 7
Calculate y_m

- $y_{m\ Center} = \gamma_{h\ shrinking} \times SCF_{shrinking}$
- **$y_{m\ Center} = 0.110 \times -20.7$**
- **$y_{m\ Center} = 2.28$ inches**

- $y_{m\ Edge} = \gamma_{h\ swelling} \times SCF_{swelling}$
- **$y_{m\ Edge} = 0.140 \times 9.9$**
- **$y_{m\ Edge} = 1.39$ inches**

Stress Change Factor Method

- Assumes initial suction is at equilibrium

- Assumes final suction is a typical "trumpet shape" suction envelope

- Assumes Depth to Constant Suction is 9ft

- Unconservative for extreme soil suction conditions (Post-Construction).

- Possibly Over- or Under- conservative for soil profiles with multiple layers where γ_h varies significantly (See PTI 3.6.3)

Summary of Soil Support Parameters (SCF)

Houston, TX

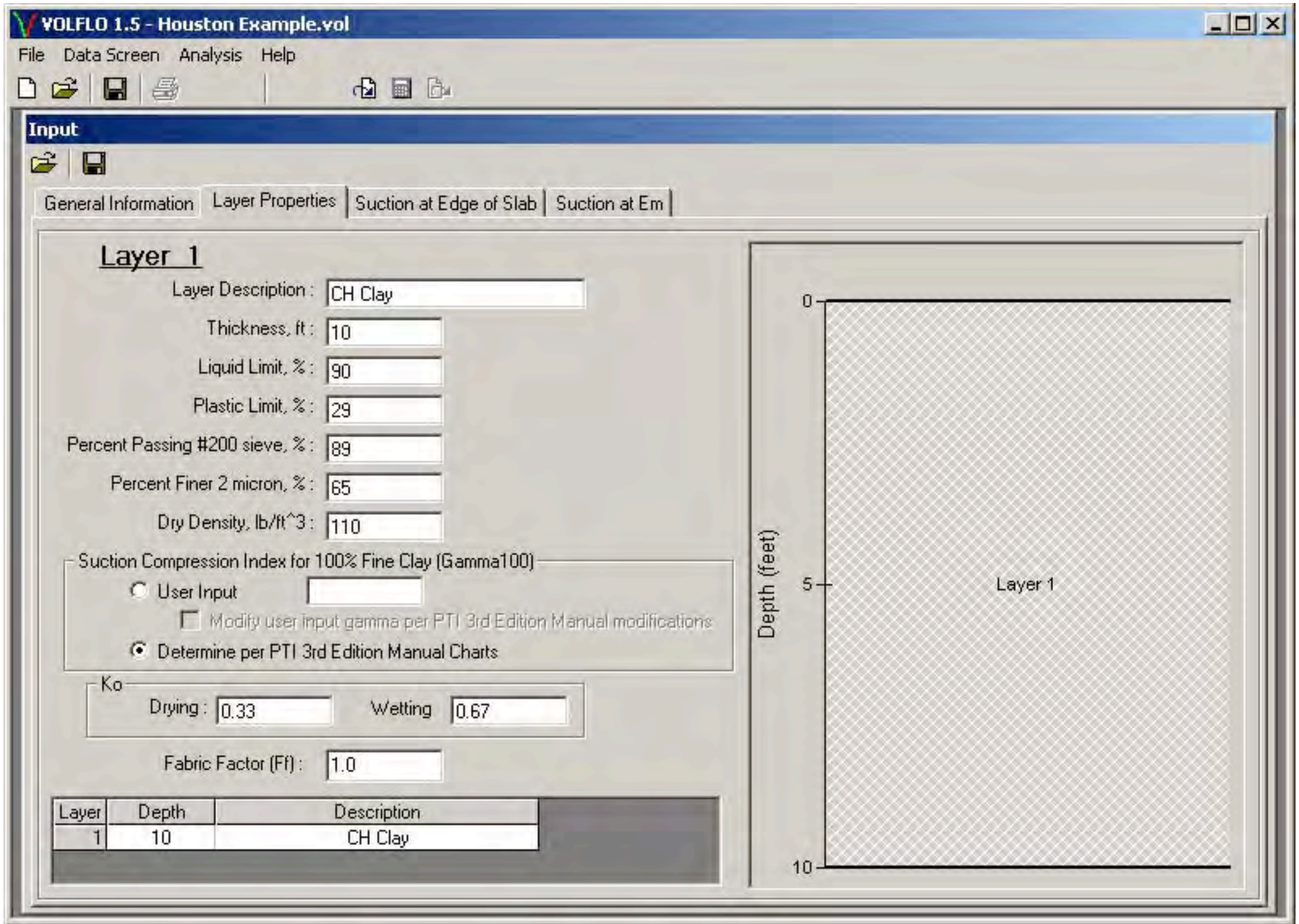
$e_{m\ Center}$ = 4.4 feet

$e_{m\ Edge}$ = 4.8 feet

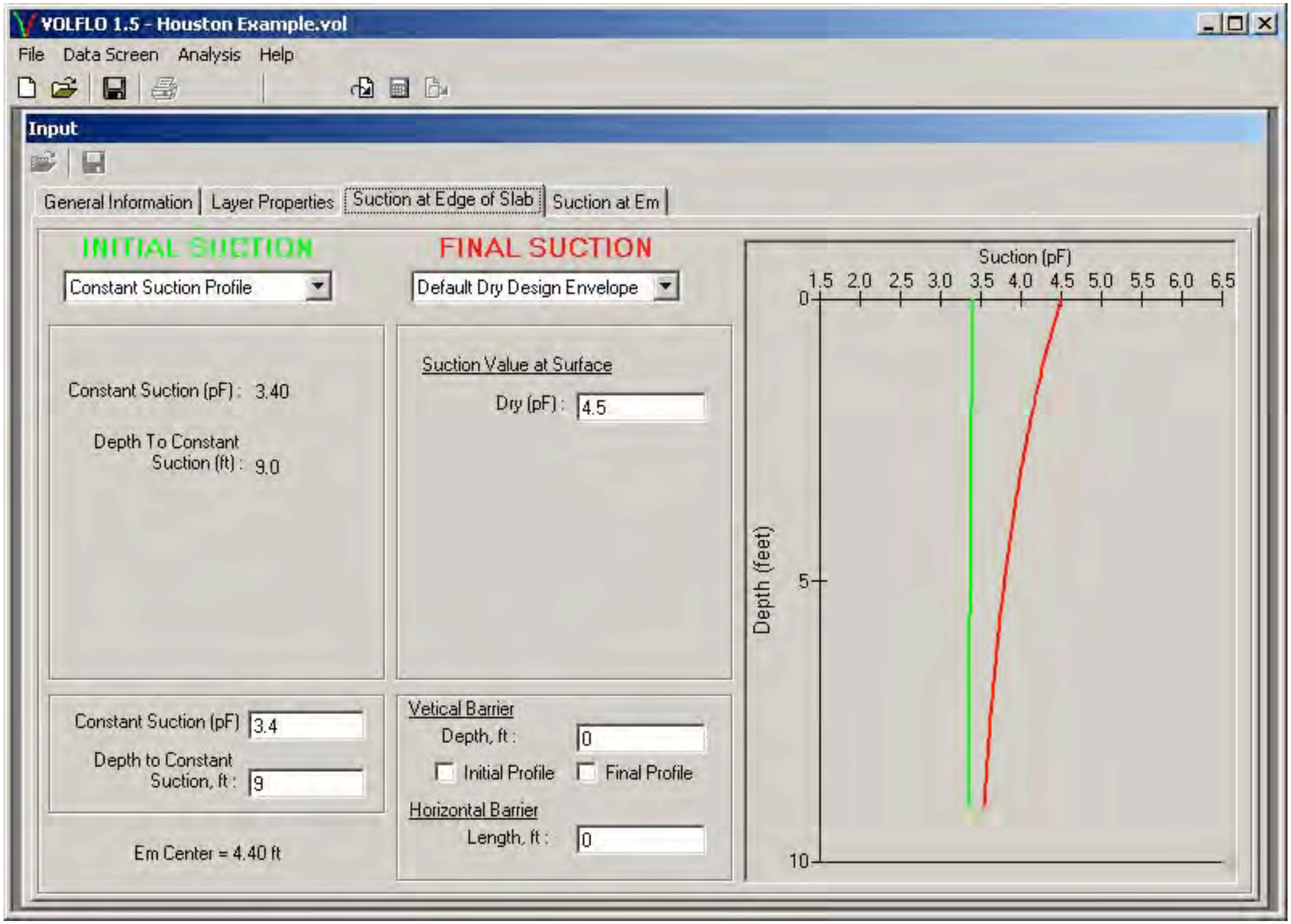
$y_{m\ Center}$ = 2.28 inches

$y_{m\ Edge}$ = 1.39 inches

VOLFLO 1.5 - Shrinking



VOLFLO 1.5 - Shrinking



VOLFLO 1.5 - Shrinking

VOLFLO 1.5 - Houston Example.vol

File Data Screen Analysis Help

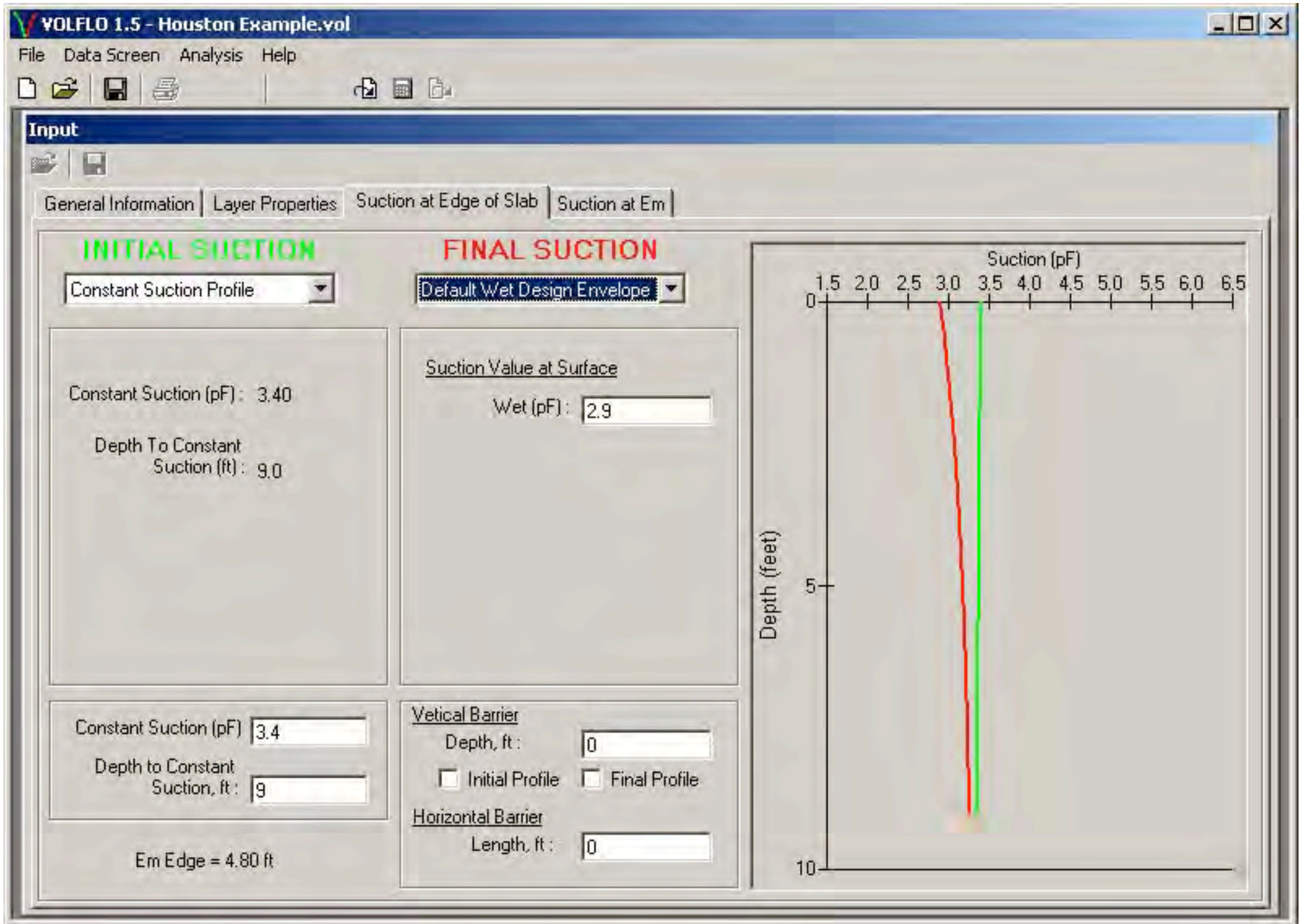
Output

Shrink / Swell | Soil Information | Suction Profiles | Initial Suction Profiles | Final Suction Profiles

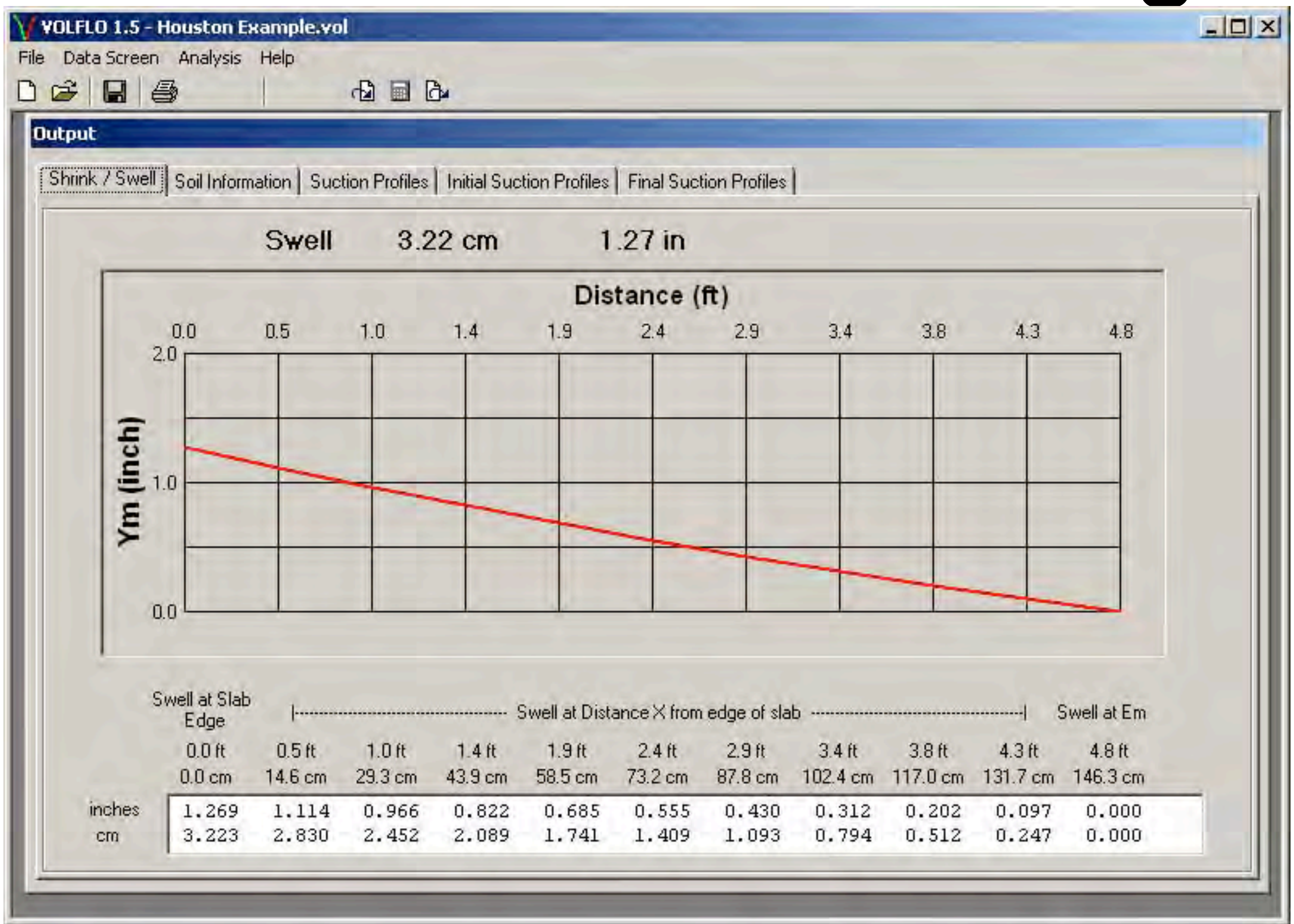
Shrink -5.10 cm -2.01 in

	Shrink at Slab Edge	Shrink at Distance X from edge of slab									Shrink at Em
	0.0 ft	0.4 ft	0.9 ft	1.3 ft	1.8 ft	2.2 ft	2.6 ft	3.1 ft	3.5 ft	4.0 ft	4.4 ft
	0.0 cm	13.4 cm	26.8 cm	40.2 cm	53.6 cm	67.1 cm	80.5 cm	93.9 cm	107.3 cm	120.7 cm	134.1 cm
inches	-2.008	-1.764	-1.527	-1.298	-1.077	-0.866	-0.666	-0.478	-0.303	-0.143	0.000
cm	-5.101	-4.481	-3.879	-3.297	-2.736	-2.200	-1.692	-1.213	-0.770	-0.364	0.000

VOLFLO 1.5 - Swelling



VOLFLO 1.5 - Swelling



Comparison of Soil Support Parameters

Houston, TX

Homogeneous Soil Profile

Post-Equilibrium Suction Envelopes

	e_m		y_m	
	Center	Edge	Center	Edge
SCF	4.4	4.8	2.28	1.39
VOLFLO 1.5	4.4	4.8	2.01	1.27
% Difference	0%	0%	13%	9%

What is the effect of the geotechnical assumptions?

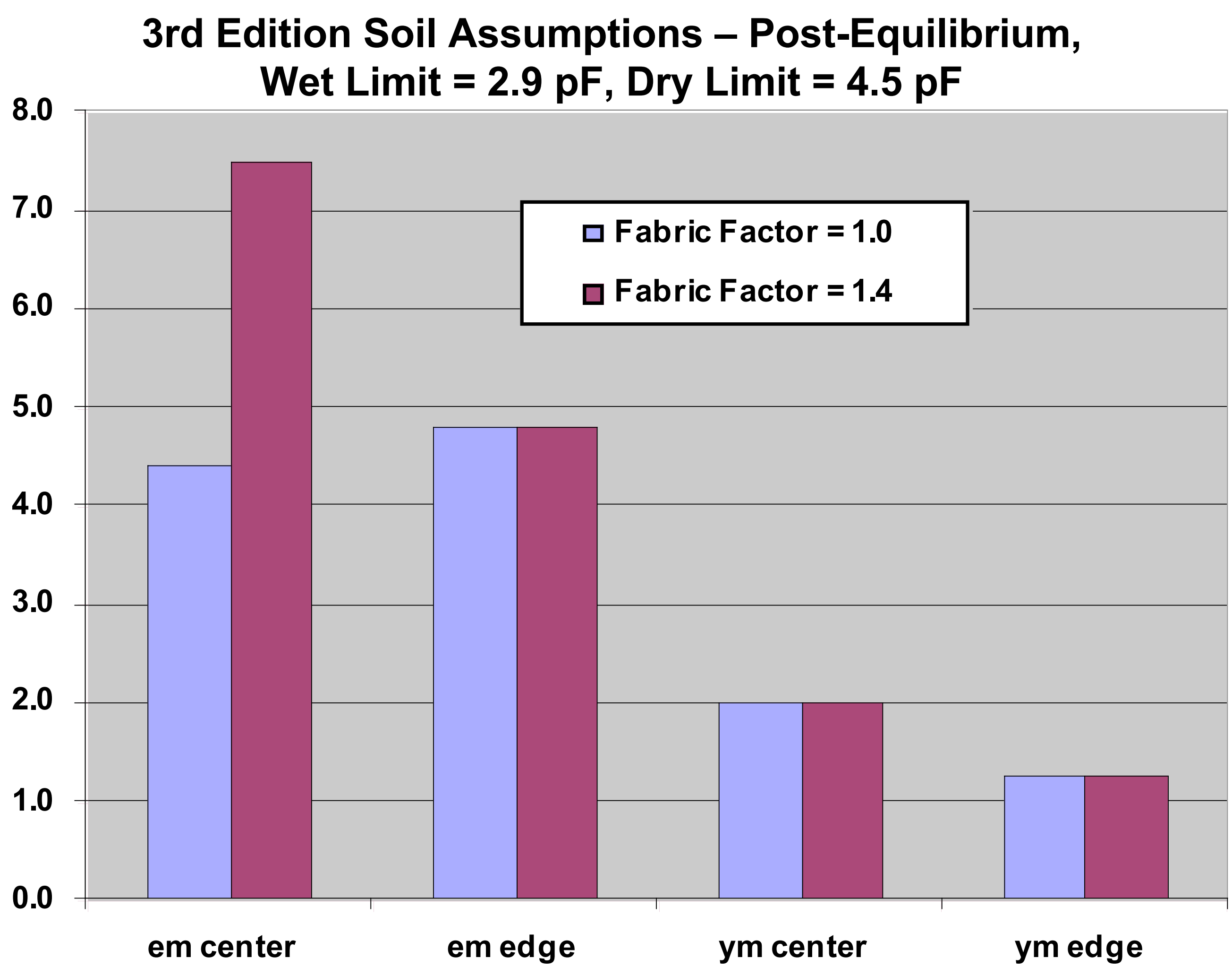
Comparison of Effect of Fabric Factor

Table 3.1 - Soil Fabric Factor

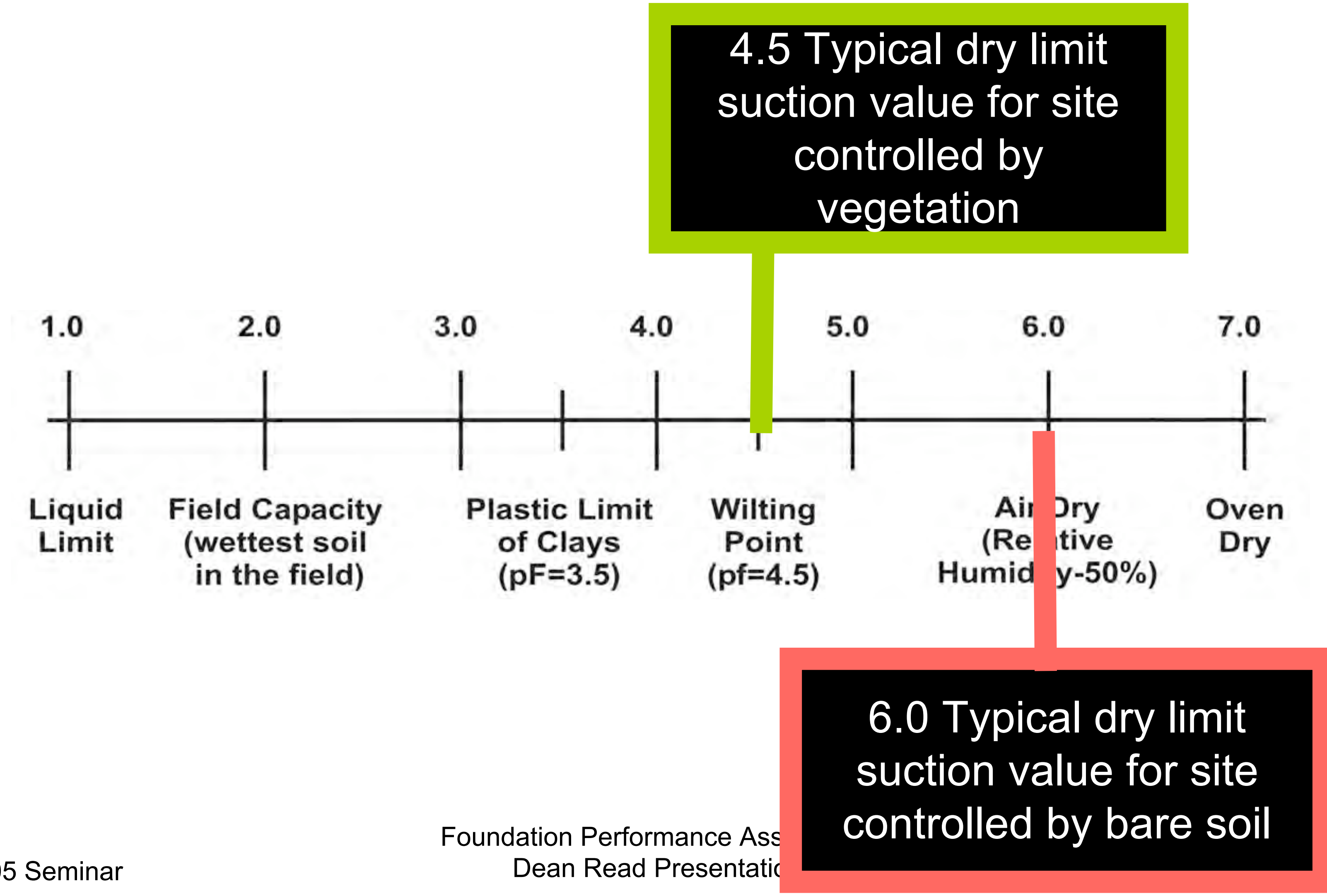
Condition	F_f
Soil profiles contain few roots, layers, fractures or joints (No more than 1 per vertical foot)	1.0
Soil profiles contain **some** roots, layers, fractures or joints (2 to 4 per vertical foot)	1.3
Soil profiles contain **many** roots, layers, fractures or joints (5 or more per vertical foot)	1.4

Comparison of Effect of Fabric Factor

Houston, TX soil profile

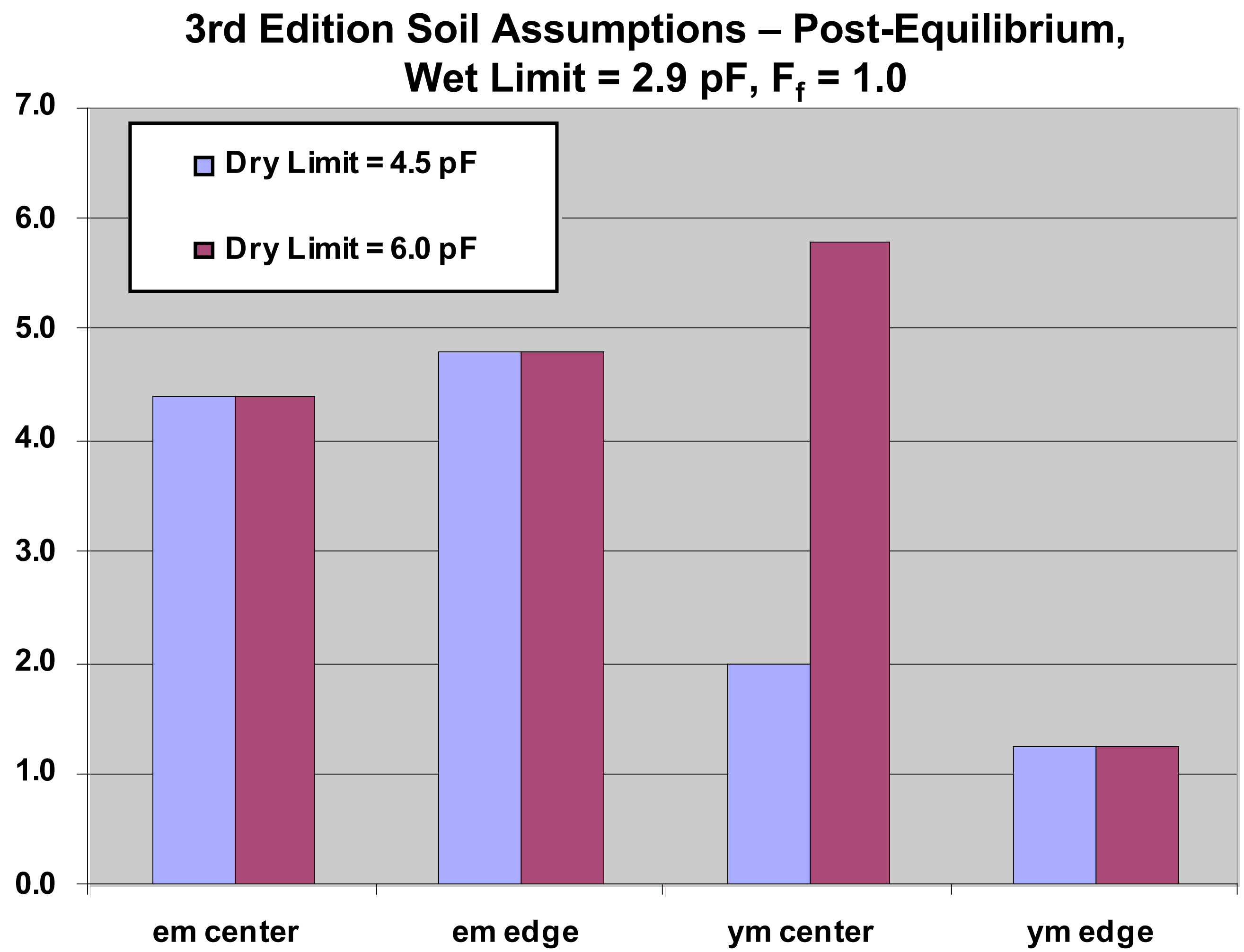


Comparison of Effect of Dry Suction Limit

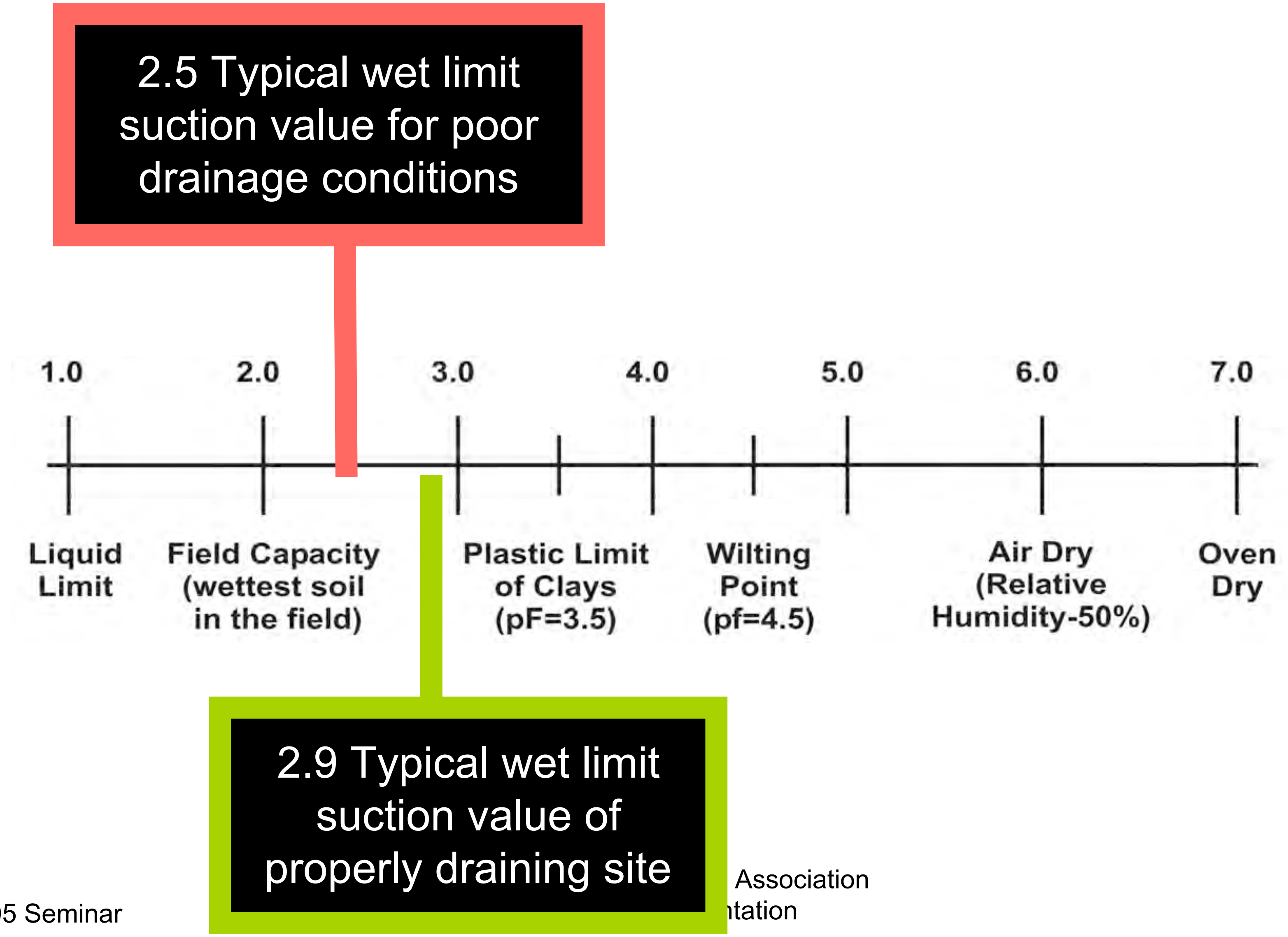


Comparison of Effect of Dry Suction Limit

Houston, TX soil profile

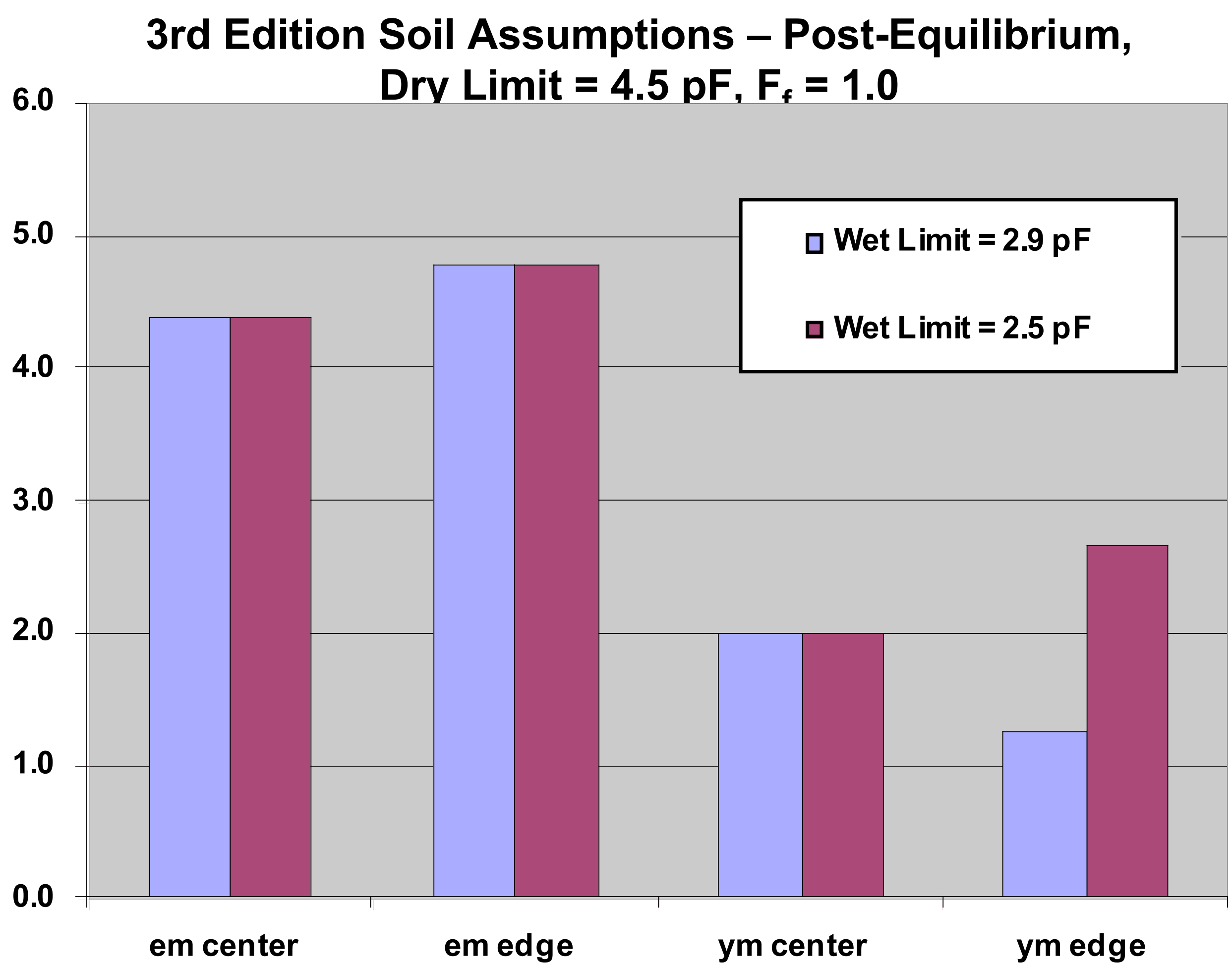


Comparison of Effect of Wet Suction Limit



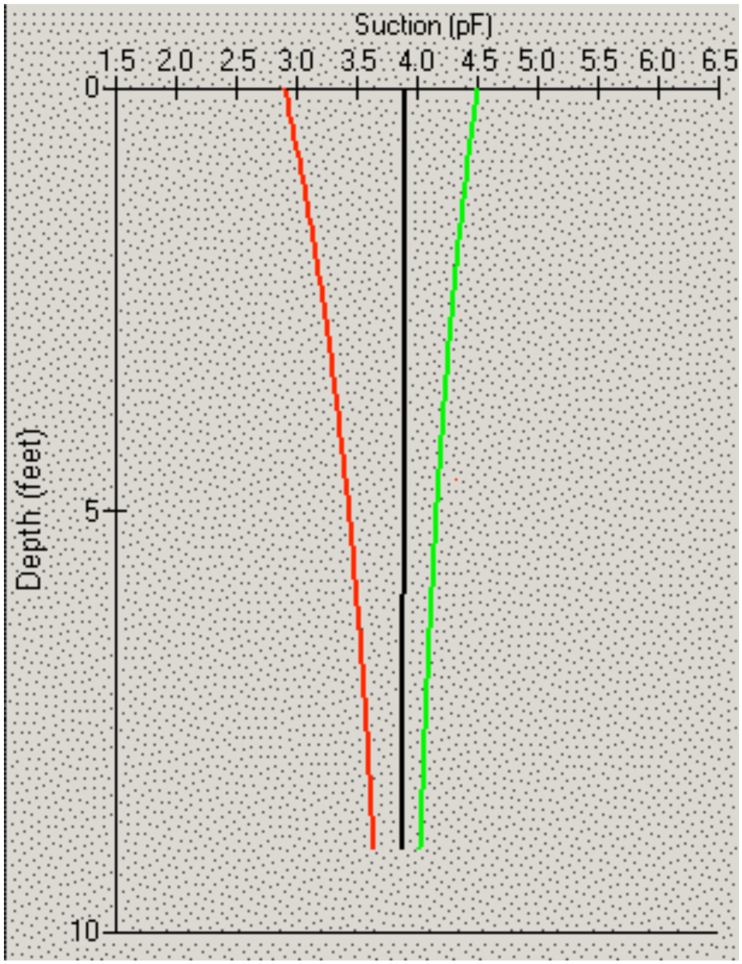
Comparison of Effect of Wet Suction Limit

Houston, TX soil profile



Comparison of Effect of Post-Equilibrium vs. Post-Construction

Post–Equilibrium

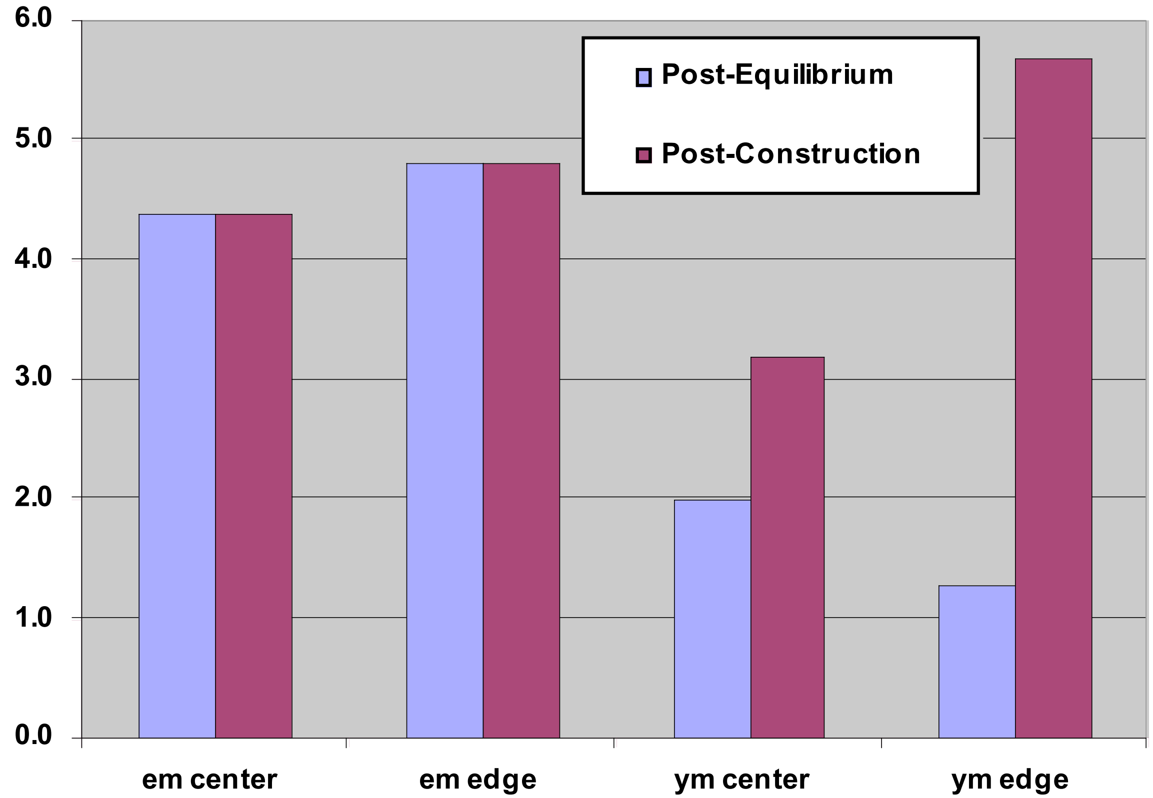


Post–Construction

Comparison of Effect of Post-Equilibrium vs. Post-Construction

Houston, TX Soil Profile

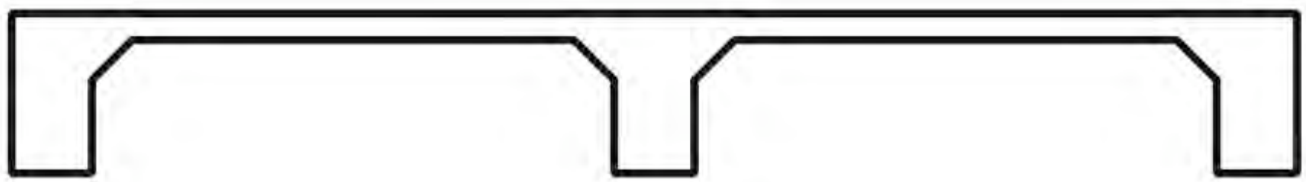
3rd Edition Soil Assumptions –Wet Limit = 2.9 pF, Dry Limit = 4.5 pF, F_f = 1.0



STRUCTURAL PROCEDURE

Foundation Types

- Ribbed Foundation



- Uniform Thickness Foundation

Foundation Design

- Design is performed based on “trial and error” procedure. Assumptions are made and then assumed design checked for “compliance”. If assumed design is “out of compliance” or over-designed modify assumptions and perform analysis again.

Ribbed Foundation

- Ribbed foundation checked for compliance with the following:
  - Flexural Stresses
    - Tension
    - Compression
  - Shear Stress
  - **Minimum Stiffness**
  - **Cracked Section Capacity**
  - Soil Bearing Capacity

Uniform Thickness Foundation

- “Constructible” and “Compliant” Ribbed Foundation must be designed before being converted to UTF using the following equation:

$$H = \sqrt[3]{\frac{I}{W}}$$

Uniform Thickness Foundation

- "Compliant" Ribbed foundation for UTF design must comply with
  - Flexural Stresses
    - Tension
    - Compression
  - Shear Stress
  - **Minimum Stiffness**
  - ~~**Cracked Section Capacity**~~
  - ~~Soil Bearing Capacity~~

Compliance not required

Uniform Thickness Foundation

- After conversion, UTF to be checked for "compliance" with:
  - Flexural Stresses
    - Tension
    - Compression
  - Shear Stress
  - **Cracked Section Capacity**

Required Variables for Structural Procedure

Concrete Properties

- Compressive Strength **- 3,000 psi**
- Creep Modulus **- 1,500,000 psi**

Creep modulus is typically assumed to be 50% the concrete modulus of elasticity. Can assume 1,500,000 for all strengths concrete.

- Unit Weight **- 145 pcf**

Tendon Properties

- Maximum Tensile Strength **- 270 ksi**
- Tendon Diameter **- 1/2”**

1/2” diameter strand tendons are virtually exclusively used.

Area of 1/2” strand tendon is 0.153 in^2

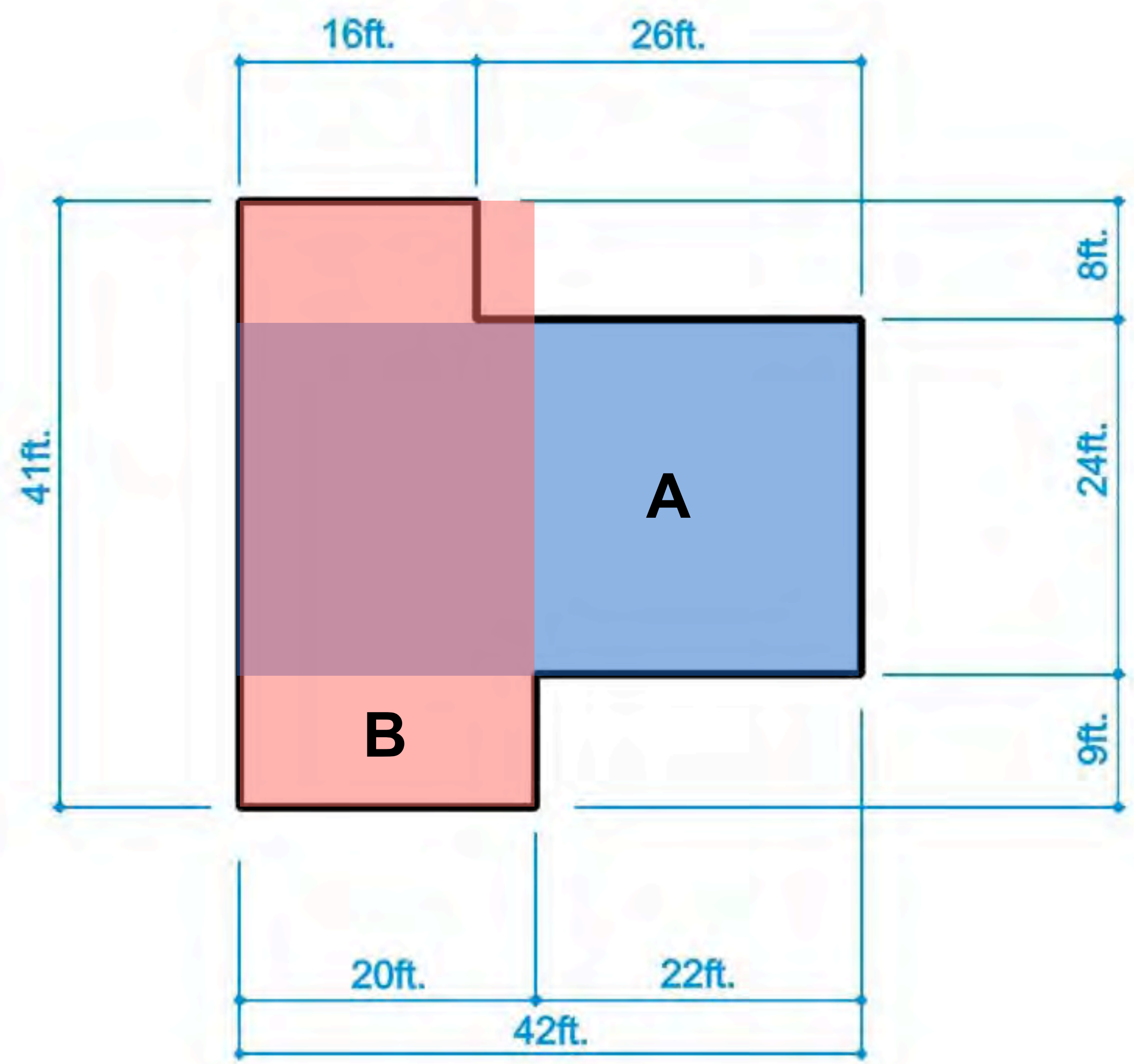
(not 0.2 in^2)

Design Rectangles

PTI 6.3 – “Slabs of irregular shape should be divided into overlapping rectangles so that the resulting boundary provides reasonable congruence with the foundation perimeter.”

PTI 6.3 –“Long narrow rectangles may not appropriately model the overall foundation and generally should not govern the design.”

Design Rectangles



Design Rectangle Geometry

Example for Design Rectangle A

- Design Rectangle Width **- 24 ft**
- Design Rectangle Length **- 42 ft**

- Slab Thickness **- 4 inches**

Prestress

- Minimum Effective Prestress Force **- 0.10A for ribbed foundation**

  PTI 6.7 – “The minimum prestress Pe required is 0.05A for ribbed foundations and 0.6HW for uniform thickness foundations”

  Note: Effective Prestress Force of 0.05A is equivalent to Effective Prestress of 50 psi

  PTI 6.7 – “When excessive shrinkage cracking is anticipated the designer should consider increasing the minimum prestress force to 0.10A for ribbed foundations and 0.12HW for uniform thickness foundations”

Rib Geometry, Spacing and Reinforcement

- Rib Depth **- 12” (initial assumption)**

PTI 4.5.2.2 - “… the total rib depth *h* shall be in no case less than 11 in., and the rib must extend at least 7 in. below the bottom of the slab.”

PTI 4.5.2.2 - “… it is permissible to use ribs of different depths in the design, provided that the ratio between the deepest and shallowest rib does not exceed 1.2.” This applies to beams in the same direction.

Ribs depths in short and long direction do not need to be the same depth.

Rib Geometry, Spacing and Reinforcement

- Rib Width **- 8" (initial assumption)**

PTI 4.5.2.3 - "… the rib width used in section property calculations must be limited to a range of 8 to 14 in."

PTI 4.5.2.3 - "Rib widths most commonly found in practice are 10 to 12 in."

Rib Geometry, Spacing and Reinforcement

- Tendons per ribs **- 1 (initial assumption)**

  Tendons in the bottom of the ribs are generally required to resist edge lift soil movements and to comply with new cracked section provisions.

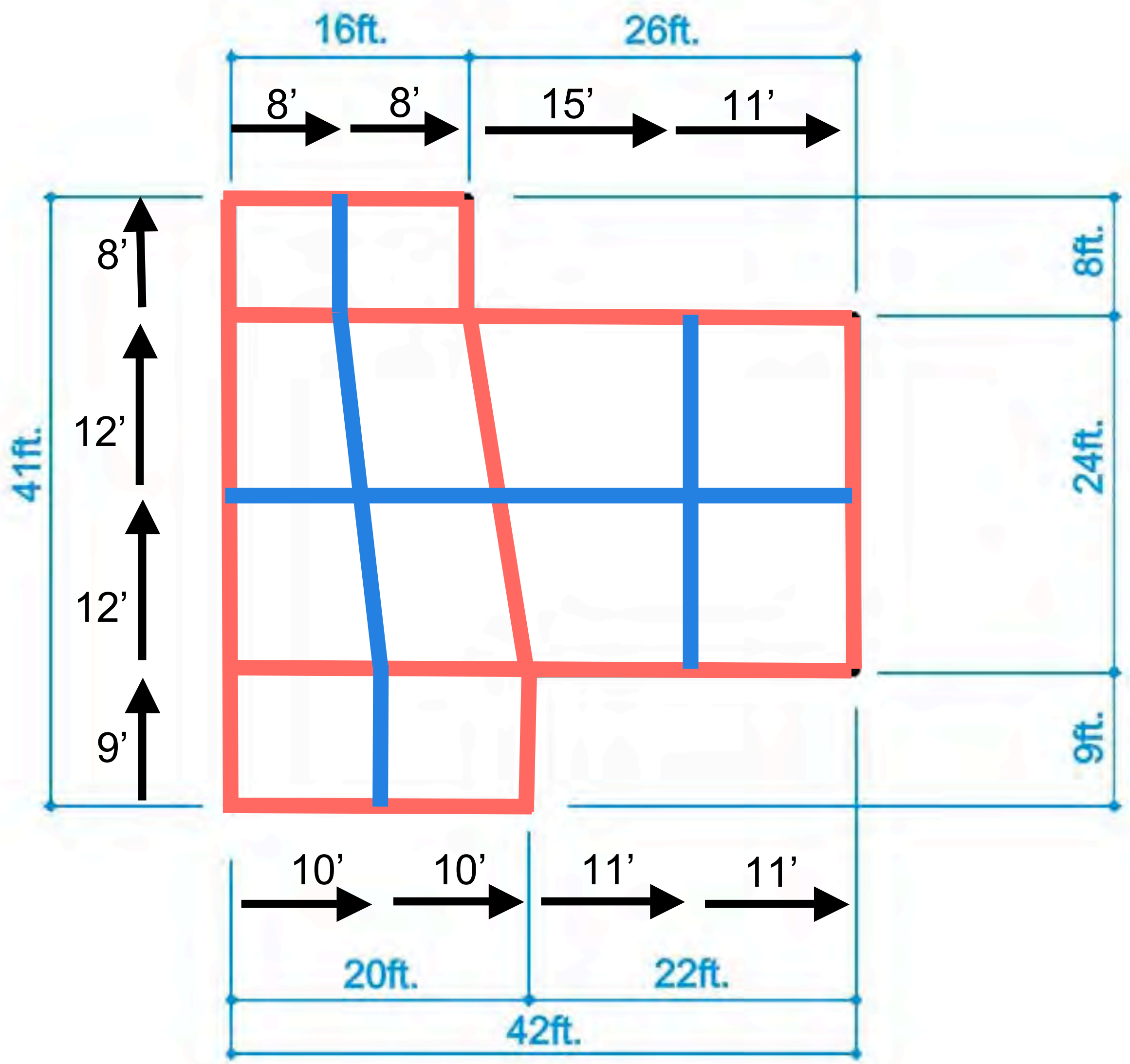
- Concrete Cover below tendons **- 2.75 in.**

Rib Locations

The minimum number of ribs in each direction is determined by the architectural layout and the maximum permitted beam spacing.

PTI 4.5.2.1 - “Rib spacing S shall be a maximum of 15 ft. A minimum rib spacing of 6 ft shall be used in the design of ribbed slabs.”

Rib Locations



Rib Geometry, Spacing and Reinforcement

- Number of Ribs **- 5 short, 3 long (initial)**
- Rib spacing **-**

  Short 15 ft max, 8 ft min (initial)

  Long 12 ft max, 12 ft min (initial)

PTI 4.5.2.1 – "When rib spacings vary, the average spacing may be used for design unless the ratio between the largest and smallest spacing exceeds 1.5. In that case, the design spacing shall be 0.85 times the largest spacing."

This provision results in a greater applied moment and shear.

Soil Properties

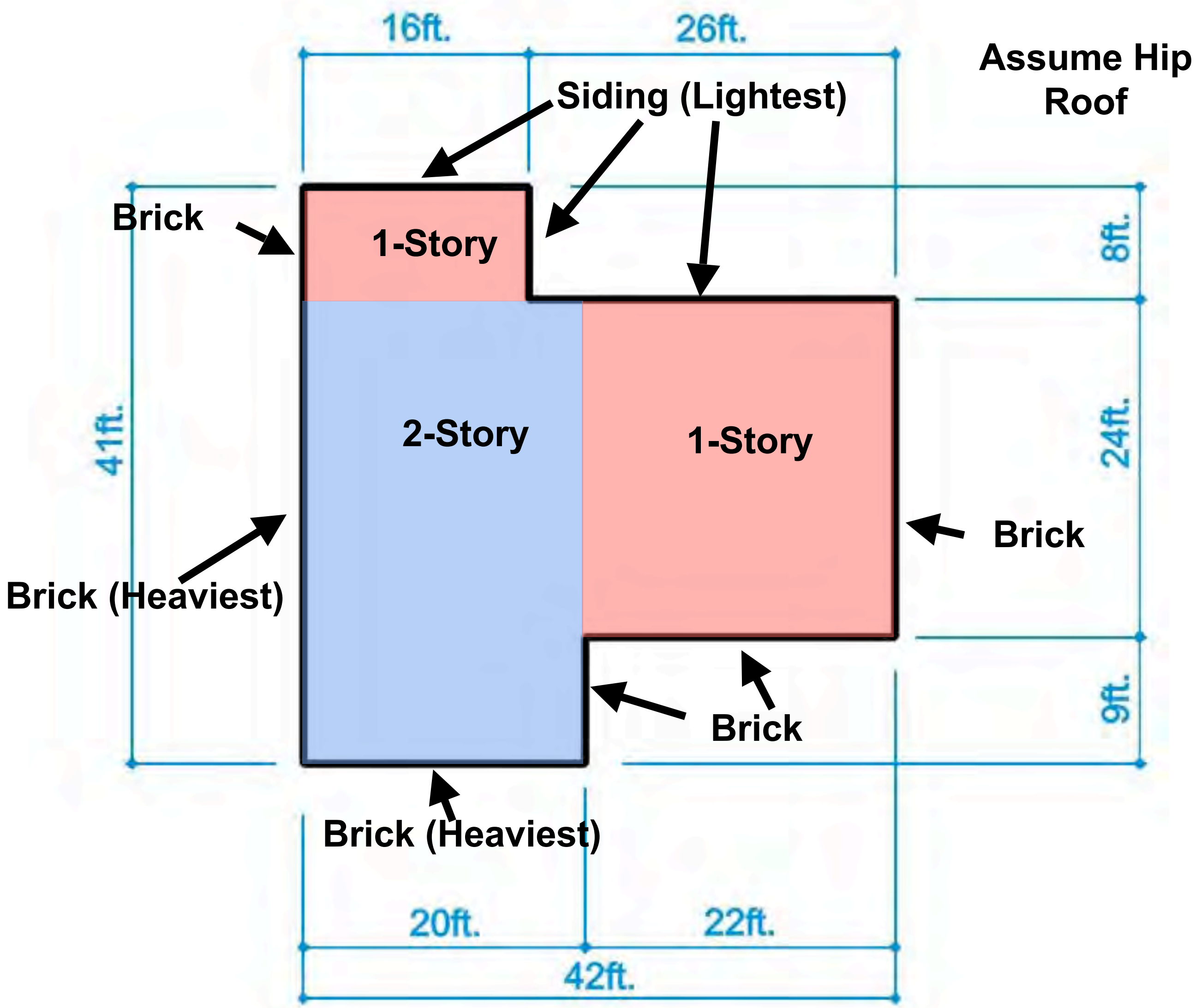
- Allowable bearing **- 2500 psf**
- Soil Support Parameters:

	Center	Edge
e_m (ft)	4.4	4.8
y_m (in)	2.01	1.27

- Soil Modulus of Elasticity **- 1000 psf**

The soil modulus of elasticity (E_{soil}) is commonly assumed to be 1000 psf. E_{soil} is not the same as the subgrade modulus (K_s)

Foundation Loads



Foundation Loads

- Average Uniform Superimposed Total Load **- 150 psf**

$$= \frac{\text{Total applied load (including perimeter load)}}{\text{square footage of design rectangle}}$$

The average uniform superimposed total load is only used in soil bearing calculations.

Foundation Loads

- Perimeter Load **- Maximum 1500 plf**

  Minimum 600 plf

  The perimeter load typically includes dead plus live load but "in the edge lift mode, designers are permitted, however to use dead load and sustained live load, or to use dead load only.

  PTI 4.5.4.3 – "When P varies significantly around the slab perimeter, and the ratio of largest to smallest exceeds 1.25, the largest value should be used for center lift design and the smallest value should be used for edge lift design."

Stiffness Coefficients

- Stiffness Coefficients **- Brick controls**

Center Lift	**480**
Edge Lift	**960**

PTI Table 6.2 – Recommended Values of Stiffness Coefficient C_{Δ}

Material	Center	Edge
Wood Frame	240	480
Stucco or Plaster	360	720
Brick Veneer	480	960
Concrete Masonry Units	960	1920
Prefab Roof Trusses	1000	2000

Prestress Loss

- Prestress Loss **- 15 ksi**

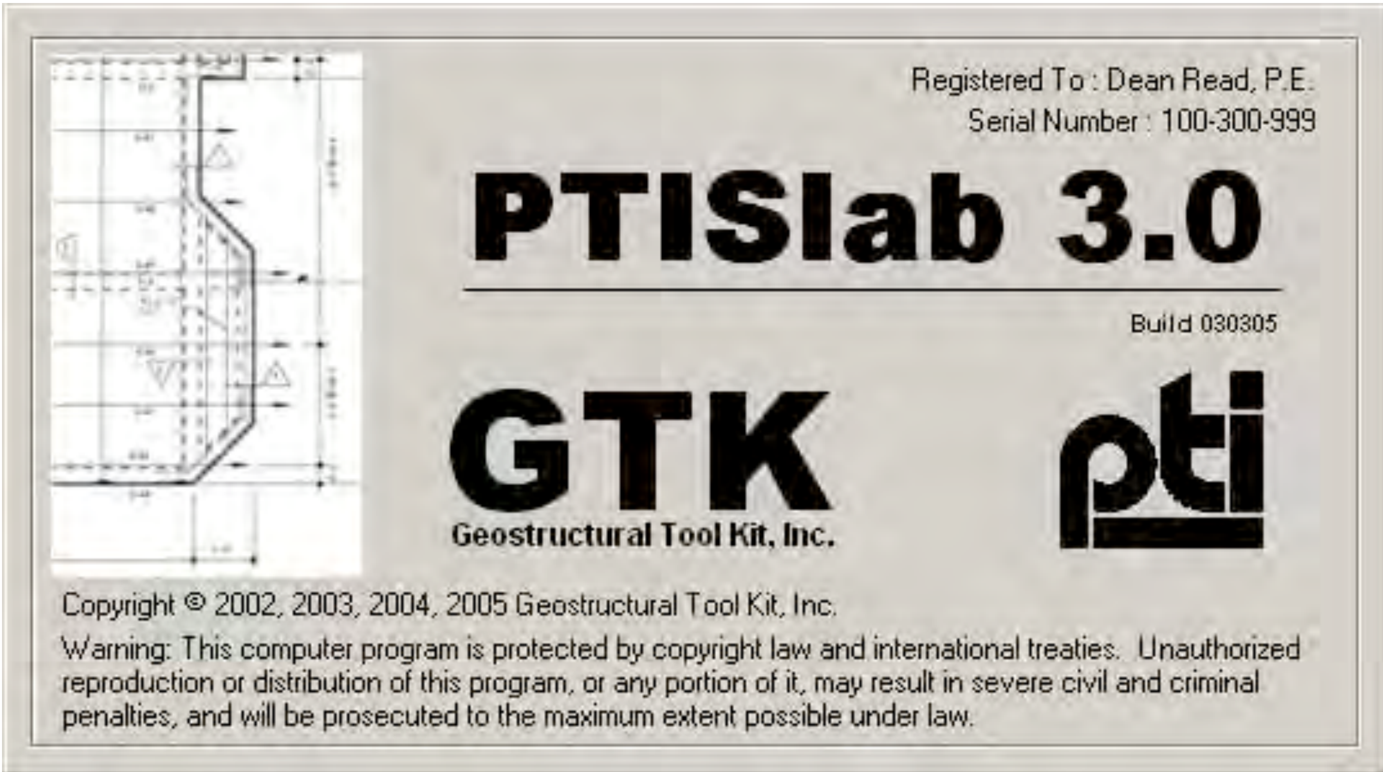
  Loss of prestress due to tendon friction, elastic shortening, creep and shrinkage of concrete and steel relaxation. In lieu of calculation, 15 ksi typically assumed

- Subgrade Friction Coefficient **- 0.75**

  Assumes slab cast directly on a sand base. Use 0.75 for slabs on polyethylene.

PTISlab 3.0

Example problem continues using PTISlab 3.0 to perform analysis.



Biography
Russell L. Price, P.E.

Russell Price is a licensed engineer in the State of Texas and is Executive Vice President of ***Suncoast Post-Tension L.P.*** *Suncoast* is the largest supplier of unbonded post-tensioning materials and services in the United States. They were established in Houston Texas in 1983 and now have 14 offices nationwide; nine of which have PTI Certified Plant production facilities. Suncoast is listed in the top #50 in Engineering News Record's listing of the Top 600 Specialty Contractors in the U.S. with sales exceeding $200 million for 2004. In 2001, Suncoast became part of The Keller Group, plc based in London England. Keller is one of the largest specialty contractors in the world. In the U.S., Keller is involved in engineered ground modification solutions through the companies Hayward Baker, Case Foundations, and McKinney Drilling as well as post-tensioning through Suncoast. Russell is an engineering graduate of The University of Texas at Arlington and has been active in the post-tensioning industry since 1971. He represents Suncoast on the Board of Directors of the Post-Tensioning Institute and is on the Executive Committee as Past-President (President for 2003-2004). He is an active member of the Unbonded Tendons, Certification (including both Plant and Field Personnel) and Slab-on-Ground committees. He co-chaired a special task group of the Unbonded Tendons subcommittee that developed recommendations for the Durability of Tendon Sheathing and Anchorages and also chaired the task group that wrote the manual *Construction and Maintenance Procedures for Post-Tensioned Slab-on-Ground Foundations* . He is also a member of American Concrete Institute and the Structural Engineers Association of Texas.

Post-Tensioned Prestressed Concrete

Post-Tensioned

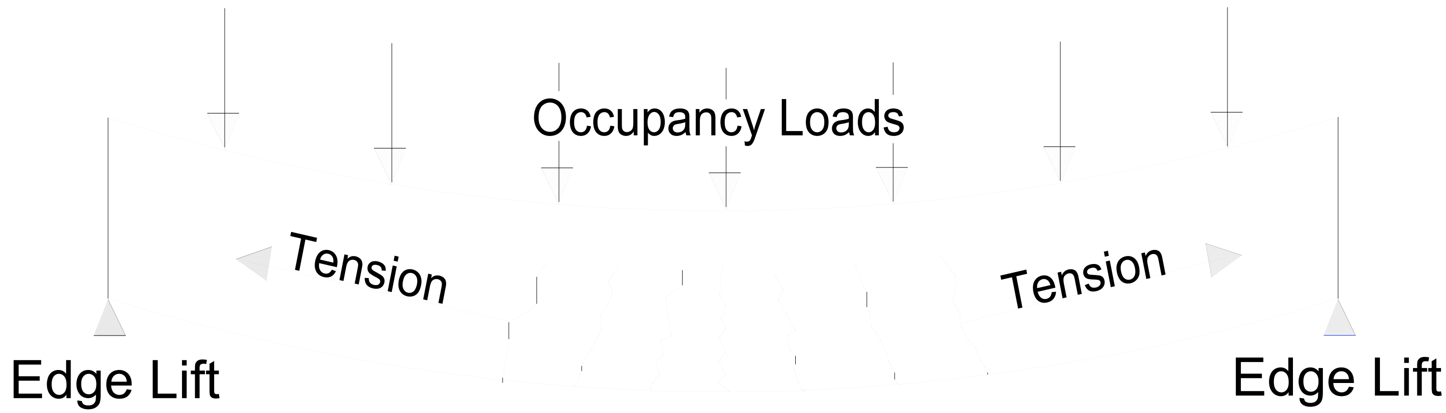
Slab-on-Ground

Construction

Reinforced vs Plain Concrete

It is important to recognize the difference between reinforced and plain concrete when referring to post-tensioned or conventionally reinforced concrete slabs-on-ground. The American Concrete Institute (ACI) defines *reinforced* concrete as concrete designed to satisfy the minimum requirements of the code. The category of plain concrete was developed by ACI to refer to concrete used in ground supported construction where loads are light, stresses are low, life safety concerns are minimal or non-existent and the minimum requirements for reinforced concrete are not necessary.

How Prestressing Works
The Basics



f_t = Mc /I or M/S_b

How Prestressing Works

For Slabs-on-Ground

$$f_t = M/S_b - \color{red}{(P/A)}$$

Advantages of Post-Tensioning

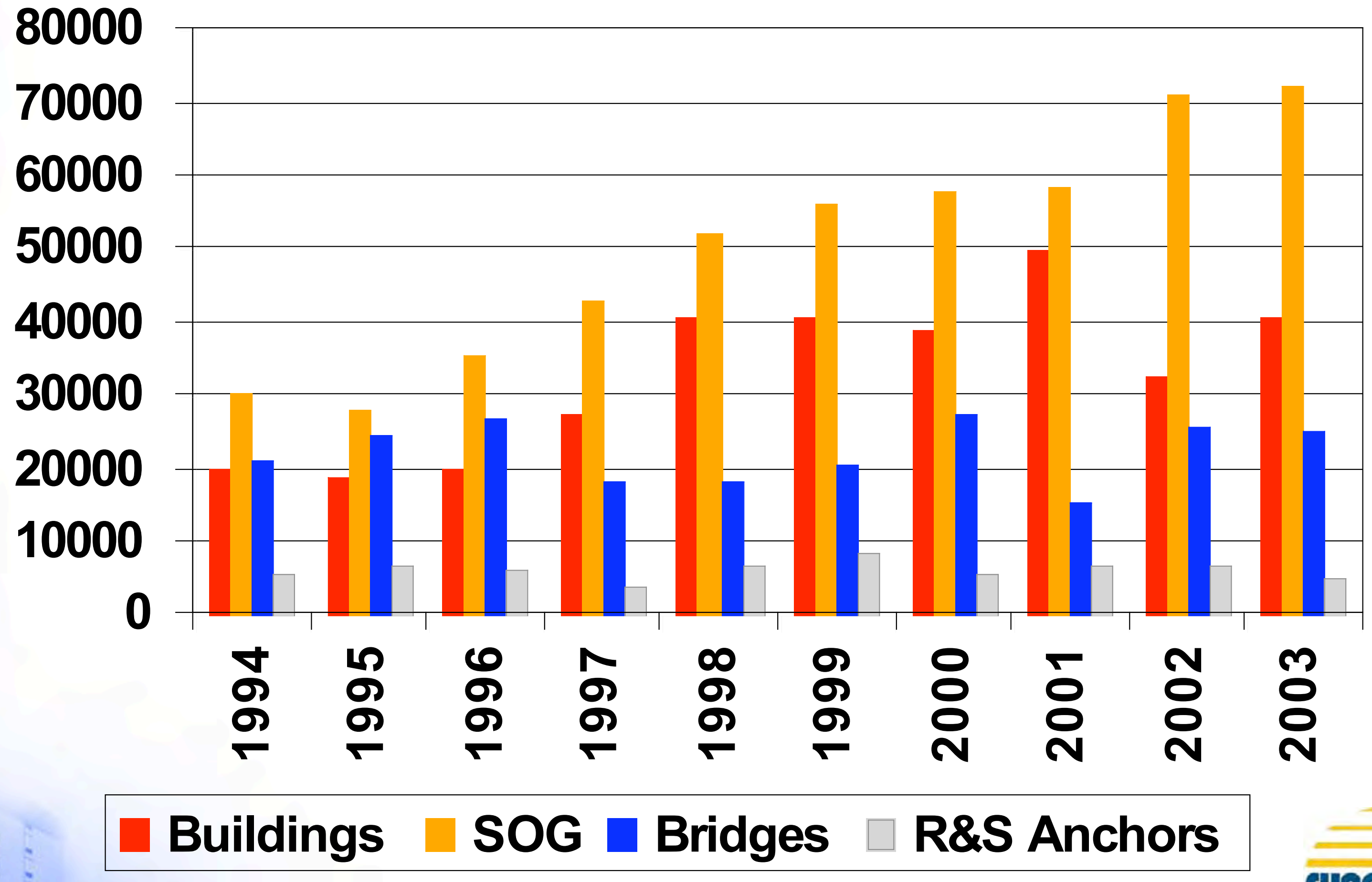
- Inherent compressive strength of concrete
- Less structural depth to achieve same design strength
- Economical use of building materials
- Speed of construction

Applications of Post-Tensioning

Slab-on-Ground Foundations

- **Single Family Residential**
- Multi-Family Residential
- Commercial / Industrial
- Sports Courts

Post-Tension Steel Tonnage Comparison



Foundations Types

Classified by BRAB Report 33

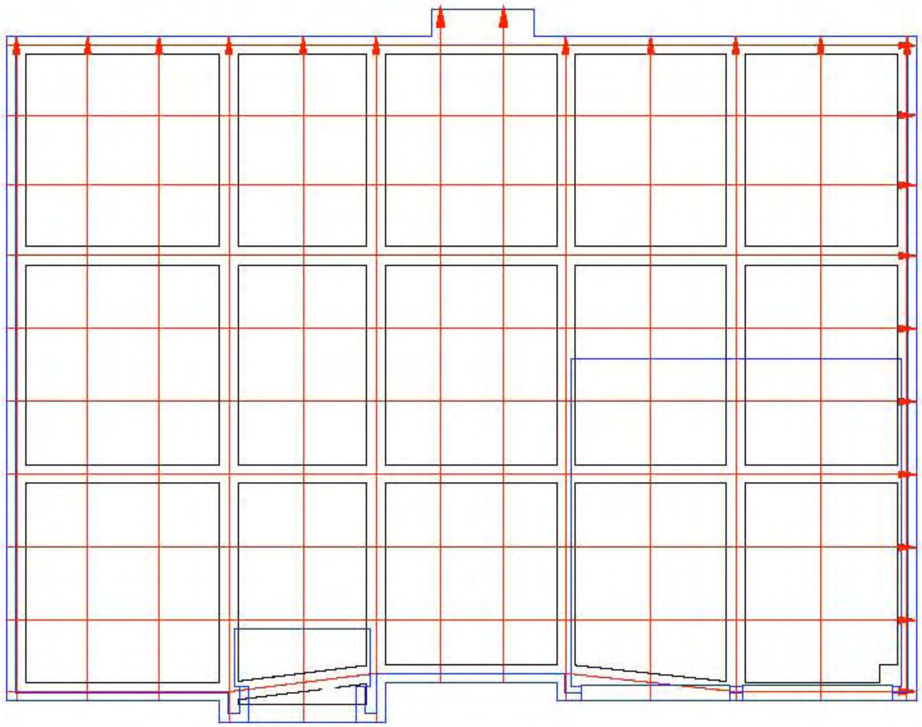
In 1962, The Building Research Advisory Board (BRAB) issued Report 33 which classified residential foundations into four design categories based upon the degree of severity of the supporting soil:

- Type I: Unreinforced
- Type II: Lightly Reinforced
- **Type III: Reinforced and Stiffened**
- Type IV: Structural

PTI Design Procedure

Slab-on-Ground Foundations

The PTI design procedure for expansive soil is based on a ribbed slab layout.



The "key-word" is Slab-on-GROUND

- Slabs-on-Ground are designed to meet a specific set of soil "parameters".
- The performance of ANY foundation is dependent up on obtaining accurate soil information about the site.
- Expansive Soils
  - Plasticity Index (PI) Greater than 15
  - Expansion Index (EI) Greater than 20
    - Em, Ym, Bearing Capacity

Site Inspection

Inspect site to look for unusual conditions

- Drainage ditches or low areas that hold water
- Trees that can influence soil moisture
- *Anything* that appears “out of the ordinary”
- Contact the geotechnical and structural engineer for recommendations

Site Preparation

Strip Site of Organics & Trash

The site should be initially stripped of all surface vegetation and other deleterious material.

Site Preparation

Recompact Scarified Surface Material

The exposed subgrade should be scarified and recompacted.

Site Preparation

Identify Voids & Recompact

Remove trees, including the root system

Proof roll the site to identify any loose soil

Site Preparation

Drainage

Grade the lot for positive drainage away from the foundation during and after construction.

Site Preparation

Compaction tests should be performed on all fill material used during the site development phase. The quality, as well as the compaction, of all fill material should be documented.

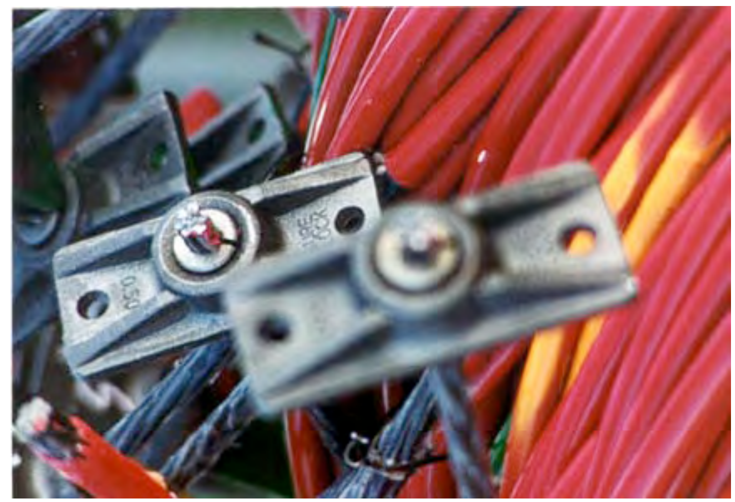
Fill should exhibit low expansion properties, be free of organics and other deleterious material, and be compatible with the existing soil characteristics.

READ the General Notes sheet prepared by the structural engineer. This sheet may contain special instructions about specific site preparation requirements.

Contact the structural engineer should anything be unclear or in question.

Components of an Unbonded PT System

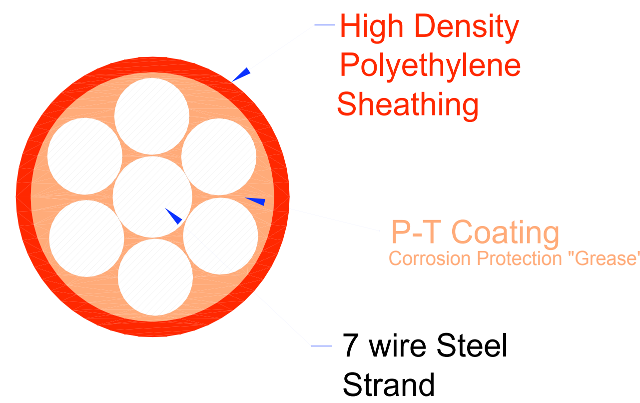
- Unbonded Strand
- Anchorages
- Accessories



Unbonded P-T Strand

Unbonded strand consists of three main components:

- Prestressing Steel
- PT Coating
- Sheathing

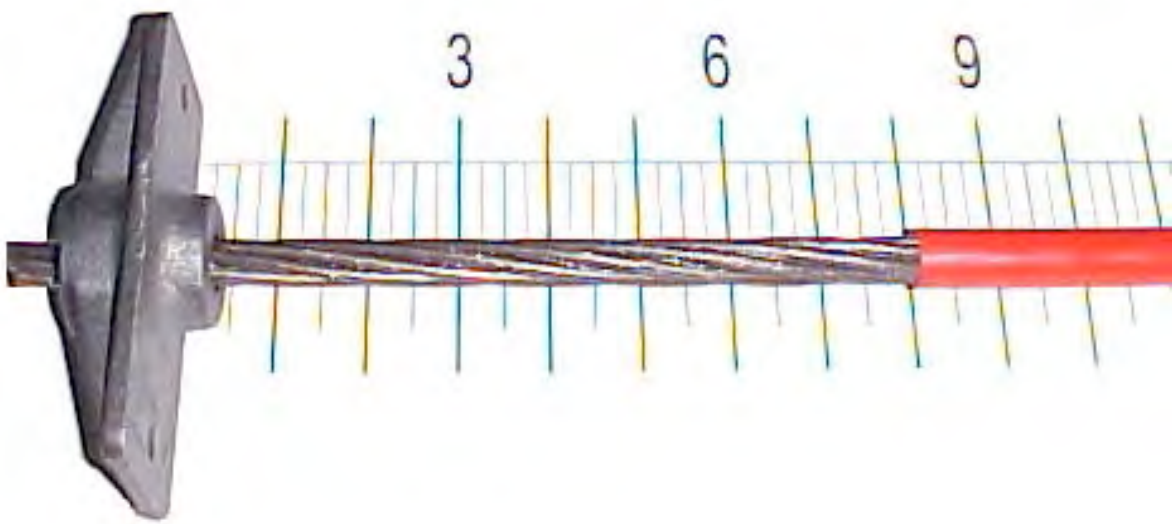


Anchorage System

Anchor

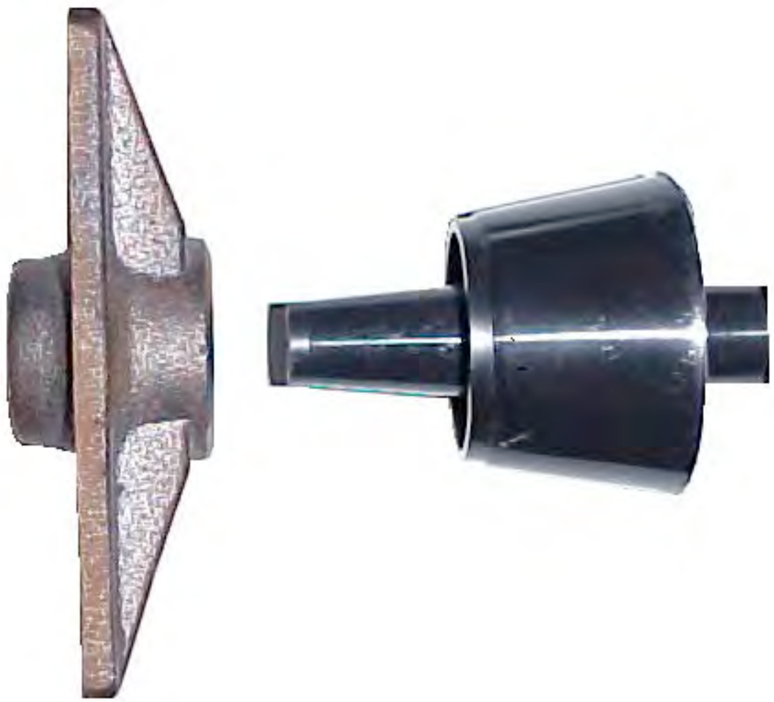
2-pc Wedge

Standard System
Fixed End Anchorage



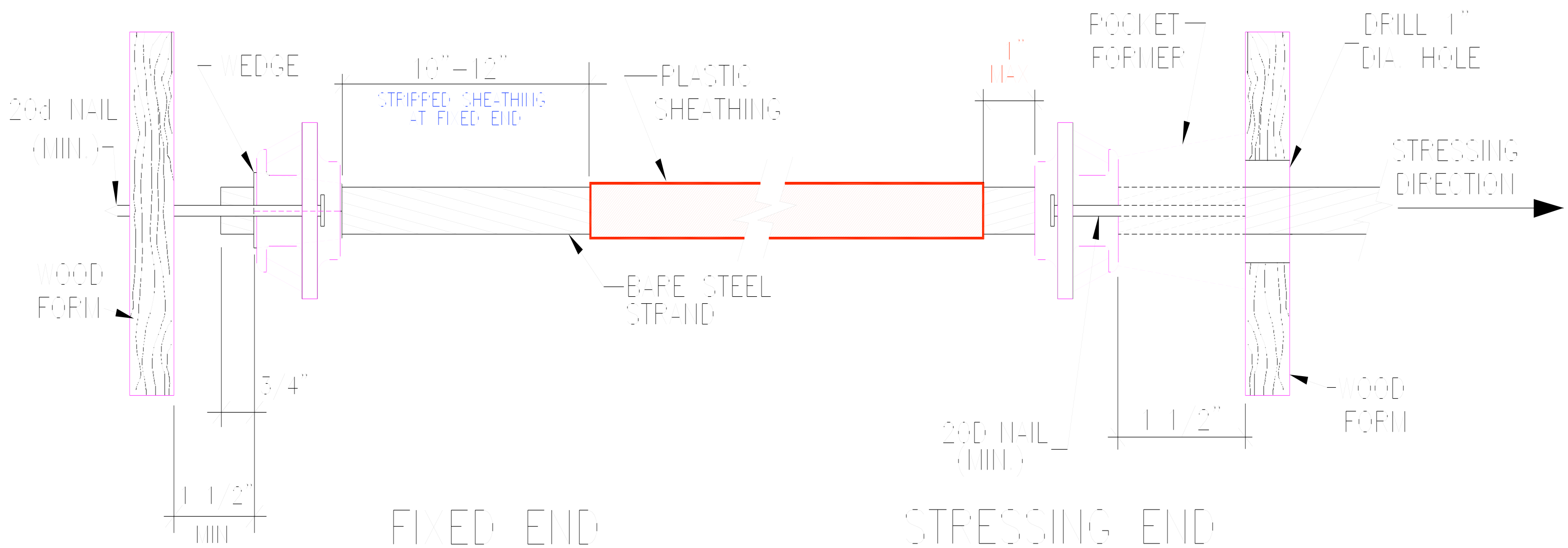
Standard System
Stress End Anchorage

Anchor



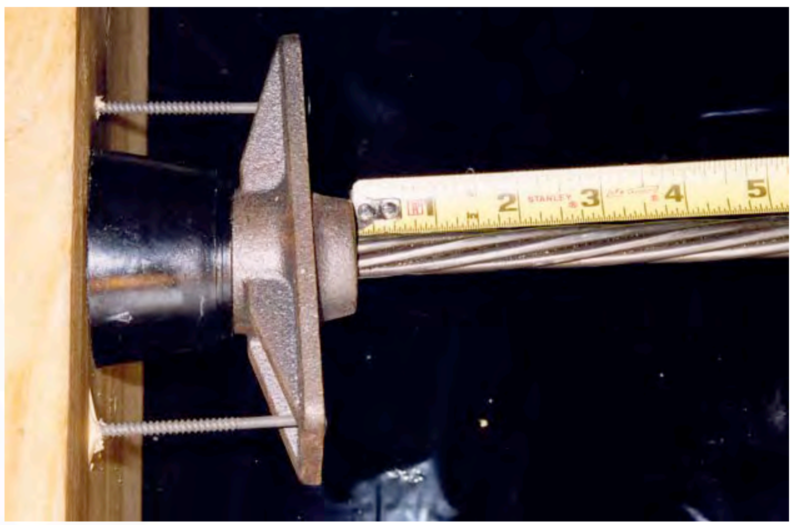
Pocket Former

Standard System Assembly

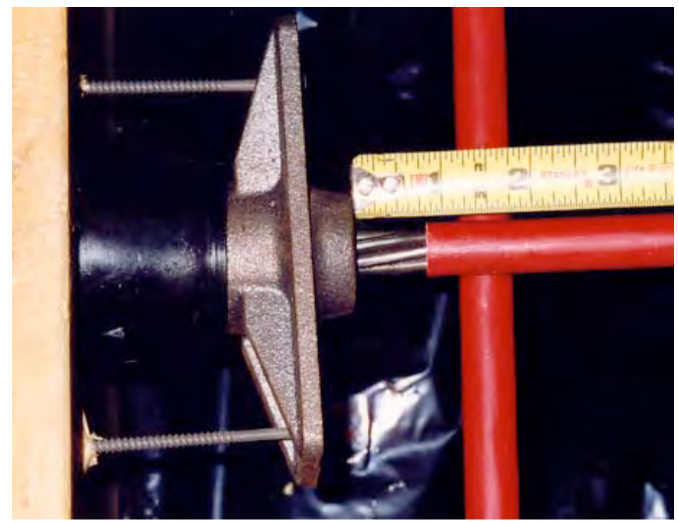


Stressing Anchorage

Incorrect



Correct



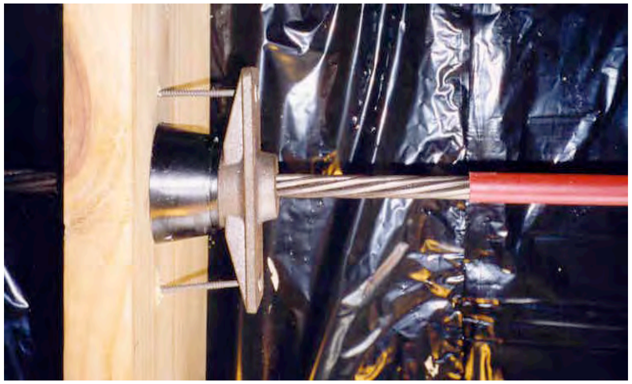
KELLER

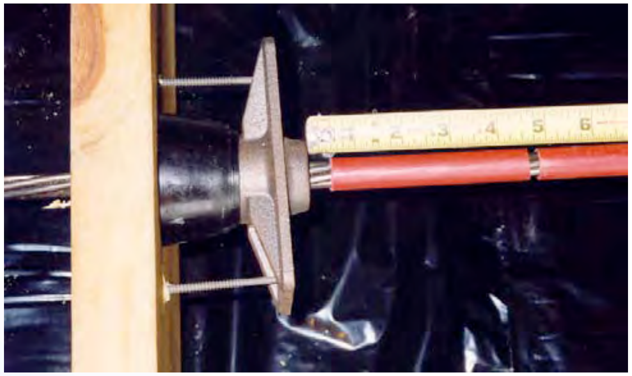
SUNCOAST

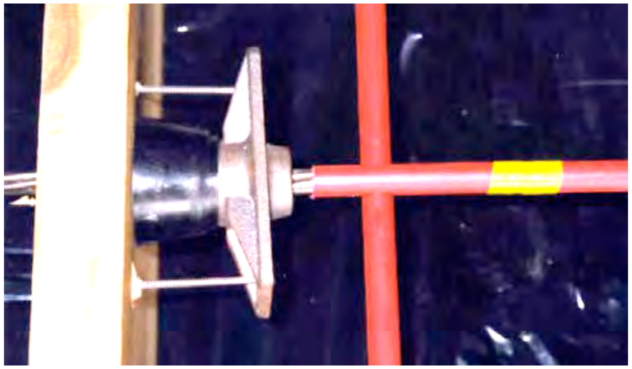
Stressing Anchorage

Tendon Sheathing Repair

- Replace Sheathing
- Tape
- 1 inch Rule

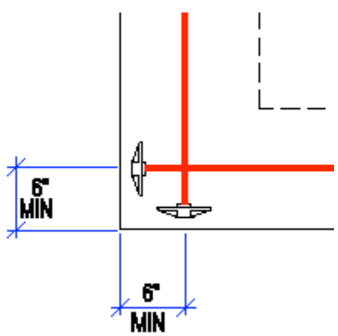


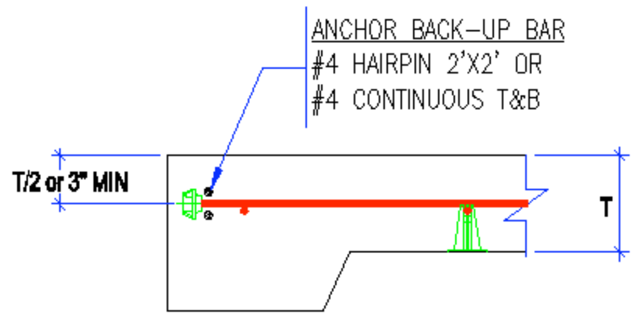




Installation

- Follow approved engineers drawings
- Anchors are placed 6 inches from edge
- Anchors are placed at T/2 or 3” below top of slab
- Anchors are **securely** attached to edge forms



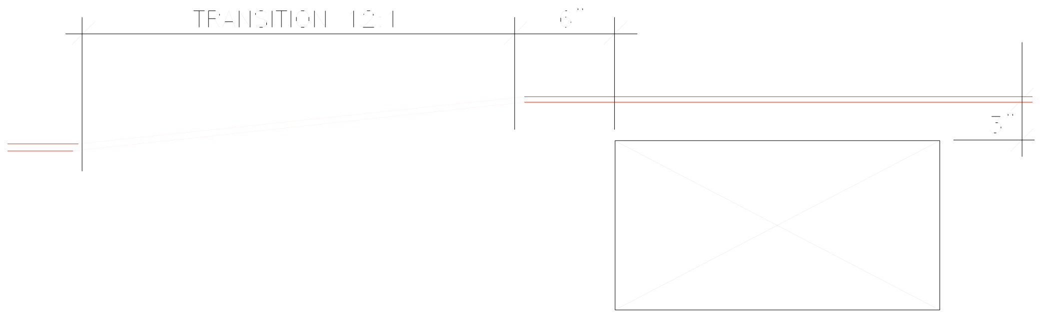


Installation

- Follow approved engineers drawings
- Anchors are placed 6 inches from edge
- Anchors are placed at T/2 or 3” below top of slab
- Anchors are **securely** attached to edge forms

Installation

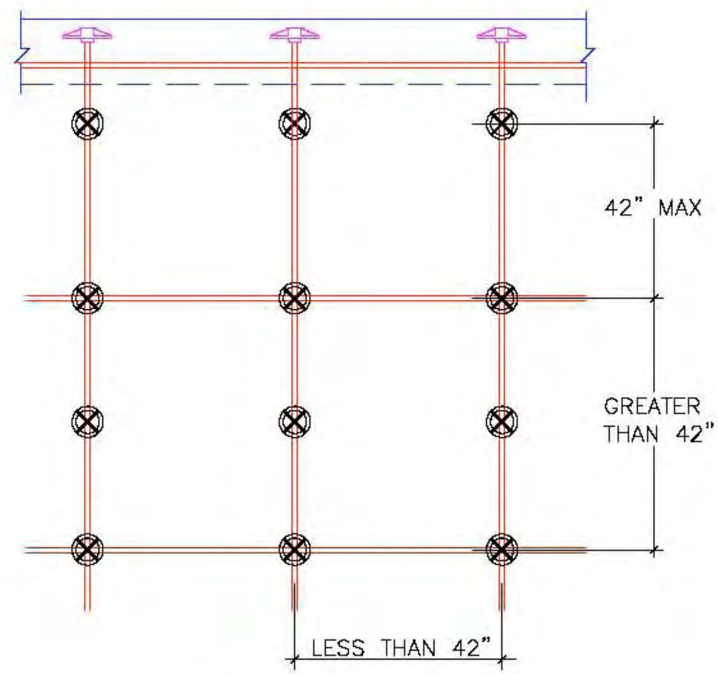
Horizontal transitions are smooth, clear all openings by 3 inches and are straight past the opening

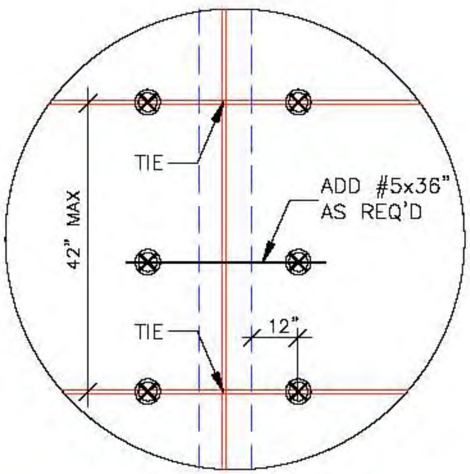


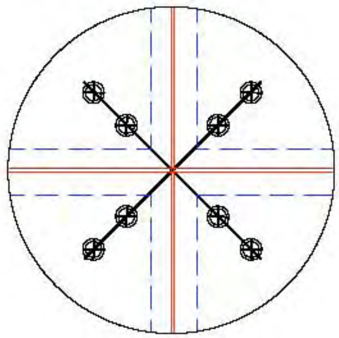
Installation

Chair Placement

Chairs are placed and tied at each tendon intersection at less than 3'-6" centers







Installation

Chair Placement

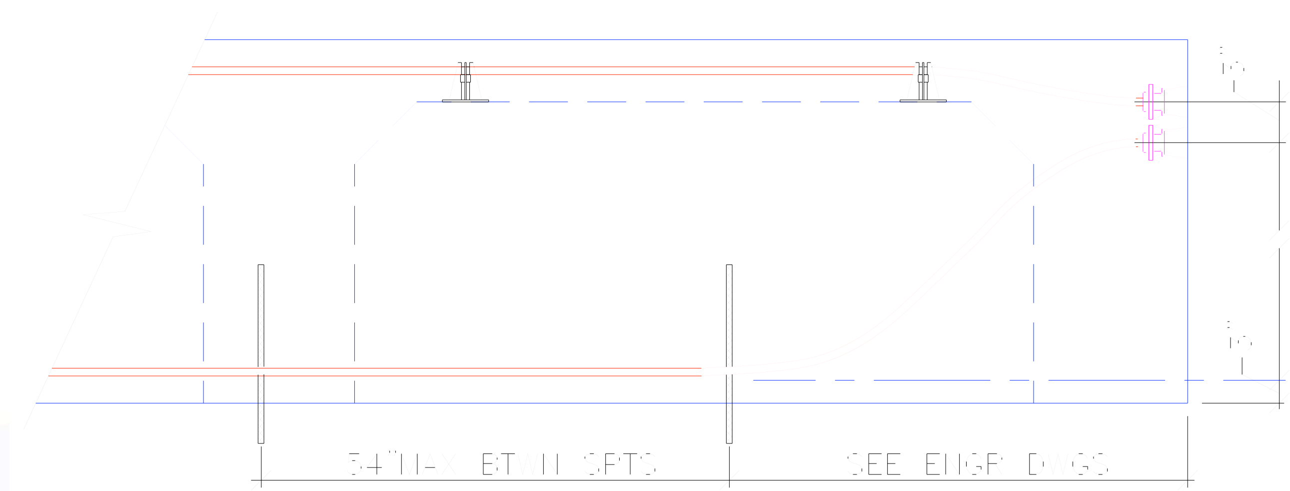
Correct

Incorrect

SUNCOAST

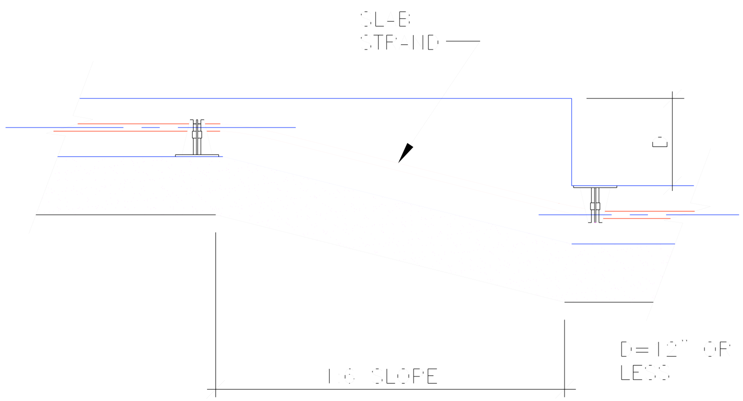
Installation

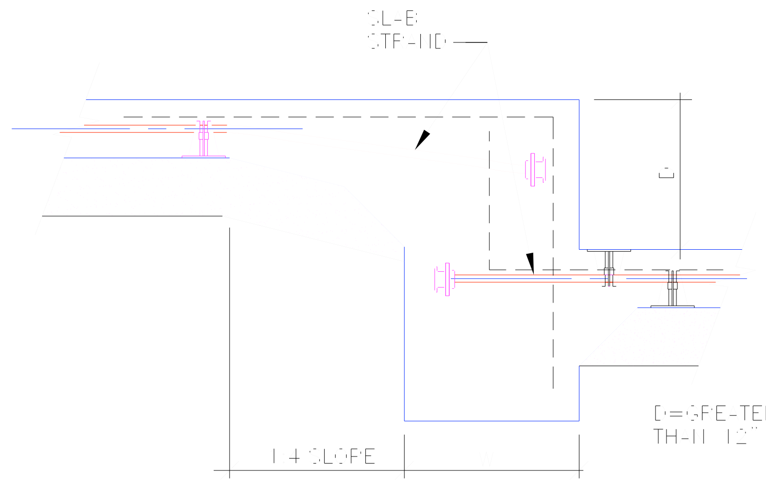
Draped tendons are used in some designs to provide compression in the bottom of beams resisting tensile stresses caused from edge lift.



Installation

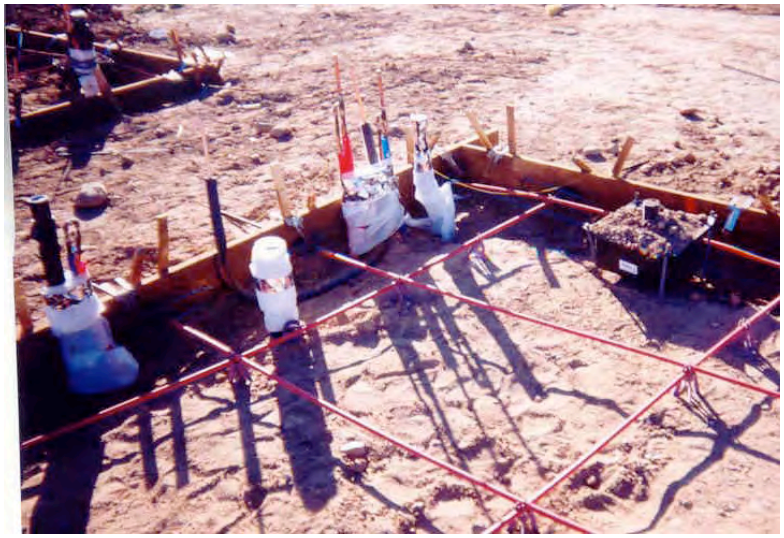
Drops in Slabs





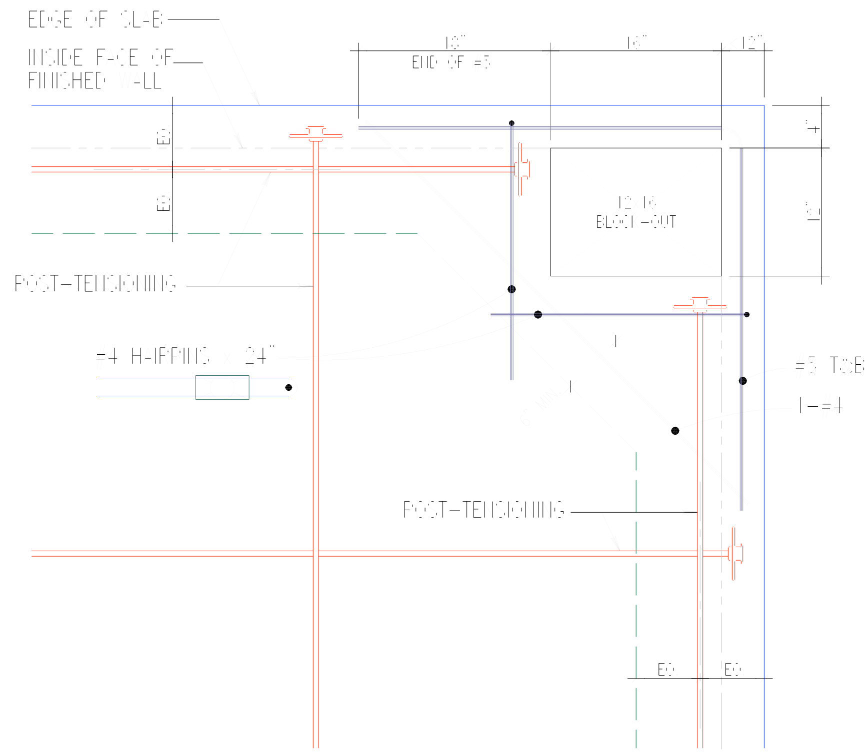
Plumbing Blockouts

- Deflect tendons to avoid blockouts through the slab.
- Maintain minimum of 3” concrete cover between tendon and blockout



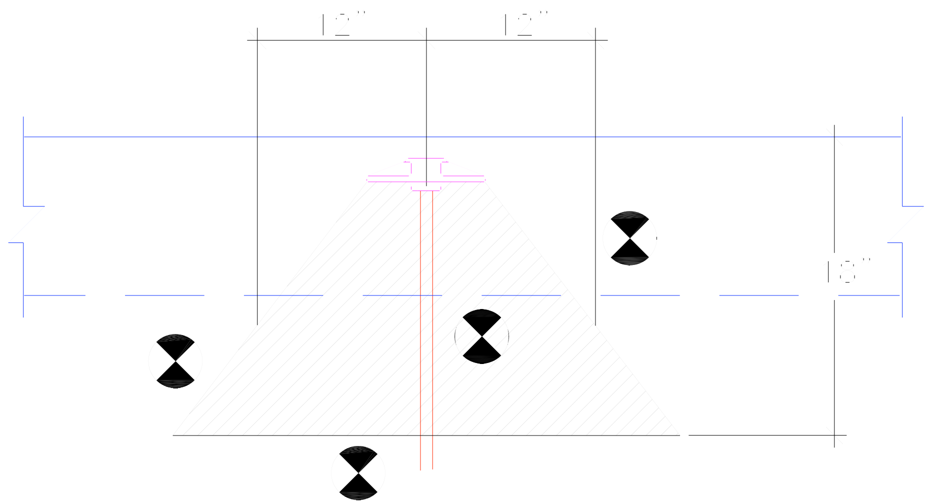
Plumbing Blockouts

Corner blockouts can cause concreting difficulties. Rebar solutions can alleviate the congestion and prevent corner failures



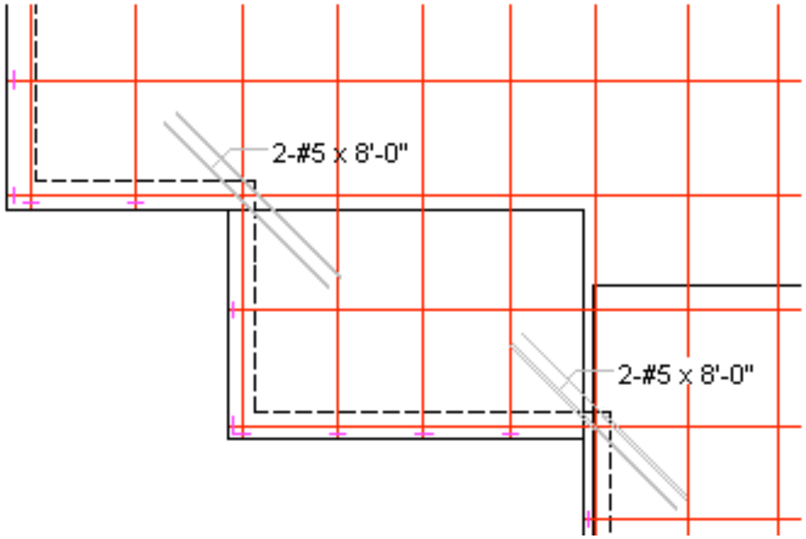
Anchorage Zone Penetrations

Penetrations through the anchorage zone should be sleeved with schedule 40 pipe.



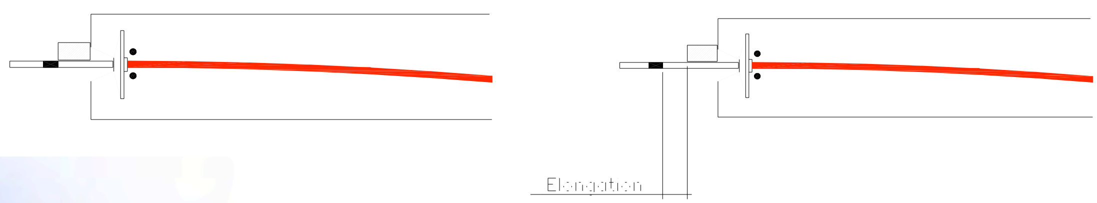
Reentrant Corners

Initial curing tensile stresses build-up at reentrant (inside) corners causing cracking to occur. Rebar is typically installed at these locations.

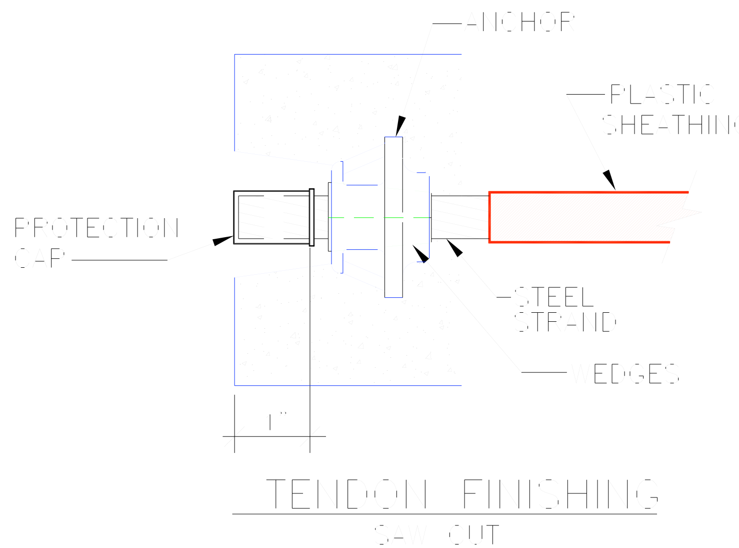


Stressing

- Initial Concrete Strength (2,000 psi <u>MIN</u>)
- Initial Marking (Elongations)
- Calibration of Jack/Gauge
- Stressing Force vs. Gauge Pressure
- Final Elongation Measurement



Cutting Stressing Tails



Finishing Stressing Recess

- Clean the pocket former recess of any dirt, grit, oil or other material that will prevent the grout from bonding to the concrete

- Grout pocket former recess with non-shrink cement, sand and water mix that will reach the minimum compressive strength of the concrete slab.

Inspections

Inspections should be conducted to insure the quality of the construction.

- Pre-pour: Installation of P-T and rebar
- Pour: Placement of Concrete
- Stressing: Tensioning of P-T Tendons

Pre-Pour Inspection

- Check the tendon and bar count
- Check the placement of the fixed-end anchors for the required cover over the end of the steel
- Check to be sure that the stressing ends are securely nailed to the forms (2 nails) and that the tendon is perpendicular to the anchor
- Check to be sure that you have adequate concrete cover over all of the steel

Pour Inspection

- Do not add excessive water at the site
- Place the concrete in one continuous operation - NO COLD JOINTS
- Consolidate (vibrate) the concrete around the P-T anchors, especially in the corners where multiple anchors are located.

Pour Inspection

<u>DO NOT</u> displace or walk on any of the reinforcement

- P-T or rebar

Post-Concrete Placement

- Notify the P-T stressing company the day after the concrete is placed to schedule stressing
- Remove the forms - ALL OF THEM. Do this within 3 days after the concrete is placed
- Do NOT damage stressing tails in rough grade
- Do NOT cover tendon tails with dirt or lumber

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Trouble-Shooting

- Honeycombs
- Low strength and poor quality concrete
- Plumbing mistakes

Trouble-Shooting

Correct

Incorrect

Foundation Maintenance

- Do not alter the drainage pattern of the site
- Provide a minimum of 3%-5% of slope away from the foundation with the first 5 feet
- Roof drains should not discharge water at the perimeter of the foundation

- Do not plant trees within the tree's drip line
- Do not cut off natural moisture around the foundation by constructing decks or pools

Performance

The long-term performance of <u>any</u> slab-on-ground foundation is dependent upon good drainage and a moisture maintenance program by the property owner.

PTI Certification Program

- Consistency of material quality
- Quality of extrusion and fabrication process
- Traceability of components
- Stressing equipment calibration
- Company commitment to the long term performance of the system

POST-TENSIONING INSTITUTE
CERTIFIED PLANTS

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Construction & Maintenance

More information concerning the construction & maintenance of post-tensioned slabs-on-ground, can be found in a manual published by the *Post-Tensioning Institute.*

The 3rd Editions is currently being reviewed by committee and will be available in the next few months.

602-870-7540

www.post-tensioning.org

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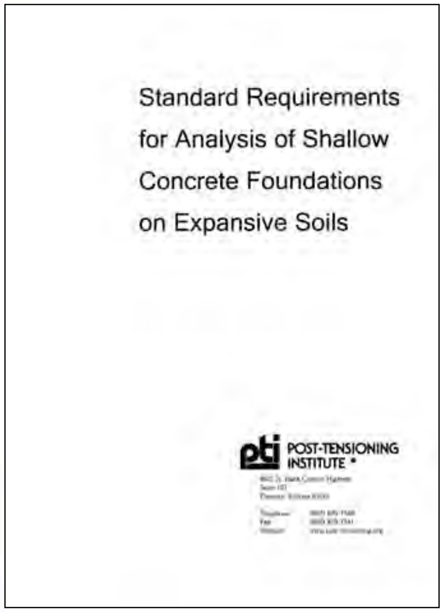
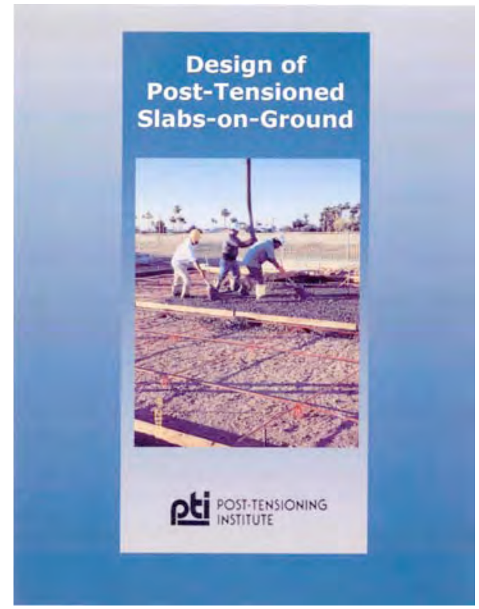
Design Procedures & Code

New in 2005





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LIABILITY MANAGEMENT IN FOUNDATION ENGINEERING

The management of a construction project requires a detailed understanding of the "construction" process. A project manager must understand the requirements of each trade and the order and process of construction. As with construction management, the liability management also requires a thorough understanding of the "liability" process. While the "nuts and bolts" of the legal process should be implemented by an attorney, foundation engineers must understand the process so that they can effectively plan for when things go wrong. A thoughtful and complete liability management plan can minimize the potential economic impact.

Liability management is defined as the application of general liability principles, in conjunction with available tools, to minimize the potential economic impact of an incident. To effectively manage liability, one must first understand the theory of liability, the basics of which are described in Section 1. Certain principles relating to liability theory in the engineering context are discussed in Section 2. And, commonly used tools to minimize potential liability are discussed in Section 3. With this introduction, a company should evaluate their current liability management plan.

David T. Dorr, PE, esq., is the author of this paper, and the Law Office of David T. Dorr, PC, is solely responsible for its content. The contents of this paper are intended for information only; the information should not be substituted for legal advice, as specific legal advice should be obtained from an attorney after consultation.

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September 23, 2005

Table of Contents

About the Author

David T. Dorr, PE, is a solo practitioner with an office in Harris County, Texas. He primarily practices in the field of small business law (business organizations, financing, employment issues, collections, and contracts) and related litigation.

Mr. Dorr attended the University of Texas at Austin and received a Bachelor of Science Civil Engineering and Master of Science Engineering in Structures. He also attended the University of Houston Law Center and received a Doctor of Jurisprudence.

Prior to practicing law, Mr. Dorr was a forensic engineer who investigated property damage related to fires, floods, hurricanes, and other disasters. He occasionally offers engineering consulting under his firm, Sigma Consulting, Ltd.

Section 1. The Theory of “Liability”

1.1 “Liability”

Before one can manage “liability,” one must understand what the term means. To be “liable” means the state of being bound or obligated by law or justice to do, to pay, or to make something; the state of one who is bound by law and justice to do something which may be enforced by action.[1] Before a party is found “liable,” there must be an “incident” that results in “damage” under a recognized “cause of action,” as those terms are defined below. Managing liability includes identifying and addressing those potential areas when liability occurs.

1.2 “Incident”

First and foremost, one must identify all major or significant potential incidents. An incident, in any situation, involves the specific facts and circumstances that occur. In the context of liability, however, an incident usually involves “damages,” as defined more fully below. It is important to note that no one can possible identify every situation that may arise, so it is important to plan for the most common incidents related to your particular industry.

In the context of a foundation design, the major potential incidents for consideration are as follows:

- An *architect* retains an *engineer* to design a foundation for a residence for a *builder*. After the house is complete, it is sold to a *buyer*. Months after the sale, the foundation moves excessively interfering with normal use.
- A *property owner* retains an *engineer* to design a foundation for a structure that was designed by an *architect*. Months after completion, the foundation moves excessively interfering with normal use.
- A *builder* retains an *engineer* to design and inspect a foundation for a specification home, which has been sold to a *buyer*. Months after completion, the foundation moves excessively interfering with normal use.

[1] Black’s Law Dictionary, sixth edition, citing *Fidelity Coal Co. v. Diamond*, 310 Ill.App. 387, 34 N.E.2d 123; *Clark v. Lowden*, D.C.Minn., 48 F.Supp. 261, 263.

- An *architect* retains a *geotechnical engineer* to perform soil testing and provide recommendations that a *structural engineer* uses to design a foundation.

In each of the scenarios above, a party is potentially liable to another. As later discussed, the engineer's obligations (and potential liability) to other parties may be very different. In developing a liability management plan, one should identify the major potential incidents, as well as the percentage of likelihood for each. By doing so, one can identify where to focus the majority of his efforts in minimizing potential liability.

1.3 Potential Parties

As illustrated in the examples above, in residential and light commercial building, there are a host of potential parties involved in the construction project: the architect, the contractor, any subcontractors, other engineers, developers, etc. One must not forget the current owner, future owners, leaseholders. Given any "incident," or set of facts, the foundation engineer may potentially be "liable" to any of these.

All parties, however, can be easily classified into two simple groups: *client* and *nonclient*. The classification also has important engineering ethical considerations, i.e., duty of confidentiality. The designation, from a liability perspective, also has significant implications, as revealed later. In developing the liability management plan, one should classify each of the potential parties.

1.4 Causes of Action

Before a defendant can be "liable," the plaintiff must plead and prove, by a preponderance of the credible evidence, that the defendant is "liable" under a theory of recovery, or "cause of action," recognized under Texas law (not all theories are recognized). A cause of action has certain elements, all of which must be proven. Failure to prove any one element is fatal in proving liability.[2] In actions against engineers, plaintiffs most often plead and attempt to prove one or more of the following:

[2] See *Cathey v. Booth*, 900 S.W.2d 339, 341 (Tex. 1995).

- Negligence. The elements of a cause of action for negligence are: (1) the defendant owed a legal duty to the plaintiff; (2) the Defendant breached that duty; and (3) the breach proximately caused the plaintiff's injury.[3]
- Negligent misrepresentation. The elements of a cause of action for negligent misrepresentation are the following: (1) the defendant made a representation to the plaintiff in the course of the defendant's business or in a transaction in which the defendant had an interest; (2) the defendant supplied false information for the guidance of others; (3) the defendant did not exercise reasonable care or competence in obtaining or communicating the information; (4) the plaintiff justifiably relied on the representation; and (5) the defendant's negligent misrepresentation proximately caused the plaintiff's injury.[4]
- Breach of Contract. The elements for a cause of action for breach of contract are: (1) there is a valid, enforceable contract; (2) the plaintiff and the defendant are in privity[5]; (3) the plaintiff performed, tendered performance, or was excused from performing its contractual obligations; (4) the defendant breached the contract; and (5) the defendant's breach caused the plaintiff's injury.[6]
- Violation of the Deceptive Trade Practice Act. The elements of a DTPA claim are: (1) the plaintiff is a consumer; (2) the defendant can be sued under the DTPA; (3) the defendant committed a wrongful act; and, (4) the defendant's action was a producing cause of the plaintiff's injuries.[7]

Each incident may have one or more causes of action. When evaluating potential liabilities, one should identify potential causes of action that parties may assert against him, as managing each potential liability is a function of the cause of action.

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[3] See *D. Houston, Inc., v Love*, 92 S.W.3d 450, 454 (Tex. 2002).
[4] See *McCamish, Martin, Brown & Loeffler v. F.E. Appling Interests*, 991 S.W.2d 787, 791 (Tex. 1999).
[5] "Privity" is established by proof that the plaintiff and the defendant were parties to the enforceable contract and had obligated themselves under it. See *C&C Partners v. Sun Exploration & Prod.*, 783 S.W.2d 707, 721 (Tex.App.—Houston [1sth Dist.] 1997, no writ).
[6] See *Valero Mktg. & Sup. Co. v. Kalama Int'l*, 51 S.W.3d 345, 351 (Tex.App.—Houston [1st Dist.] 2001, no pet.)
[7] See *Amstadt v. U.S. Brass Corp.*, 919 S.W.2d 644, 649 (Tex. 1996).

1.5 "Damages"

Texas law recognizes three main categories of damages: actual, nominal, and exemplary (punitive) damages. Actual damages, also called "compensatory damages," are awarded to repair a wrong or to compensate for an injury.[8] Actual damages are further categorized into "general" damages, which are those that are necessary and a usual result of the defendant's wrongful act, and "special" damages, which are those that result naturally, but not necessarily from the defendant's wrongful act.[9]

Depending on the cause of action, the plaintiff can recover for damages to real property. Damages to real property depend on whether the injury to land is "permanent," or "temporary." A permanent injury consists of a constant and continuous injury, not an occasional, intermittent, or recurrent one.[10] Permanent injury to land is measured by "valuation damages," as determined by market value. A temporary injury to land is sporadic and intermittent, not continuous.[11] Temporary injury to land is measured by cost to repair, as determined by the cost and expense of restoring the land, plus any loss for being deprived of use.

The plaintiff may also recover for damages to improvements. Whether the plaintiff can recover valuation damages or repair damages for injuries depends on whether the improvement can be restored to its original condition. When the improvement is destroyed, valuation damages are appropriate. If the improvement can be repaired, repair damages are appropriate.

In a personal injury case, a plaintiff can recover damages for past and future mental anguish. Generally, however, the plaintiff must be physically injured unless: there was intentional or malicious conduct; there was a breach of duty arising from a special relationship; or it involved particularly disturbing events. Courts have held that a plaintiff whose real property has been damaged by the defendant's negligence cannot recover mental anguish damages.[12]

The payment of the opposing party's attorney fees, while not unusually considered "damages" per se, can be a significant economic factor to consider in the

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[8] See *Robertson Cty. V. Wymola*, 17 S.W.3d 334, 344-44 (Tex.App.—Austin 2000, pet. denied).
[9] See *Arthur Anderson & Co, v. Perry Equip. Corp*. 945, S.W.2d 812, 816 (Tex. 1997).
[10] See *Bayouth v. Lion Oil Co.*, 671 S.W.2d 867, 868 (Tex. 1984).
[11] See *Atlas Chem. Indus. v. Anderson*, 524 S.W.2d 681, 685 (Tex.1975).
[12] See *City of Tyler v. Likes*, 962 S.W.2d 489, 497 (Tex. 1997).

management of liability. Under Texas law, a person may recover reasonable attorney's fees from an individual or corporation, in addition to the amount of a valid claim and costs, if the claim is for: (1) rendered services;(2) performed labor; (3) furnished material; (4) freight or express overcharges; (5) lost or damaged freight or express; (6) killed or injured stock; (7) a sworn account; or, (8) an oral or written contract.[13] Other statutes, such as the DTPA, may also specifically permit the recovery of attorney's fees.

In the context of foundation engineering, therefore, the major potential damages measurements for consideration are as follows:

- Costs to repair, plus damages for loss of use
- Valuation damage
- Attorney's fees to the extent they apply

When managing liability, one should estimate the amount of potential damages in the event that something goes wrong. Criteria can be established to identify high-damage potential projects.

1.6 Liability Management

In this section, we examined the theory of liability. In review, liability means that Texas law recognizes a cause of action whereby a party is obligated to pay another for damages resulting from an incident. In the next section, we examine certain principles of liability that are specific to engineering and related professions.

[13] TEX. CIV. PRAC. & REM. CODE ANN §38.001.

Section 2. Certain Principles of Liability Related to Engineers

2.1 Sources of Principles

Now that we have discussed the theory of liability, we shall turn to certain principles of liability specifically related to engineers and related professions. These principles are based upon certain case law in Texas. Where available, cases relating specifically to engineers are mentioned. However, case law relating to other professions is discussed as potentially applicable. In theory, it is the law of the land. In actuality, the courts decided the specific cases on the specific facts. An extrapolation of a principle may provide little, in any, legal protection. And a different court may decide the same cases differently. Nonetheless, the cases are worthy of consideration because the principles can be utilized in liability management. Other sources of principles may also apply, which include statutes, such as the Deceptive Trade Practice Act, and rules of professional ethics.

The principles of engineering liability are highly complex and detailed. The following cases are merely introductory and are interesting in nature. These principles are offered merely for discussion and not as legal advice. Any legal advice should be obtained from an attorney after consultation.

2.2 The Architect's Duty to Client

The Corpus Christi Court of Appeals considered the duty of an architect to his client in *Ryan v. Morgan Spears Associates*.[14] That case was, in part, an action for negligence in the performance of professional services. The homeowner retained an architect to develop certain plans and specifications. The architects hired Wally Wilkerson, a structural engineer, to prepare the plans and specifications for the foundation of the building. Within months, the building began to deteriorate; the concrete floor cracked in several places, the air conditioner ducts pushed out of the walls, the doors warped and the concrete block walls showed signs of separation. Upon examination, the architect and engineer concluded that the damage was caused by massive water migration that resulted in the slab to heave upward in some areas. They

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[14] See *Ryan v. Morgan Spear Associates, Inc.,* 546 S.W.2d 678 (Tex.Civ.App.---Corpus Christi 1977, writ ref'd n.r.e).

recommended peripheral drainage of the building, particularly the critical areas. The plaintiff, the homeowners, sue the architect alleging that the defendant architect was negligent 1) in failing to "properly test the soil at the time and place in question", and 2) in failing "from what soil tests were made, to draw up proper plans and specifications under the attending circumstances and specifically, plans and specifications for a suitable foundation." The Court stated that an architect must use the skill and care in the performance of his duties commensurate with the requirements of his profession, and he is liable in damages if he is negligent in performing such duties. Citing legal reference materials,[15] however, the Court noted that:

> The architect, by his contract, implies his possession of ordinary good taste, skill and ability, and a promise to exercise them reasonably, without neglect, and with a certain exactness of performance, in seeing that the work is properly done. The degree of skill required is such as would produce, if followed, a building of the kind called for, without marked defects in character, strength or appearance.
>
> In the absence of special agreement, *an architect is not liable for faults in construction resulting from defects in the plans, as his undertaking does not imply or guarantee a perfect plan or a satisfactory result*, it being considered enough that the architect himself is not the cause of any failure, and there is no implied promise that miscalculations may not occur. Thus, *an architect is only liable for a failure to exercise reasonable care and skill* [emphasis added].

This case, in the author's opinion, is a bad principle. As discussed later, professional negligence in the context of engineering services means doing that which an engineer of ordinary prudence in the exercise of ordinary care would not have done under the same or similar circumstances or failing to do that which an engineer of ordinary prudence in the exercise of ordinary care would have done under the same or similar circumstances. Depending on the incident, a different court may conclude that a similar "miscalculation" constitutes negligence. Thus, a court may decide the *Ryan* case differently today.

[15] 6 C.J.S. Architects § 27.

2.3 Professional's Duty to Non-Clients

In the case *Dear v. Scottsdale Ins. Co*.,[16] the Dallas Court of Appeals considered the implication of relationship between the parties in determining whether a duty exists. In Dear, an insurance carrier retained an independent adjusting firm to do independent adjusting work in connection with pending litigation. The independent adjusting firm never had a contractual relationship with the Plaintiff insured. The Plaintiff-insured contended that, among other things, the independent contractor negligently investigated and evaluated the claims and then changed its evaluation of the case based on pressure from its client. The Court held that claims that an independent adjusting firm negligently investigated a claim against an insured must fail as a matter of law, citing case precedence.[17] *The Court held that the independent adjusting firm hired exclusively by the insurance carrier, had no relationship with, and therefore owed no duty to, the Plaintiff-insured.* The Court went on to state: *Absent such a relationship and concomitant duty, the independent adjusting firm could not be liable to Dear for improper investigation and settlement advice, regardless of whether the Plaintiff-insured phrased his allegations as negligence, bad faith, breach of contract, tortious interference, or DTPA claims [emphasis added].*[18]

The court in this case established a far-reaching principle that is potentially applicable to engineers and third-party claims.

2.4 Engineer's Duty to Non-Clients

The Corpus Christi Court of Appeals considered the implication of a "client relationship" on the required duty. In the case *Hartman v. Urban*,[19] the Court considered whether an engineer owes any duty of care in the preparation of the original plat of the subdivision or to correct of record any errors the plat contained. A developer hired an engineering firm to prepare a plat, which was filed of record. The third-party Plaintiff sued the engineer, whom they never retained, alleging that they bought waterfront land based upon representations made in his filed plat. The Plaintiff asserted that the engineer's representations in its incorrect plat constituted negligence, among other things.

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[16] See *Dear v. Scottsdale Ins. Co*., 947 S.W.2d 908, 916 (Tex.App.—Dallas 1997).
[17] See *Bui v. St. Paul Mercury Ins. Co*., 981 F.2d 209, 210 (5th Cir.1993).
[18] See *Dear*, 947 S.W.2d at 917.
[19] See *Hartman v. Urban*, 946 S.W.2d 546 (Tex.App.—Corpus Christi 1997).

In reaching its decision, the Court considered legal precedence in similar cases and certain facts, including: (1) the duty of care that the engineer owed to the third-party Plaintiff based on analogies to other similar professions: lawyers[20] and medical doctors;[21] (2) the fact that the developer hired the engineer to plat a subdivision; (3) the fact that the purpose of the subdivision was to sell lots to the public after the subdivision was approved; (4) the fact that the engineer should have known that it would be for sale to the public and would probably be bought; (5) the fact that the engineer was hired by the owner of the property to lay out the lots so the subdivision could be created; (6) the fact that the developer, once the engineer had prepared a plat to its liking, had it approved and filed it; and (7) the fact that the engineer delivered the corrected plat to the developer after the engineer corrected his mistake.

Under these facts, the Court held that no privity existed between the engineer and the Plaintiff. Thus, the Court held that the engineer did not owe the Plaintiff a duty of care in the preparation of the original plat of the subdivision, nor to correct of record any errors the plat contained.[22]

This case, in the author's opinion, is also bad principle. Courts have also consistently held that accountants[23] and lawyers[24] owe a duty to nonclients not to make misrepresentations. One could easily argue that the erroneous plot plan constituted actionable negligent misrepresentation. As such, a court may decide the *Dear* case differently today.

2.5 Engineer's Duty under Contract

The Houston [1st District] Court of Appeals considered the implications of a contract on duty. In *I. O. I Systems, Inc., v. City of Cleveland, Texas*,[25] IOI and the City of Cleveland, Texas (City) had entered into said contract for the construction of sanitary sewer improvements in Cleveland. Bayshore Engineers, Inc. (BEI) and C. Dieter Ufer

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[20] See *Barcelo v. Elliott*, 923 S.W.2d 575 (Tex.1996) (Professional duty of care to a testator or settlor to draft a will or trust does not extend to persons named as beneficiaries under the will or trust).
[21] See *Krishnan v. Sepulveda*, 916 S.W.2d 478 (Tex.1995) (Duty to provide competent medical care extends to patient and not husband).
[22] See *Hartman*, 946 S.W.2d at 550.
[23] See *Shatterproof Glass Corp*., 466 S.W.2d 873, 880 (Fort Worth 1971, writ ref'd n.r.e.)
[24] See *Barcelo* S.W.2d at 577.
[25] *I.O.I. Systems, Inc. v. City of Cleveland, Texas,* 615 S.W.2d 786, 790 (Tex.Civ.App.---Houston [1st Dist.] 1980, writ ref'd n.r.e.).

(Ufer) designed the plans for this project and served in a supervisory capacity, although they did not act as resident inspectors for the project. There was never any contract executed between BEI, Ufer and IOI. During construction, IOI encountered water in the trench where the pipes were to be laid. At this time, IOI realized there might be problems with this construction and requested Ufer and the City to remedy the problem. In order to correct the situation, additional fill material was needed for which neither City nor IOI would agree to pay. After pumping the visible water out of the trench, but without adding the fill material, IOI continued the project. Subsequently, six breaks resulted in this pipeline. IOI fixed these breaks, receiving approximately $4,000.00 from the City to pay for liners to repair the pipes. However, IOI alone spent an additional $91,000 on these repairs. The court held that in contracting for personal services, an architect's or engineer's duty depends on the *particular agreement entered into with his employer*.[26]

2.6 Engineer's Duty to Non-Clients under DTPA

The Houston [14th District] Court of Appeals considered an engineer's duty, under the DTPA, related to a "special relationship" in *Dagley v. Haag Engineering Co*.[27] In that case, State Farm Insurance Company hired Haag to perform certain engineering services on five homes with regard to the hail storm. Prior to the storm, Haag also had provided State Farm with materials regarding the evaluation of hail storm damage. Those materials generally state that hail stones less than one inch in diameter will not cause damage to composition shingle roofs. The Plaintiffs contended that based on Haag's estimates that the hail stones were 1/2" to 3/4" in diameter, State Farm's rejection of their claims was "preordained." They brought claims against Haag for negligence, conspiracy, tortious interference, and violations of the Texas Deceptive Trade Practices Act ("DTPA") and the Texas Insurance Code related to wrongful denial of their claims. The Court noted that none of Haag's alleged misrepresentations were directly communicated to the Plaintiffs. State Farm hired Haag to investigate certain hailstorm damage claims, and Haag submitted its evaluation materials, findings, and opinions to State Farm, not to the Plaintiffs. The Court held that, under these circumstances, *in the absence of a special*

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[26] Citing *Cobb v. Thomas,* 565 S.W.2d 281 (Tex.Civ.App.--- Tyler 1978, writ ref'd n.r.e.).
[27] See *Dagley v. Haag Engineering Co.*, 18 S.W.3d 787 (Tex.App.—Houston[14th Dist] 2000).

relationship, Haag cannot be liable under the DTPA for its alleged improper investigation of the Plaintiff's claims.

2.7 Engineer's Ethical Duties

Aside from legally recognized duties, an engineer must also be mindful of his ethical obligations under the Rules of Professional Conduct. While the engineer's violation of an ethical rule may, or may not, make one liable to a third party, the Texas Board of Professional Engineers certainly can impose sanctions when violations of their rules occur. Of importance, without limitation, are the following engineering ethical principles for considerations:

- Engineers Shall Protect the Public.[28]

  (a) Engineers shall be entrusted to protect the health, safety, property, and welfare of the public in the practice of their profession. The public as used in this section and other rules is defined as any individual(s), client(s), business or public entities, or any member of the general population whose normal course of life might reasonably include an interaction of any sort with the engineering work of the license holder.

  (b) Engineers shall not perform any engineering function which, when measured by generally accepted engineering standards or procedures, is reasonably likely to result in the endangerment of lives, health, safety, property, or welfare of the public. Any act or conduct which constitutes incompetence or gross negligence, or a criminal violation of law, constitutes misconduct and shall be censurable by the Board.

  (c) Engineers shall first notify involved parties of any engineering decisions or practices that might endanger the health, safety, property or welfare of the public. When, in an engineer's judgment, any risk to the public remains unresolved, that engineer shall report any fraud, gross negligence, incompetence, misconduct, unethical or illegal conduct to the Board or to proper civil or criminal authorities.

  (d) Engineers should strive to adequately examine the environmental impact of their actions and projects, including the prudent use and conservation of resources and energy, in order to make informed recommendations and decisions.
- Engineers' Actions Shall Be Competent[29]

  (a) Engineers shall practice only in their areas of competence, in a careful and diligent manner, and in conformance with standards,

[28] *Rules Concerning the Practice of Engineering and Professional Engineering Licensure* §137.55
[29] Id. at §137.59

laws, codes, and rules and regulations applicable to engineering practice.
(b) The engineer shall not perform any engineering assignment for which the engineer is not qualified by education or experience to perform adequately and competently. However, an engineer may accept an assignment which includes phases outside of the engineer's area of competence if those other phases are performed by legally qualified consultants, associates, or employees.

- Engineers' Responsibility to the Profession.[30] The engineer shall endeavor to meet all of the applicable professional practice requirements of federal, state and local statutes, codes, regulations, rules or ordinances in the performance of engineering services.

2.8 Miscellaneous Principles

Other principles worth noting, for purposes of liability management and related to the construction industry, are as follows:

- Existence of duty. In an action under negligence, the preliminary "threshold" issue is the duty of care, if any, that the Defendants owed to the Plaintiff.[31] The existence of a duty is a question of law for the court to determine from the facts surrounding the particular occurrence.[32]
- Privity of Contract. Courts have held, in construction contracts, a subcontractor is usually in privity only with the general contractor and cannot assert a claim against the owner.[33]
- Third- Party Beneficiary. A third party can recover on a contract only if (1) the contracting parties intended to secure a benefit to the third party; and, (2) the contracting parties entered into the contract directly for the third-party's benefit.[34] The agreement must clearly and fully express an intent to confer a direct benefit to the third party.[35]
- Standard of Care. Courts have held that professional negligence in the context of engineering services means doing that which an engineer of ordinary prudence in the exercise of ordinary care would not have done

[30] Id. At §137.63(b)(1)
[31] See *Greater Houston Transp. Co. v. Phillips*, 801 S.W.2d 523, 525 (Tex. 1990).
[32] See *Greater Houston* 801 S.W.2d at 525.
[33] See *City of LaPort v. Taylor*, 836 S.W.2d 829, 831 (Tex.App.—Houston [1st Dist] 1992, no writ).
[34] See *Stine v. Steward*, 80 S.W.3d 586, 589 (Tex. 2002).
[35] Id. at 589.

under the same or similar circumstances or failing to do that which an engineer of ordinary prudence in the exercise of ordinary care would have done under the same or similar circumstances. The standard of care must be established by the testimony of a qualified expert. To qualify as an expert able to set the standard of care for a given profession, the witness must be licensed in the same profession.[36]

[36] Citations omitted.

Section 3. Tools of Liability Management

3.1 Common Methods

As stated earlier, liability management is defined as the application of general liability principles, in conjunction with available tools, to minimize the potential economic impact of an incident. Now that we have discussed the theory of liability, and certain legal principles relating to the engineering, we turn to the tools of liability management. There are many tools used in the management of liability, including:

- Insurance contract
- Contract
- Peer Review
- Empty Shell
- Other tools, such as disclaimers, confidentiality policies, etc.

3.2 Insurance Contracts

Insurance contracts are very commonly used to manage liability. The terms, conditions, and protections offered by the policy vary widely. Premiums, however, can be expensive, and the deductible can be high. In addition, insurance attracts lawsuits. And, conversely, the lack of insurance discourages lawsuits. Plaintiff's lawyers, who are paid a percentage of any money recovery, will not invest their money in developing a case unless they think they will win money. In general business, common types of policies include:

- Errors and omissions, i.e., professional liability
- General comprehensive
- Automotive liability
- Specialty equipment coverage
- Workers' Compensation Insurance

In the context of foundation design, E&O insurance is intended to cover most "negligence" related incidents. Note, however, that an E&O policy may not cover contract actions, ethics violations, or intentional torts. As such, insurance should only be considered a portion of a liability management plan. When selecting coverage for your company, consider the following:

- The financial health of the carrier matters. Nothing is worse than paying premiums and then having the carrier default. Obtain ratings from various services to ensure the carrier is sound. Also note that Texas law may or may not regulate the carrier.
- Select the appropriate coverage for your business. A particular policy may exclude a significant portion of your business areas.
- Select the appropriate type. There are many different types of insurance polices, such as "term," "claims made," "claims made and reporting," etc. Understand the difference between the available policies and understand what you purchase.
- Determine whether the policy provides for the cost of defense. The attorney's fees for the defense of a case, however meritless the claims, can be significant. Policies may or may not include the cost of defense in the limits.
- Select the appropriate deductible. The selection of an appropriate deductible can be critical. A $25,000 deducible may not provide any coverage for a $20,000 claim. Consider selecting a deductible that correlates with your average fee per project.
- Expect to pay. Insurance is expensive and is usually a function of your projected revenue. The cost of insurance is generally incorporated into company pricing.
- Know the policy. Understand exclusions, or areas where the policy does not provide any coverage.
- Comply with the policy. Compliance with the policy is usually a condition precedent to coverage. Failure to comply may forfeit your rights under the policy. Failure to timely pay premiums, and/or failure to properly or timely notice claims, may waive possible coverage of a claim.

3.3 Contracts

Contractual protections can also be effectively implemented in a liability management plan. Such contractual protections, however, are usually under utilized. Consider the principles discussed earlier:

- Absent such a special relationship and concomitant duty, an independent adjusting firm is not be liable to plaintiff-insured for improper investigation and settlement advice, regardless of whether the Plaintiff-insured phrased his allegations as negligence, bad faith, breach of contract, tortious interference, or DTPA claims;
- Absent privity of contract, the engineer did not owe a third-party Plaintiff a duty of care in the preparation of the original plat of the subdivision, nor to correct of record any errors the plat contained [potentially bad principle];
- In contracting for personal services, an architect's or engineer's duty depends on the particular agreement; and,
- In the absence of a special relationship, an engineer cannot be liable under the DTPA for its alleged improper investigation of a third party's claims.

These principles clearly demonstrate the importance of a contract. Standard contracts are widely available from professional associations, such as the AIA. Fees for standard contracts are typically per use and can be less than $5.00. For most residential and light commercial construction projects, the contracts do not have to be complicated. Standard "terms and conditions," or TC's can be attached to, and reference in, proposals. When evaluating the use of a potential contract, consider the following:

- The contract must identify the client. Consider the situation where an architect instructs you to design a specific change to the construction drawings and charge the builder. The client also has implications as far as "duty," "privity," and "third-party beneficiary," as noted above. Does your contract identify the client and any third-party beneficiaries?

- The contract should clearly identify the scope of services. Each party to the contract may have different expectations, which can lead to problems. When the contract clearly defines scope, each party understands his or her responsibilities. Is your contract too vague?
- The contract should attempt to limit or cap damages. Consider the situation where an owner incurs damages due to a delay in completion caused by an error in your drawings. As noted above, damage to real property and improvements may consist of "foreseeable damages" which could arguably include lost profits. Does your contract limit damages?
- The contract should specify applicable design standards. Consider the situation where an architect fails to communicate design or loading changes, or a change in the building layout. Does your contract identify the applicable codes?
- The contract should be clear regarding jobsite safety. The contract should require Workers' Compensation insurance, as appropriate. Who is responsible when a contractor, or sub, sustains personal injuries? What about injuries to third-party bystanders?
- The contract should be clear on any construction inspection requirements. A properly designed, but improperly constructed, foundation can still be a problem. Who is responsible for inspection?
- The contract should be crystal clear on payment. Nothing is worse than performing a service and not getting paid. Does your contract include provisions to ensure timely payment?
- The contract should address mechanics and materialmen's liens. Texas law has specific requirements for liens, and the failure to comply invalidates any lien claims. Can your contract create a lien?

These suggestions are not intended to be all inclusive. Rather, this is merely a list to evaluate contractual protections.

3.4 Peer Review

Another method of minimizing liability is the use of a "peer review" process. Peer review is a method of quality control where a second engineer or company reviews the final work product for errors or omissions. A peer review is based on the principle that professional negligence means doing that which an engineer of ordinary prudence in the exercise of ordinary care would have done under the same or similar circumstances. Having two engineers independently verify work significantly reduces claims that the engineer did not exercise "ordinary care." Peer review within their office is highly effective and should be incorporated into every project. On complex projects, the engineer should suggest and encourage third-party peer reviews. When implementing a peer review process in a liability management plan, consider the following:

- The scope of any review should be clearly established so that both parties understand exactly what each other is doing.
- The process should be established for its particular purpose. What types of errors are you trying to minimize, i.e., standard details, general notes, etc.?
- The process may require written procedures. Note, however, those written procedures are subject to discovery after an incident occurs. If you have procedures, you better well comply with your own procedures, or that fact will be used against you.
- Respective liability should be clearly apportioned. The peer review may contractually accept some or none of any resulting liability.

3.5 Empty Shell

An empty shell is an entity formed under law that owns no assets. Contractors typically use empty shells, and this technique can be effective. When an incident occurs, the contractor simply goes out of business. Proper application of an empty shell depends entirely on the specific circumstances.

When utilizing an empty shell, make sure you comply with all formalities of the applicable law. Failure to comply with formalities may subject investors to personal liability, which effectively defeats this method.

For engineers, the interaction of the licensing laws makes the empty shell difficult to apply. If an action is based on negligence, the plaintiff may name the individual personally who sealed the drawings in an effort to force his employer to pay damages. Nonetheless, this method can be effectively used to limited potential liability.

Section 4. Application of Liability Management

Now that we have discussed the theory and certain principles of liability, and we have discussed commonly available tools, we turn to examples of the application of liability management. The discussion and analysis of these examples are for illustrative purposes only, as the actual analysis may widely vary between applications.

Example 1: An *architect* retains an *engineer* to design a foundation for a residence for a *builder*. After the house is complete, it is sold to a *buyer*. Months after the sale, the foundation moves excessively interfering with normal use.

Potential parties: architect, engineer, builder, and buyer.

Major Causes of action: the *buyer* may have a cause of action against the *builder* for breach of contract (among others); the *builder* may have a cause of action for breach of contract against the *architect*; the *architect* may have a cause of action for breach of contract and negligence against the *engineer*. (As for the *buyer* and the *builder,* they lack privity of contract with the *engineer.* Yet, the *buyer* may attempt a DTPA and negligence claims against the *engineer*.)

Potential Damages: Costs to repair plus damages for loss of use; potential valuation damage (market value of home in the intended condition); and, attorney's fees under theory of breach of contract.

Guiding Principles: The *engineer's* duty to non-clients is limited. However, if the *engineer* was actually negligent, he would be liable to his client, the *architect*.

Possible Liability Management Plan: Peer review to minimize design mistakes; written contract to govern architect's claims, to cap damages, and to limit the third-party beneficiary (the builders and the buyers) claims; E&O insurance to protect against negligence claims from client and from third parties.

Alternative Liability Management Plan: Empty shell.

Example 2: A *property owner* retains an *engineer* to design a foundation for a structure that was designed by an *architect*. Months after completion, the foundation moves excessively interfering with normal use.

Potential parties: property owner, engineer, architect.

Major Causes of action: the *property owner* may have a cause of action for breach of contract and negligence against both the *engineer* and the *architect* (the property owner retained them both).

Potential Damages: Costs to repair plus damages for loss of use; potential valuation damage (market value of home in the intended condition); and, attorney's fees under theory of breach of contract.

Guiding Principles: The *engineer's* duty to clients is doing what an engineer of ordinary prudence in the exercise of ordinary care would have done under the same or similar circumstances; and the engineer's duty is governed by the contract.

Possible Liability Management Plan: Peer review to minimize design mistakes; written contract to govern resolution of the claims and to cap damages; E&O insurance to protect against negligence claims.

Example 3: An *architect* retains a *geotechnical engineer* to perform soil testing and provide recommendations that a *structural engineer* uses to design a foundation. (considering only the geotechnical engineer's position related to the structural engineer)

Potential parties: architect, structural engineer.

Major Causes of action: the *architect* may have a cause of action for breach of contract and negligence against the *geotechnical engineer* and the *structural engineer*. The *structural engineer* may have a claim for negligent misrepresentation against the *geotechnical engineer*.

Potential Damages: actual damages based on negligent misrepresentation claim (which would consist of the costs to repair plus damages for loss of use, potential valuation damage) (no attorney fees for negligence claims).

Guiding Principles: In the absence of a special relationship, an engineer cannot be liable under the DTPA for its alleged "misrepresentations" to a third-party when the communications are not directed to that third-party. Note, however, this principle as stated earlier is highly questionable and may not provide any legal protection.

Possible Liability Management Plan: Peer review to minimize design mistakes; written contract to govern architect's claims and to cap damages; E&O insurance to protect against negligence claims from engineer; application of engineering ethics to protect confidentiality and limit third-party claims.

Liability management is defined as the application of general liability principles, in conjunction with available tools, to minimize the potential economic impact of an incident. We discussed the theory of liability, we examined certain legal principles relating to the engineers, and we examined tools used to minimize the economic impact. As with clothes, rarely does "one size fit all" when managing potential liability. Each company must identify its own business areas and its guiding principles. Using appropriate insurance, standard contracts, and peer reviews as tools can effectively manage the potential liability. As illustrated in the examples, a blended approach usually provides the best protection at an economic cost when things go wrong.

Liability Management

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Liability Management

The application of general liability principles, in conjunction with available tools, to minimize the potential economic impact of an incident.

Definitions
Potential Parties
Causes of Action
Damages
Guiding Principles
Tools
Examples

Definitions

Liability*: The state of being bound or obligated by law or justice to do, to pay, or to make something; the state of one who is bound by law and justice to do something which may be enforced by action.

Incident: An incident, in any situation, involves the specific facts and circumstances that occur.

*Black's Law Dictionary, sixth edition, citing *Fidelity Coal Co. v. Diamond*, 310 Ill.App. 387, 34 N.E.2d 123; *Clark v. Lowden*, D.C.Minn., 48 F.Supp. 261, 263.

Potential Parties

"Clients" "Non Clients"

architect
contractor
subcontractors
other engineers
developers
current owner
future owners
leaseholder

Causes of Action

VALID

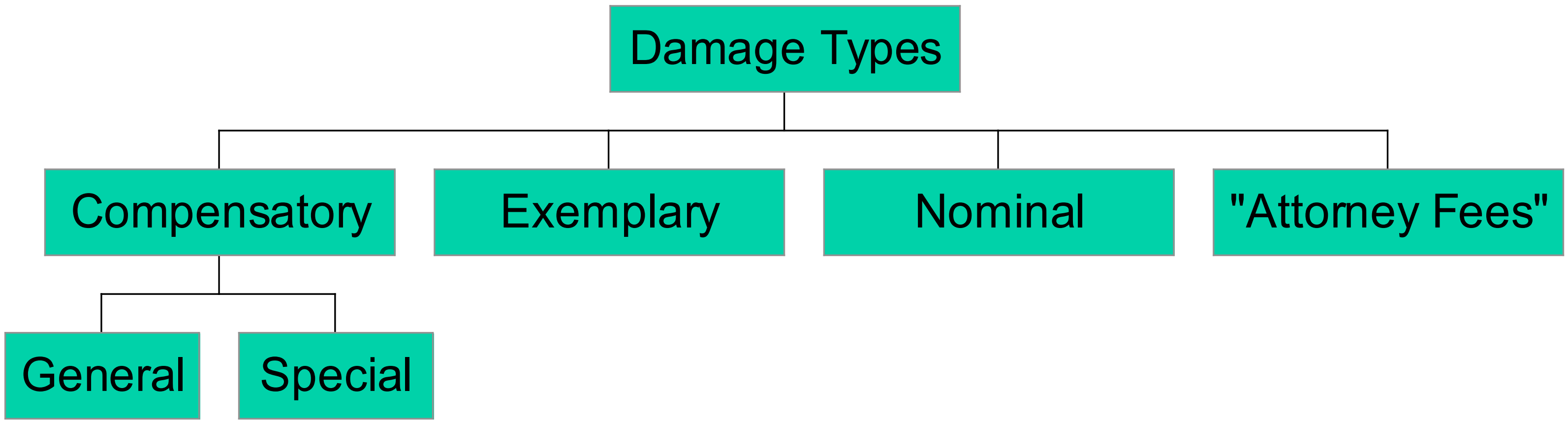
- Breach of Contract
- DTPA (Consumer Statute)
- Negligence
- Negligent Misrepresentation
- Fraud
- Breach of Warranty

INVALID

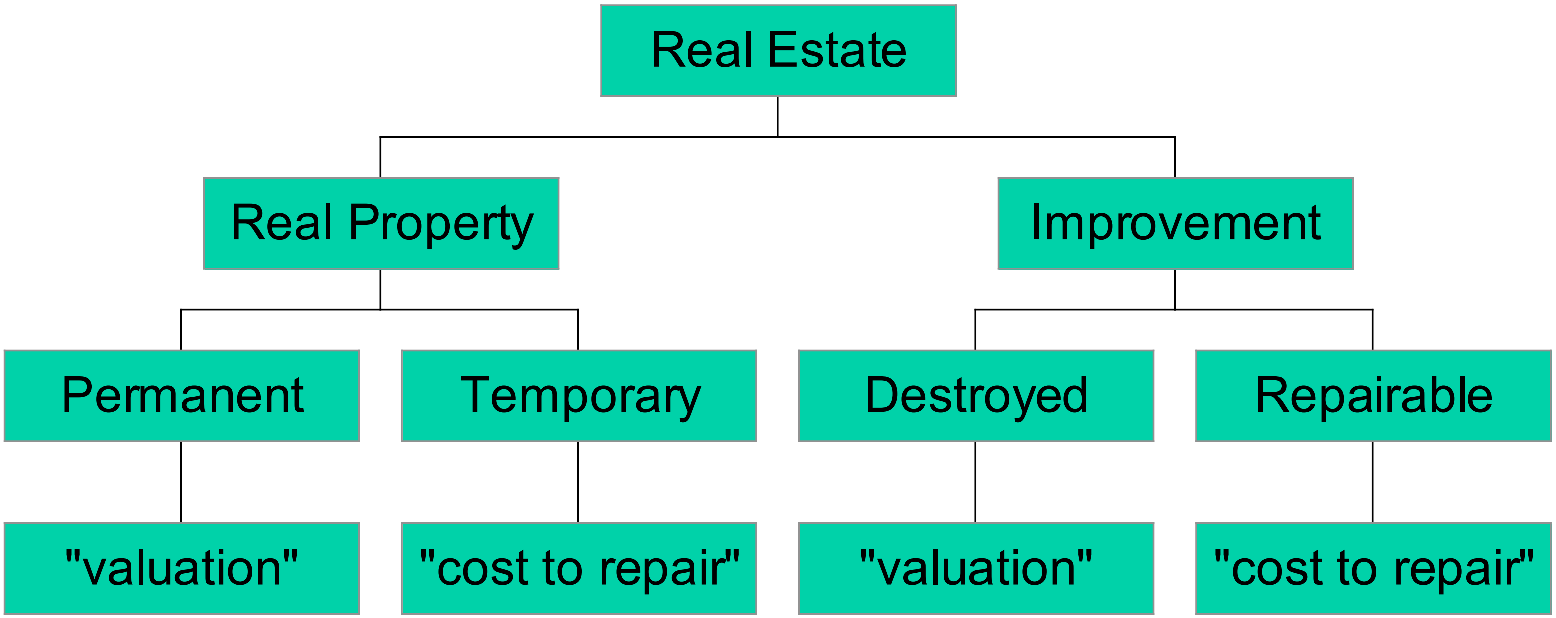
- Negligent Infliction of Emotional Distress
- Breach of Implied Warranty of Professional Services

DTD Comments: Some causes of action can be assigned to third-parties!

Damages


Damage Types
Compensatory
Exemplary
Nominal
"Attorney Fees"
General
Special

Damages: Real Property



Damage: Mental Anguish

- Past and future
- Available in personal injury case
- Must be physically injured; unless:
  - Intentional/malicious conduct
  - Breach of Duty arising from special relationship
  - "Disturbing events"

- Courts have held that plaintiff whose real property has been damaged by defendant's negligence cannot recover mental anguish damages.

Guiding Principles: Caution

While case law is authority, court decisions and opinions can vary significantly!

Appeals cost money

JURISDICTION • JURISDICTION • JURISDICTION

Guiding Principles: Architect's Duty to Clients

In the absence of special agreement, *an architect is not liable for faults in construction resulting from defects in the plans, as his undertaking does not imply or guarantee a perfect plan or a satisfactory result*, it being considered enough that the architect himself is not the cause of any failure, and there is no implied promise that miscalculations may not occur. Thus, *an architect is only liable for a failure to exercise reasonable care and skill* [emphasis added].

DTD Comments: Dangerous! Expert testimony may have changed result.

Ryan v. Morgan Spear Associates, Inc., 546 S.W.2d 678 (Tex.Civ.App.---Corpus Christi 1977, writ ref'd n.r.e).

Guiding Principles: Professional's Duty to Non-Clients

The Court held that the independent adjusting firm hired exclusively by the insurance carrier, *had no relationship with, and therefore owed no duty to*, the Plaintiff-insured. The Court went on to state: Absent such a relationship and concomitant duty, the independent adjusting firm could not be liable to the Plaintiff for improper investigation and settlement advice, regardless of whether the Plaintiff-insured phrased his allegations as negligence, bad faith, breach of contract, tortious interference, or DTPA claims *[emphasis added]*.

DTD Comments: Dangerous!

Dear v. Scottsdale Ins. Co., 947 S.W.2d 908, 916 (Tex.App.—Dallas 1997).

Guiding Principles: Engineer's Duty to Non-Clients

The Court held that no privity existed between the engineer and the Plaintiff. Thus, the Court held that the engineer did not owe the Plaintiff a duty of care in the preparation of the original plat of the subdivision, nor to correct of record any errors the plat contained.

DTD Comments: Very Dangerous! Could have alleged negligent misrepresentation.

Hartman v. Urban, 946 S.W.2d 546 (Tex.App.—Corpus Christi 1997).

Guiding Principles: Engineer's Duty to Clients

The court held that in contracting for personal services, an architect's or engineer's duty depends on the *particular agreement entered into with his employer*.

DTD Comments: Important

Cobb v. Thomas, 565 S.W.2d 281 (Tex.Civ.App.--- Tyler 1978, writ ref'd n.r.e.).

Guiding Principles: Engineer Ethics

Engineers Shall Protect the Public
Engineers' Actions Shall Be Competent
Engineers' Responsibility to the Profession

DTD COMMENTS: Not necessary a separate cause of action; BUT: discipline action could be evidence of negligence; complaints can be problematic.

Tools of Liability Management

Insurance contracts

Contracts with clients

Peer review

Others

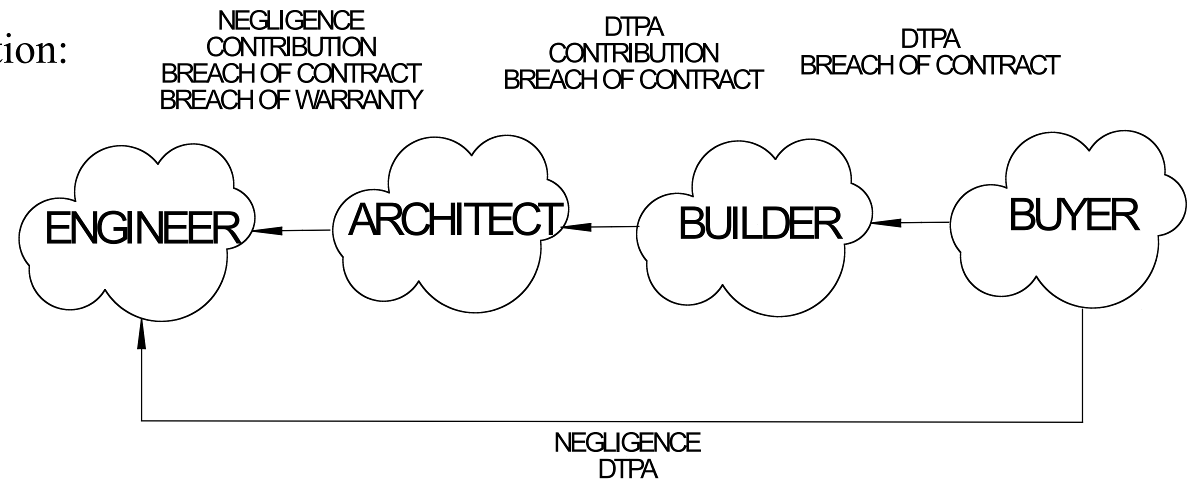
Example #1: Situation

An *architect* retains an *engineer* to design a foundation for a residence for a *builder*. After the house is complete, it is sold to a *buyer*. Months after the sale, the foundation moves excessively interfering with normal use.

Example #1: Considerations

Potential parties: architect, engineer, builder, and buyer.

Causes of Action:



Potential Damages

COST TO REPAIR + LOSS OF USE

OR

VALUATION

\+ ATTORNEY'S FEES

Guiding Principles: The *engineer's* duty to non-clients is limited. However, if the *engineer* was actually negligent, he would be liable to his client, the *architect*.

Example #1: Possible Plan

- Peer review to minimize design mistakes
- Written contract (1) to govern architect's claims, (2) to cap damages, and (3) to limit the third-party beneficiary (the builders and the buyers) claims
- E&O insurance to protect against negligence claims from client and from third parties.

DTD Comments: Specifics depend on typical project size, etc.

Example #2: Situation

An *architect* retains a *geotechnical engineer* to perform soil testing and provide recommendations that a *structural engineer* uses to design a foundation.

(considering only the geotechnical engineer’s position related to the structural engineer)

Liability Management

The application of general liability principles, in conjunction with available tools, to minimize the potential economic impact of an incident.

Potential Parties
Causes of Action
Damages
Guiding Principles
Tools

Questions

Liability Management in Foundation Engineering, Dorr, David T., September 23, 2005.

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POST FOUNDATION REPAIR PERFORMANCE

OF

RESIDENTIAL AND OTHER LOW-RISE BUILDINGS

ON EXPANSIVE SOILS

by

The Repair Committee

of

The Foundation Performance Association

www.foundationperformance.org

Houston, Texas

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Rev #	Date	Description	Subcommittee Chair	Subcommittee Members
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B	5 Nov 02	For Committee Comments		
C	26 Apr 03	For FPA Peer Review		
D	9 Oct 03	For Committee Comments		
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PREFACE

This document has been developed by a group of foundation repair contractors and consultants in southeast Texas with the goal to educate and inform those involved in the repair of foundations.

The need for this document was prompted by the lack of satisfactory performance of some foundation repairs. As a result, this document has been prepared and made freely available to the public through the Foundation Performance Association at www.foundationperformance.org so that owners, tenants, realtors, builders, inspectors, engineers, architects, repair contractors, and others involved with residential and other low-rise building foundations may benefit from the information it contains.

This document was written specifically for use in the southeast region of the state of Texas and primarily within the City of Houston and the surrounding metropolitan area. Therefore, it should be used with caution if used elsewhere, or if adapted for foundations other than those supporting residential or low-rise structures. The Foundation Performance Association and its members make no warranty regarding the information contained herein and will not be liable for any damages, including consequential damages, resulting from the use of this document.

TABLE OF CONTENTS

1.0 INTRODUCTION

What is foundation repair?

Many methods are available to stabilize foundations from the effects of expansive soils, some which work better than others. The process of foundation repair usually includes foundation stabilization and the implementation of one of these methods. This may involve modification of the foundation support system itself, under pinning with deeper support, the construction of peripheral barriers that function to isolate the soils under the structure, or the installation of some type of system that attempts to control the moisture content of the soils or the molecular action of the clay when the moisture content is changed.

In addition to stabilizing a foundation from future movements, foundation repair may also include the process of restoring a foundation to its original constructed position by using some type of leveling procedure. This usually involves jacking the concrete grade beams at the lower portions of the foundation upward until they are level with the other areas, and then re-supporting the grade beams at the areas that have been raised. In limited circumstances, lowering high areas may be possible by supporting the foundation in the high area, excavating a void, and then lowering the foundation with the installed support. In some cases, it may be desirable to level and then raise the whole ground floor even higher than the original constructed position. The leveling procedure may also include the injection of grout or foam under the low areas of the slab-on-grade.

Generally, in the southeast Texas area, damage to residential and low-rise buildings as a result of foundation movements, is most often due to changes in moisture content of expansive soils. Damage may occur to components of the superstructure, the foundation, or both. In some cases, foundation repair can simply consist of only repairing damage that occurred to the foundation elements.

Why does foundation repair sometimes fail?

Aside from the possibility of a manufacturing defect, the success or failure of the product installed can be governed by the experience of the inspector, engineer, or contractor. A lack of good quality control by the contractor or improper foundation maintenance by the property owner can adversely affect the longevity of the foundation repair. These and many other things can go wrong when repairing a foundation.

Piers or pilings will not prevent foundation uplift unless they are designed to do so. The type of repair piers or pilings that are most commonly used for residential and low-rise building repairs in the Houston area are not tied to the grade beams, and thus cannot prevent foundation uplift due to heaving soils.

Heave or uplift is not generally considered to be a failure of the piers or pilings, but if it occurs, the piers or pilings could be perceived to have failed since they are part of the overall foundation system. If heaving is a concern, the entire ground floor system should be lifted above the maximum potential vertical rise of the soils. Leaving a void between the soils and the slab allows space, which can accommodate the heaving soil without causing the foundation to heave. In this case, the piers or pilings must have sufficient depth and tensile resistance to resist the uplift forces resulting from friction along the pile or pier surface from the upward moving soil.

Foundation uplift is an example of post-foundation-repair performance that may be incorrectly attributed to a defect in a particular foundation repair method. Included in this document is a partial list of systems used to help repair foundations and the ways they can fall short of acceptable performance. These lists are provided in order to assist building owners, and others who may not be familiar with foundation repair techniques, in identifying the cause of poor post-foundation-repair performance.

2.0 FOUNDATION REPAIR SYSTEMS

Below is a description of some foundation repair systems and typical problems that cause poor performance. Please note that in order for most of these systems to perform properly, the moisture content of the soils under and around the perimeter of the foundation must be maintained a constant, and stormwater drainage must be directed away from the foundation. Improper drainage can cause continued movement to the repaired slab because of the resulting fluctuations in moisture content of the expansive soils that may occur. Continuously saturated soil is detrimental to the foundation and the repair piers. In some instances, void boxes under the slab or grade beams can fill with water due to improper drainage and are believed to be the cause of differential movement.

2.1 Mud Jacking

Mud jacking is a procedure whereby a foundation is lifted by pressure injecting a slurry composed of sand, fly ash, or topsoil and cement between the slab and the soil. Some reasons why this type of repair might fail are:

A. If the soil moisture content changes, the slab can move.
B. This is a shallow repair system effected by active moving soil
C. Moisture changes from broken sewer or water pipes under the slab
D. Trees allowed to grow within close proximity of the slab
E. Extreme seasonal moisture changes
F. Poor drainage
G. Mud can become entrapped by tree roots, pipes, or unknown objects and cause an uneven lift
H. Failure to control the amount of mud injected can cause upward bulging or cracking of the concrete slab
I. Excessively high pressures from injected mud can cause damage due to blowouts
J. If underground utility lines have holes or cracks, the grout may possibly enter the holes and fully or partially plug the lines

2.2 Foam Injection

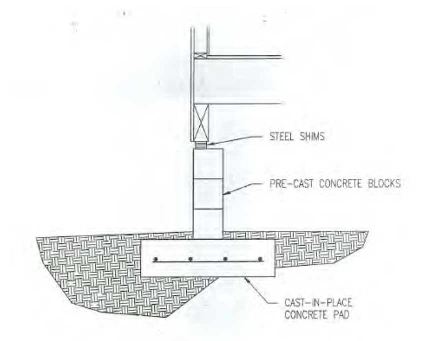
Generally foam injection is used to lift low areas of a slab by injecting a urethane based foam material under the slab area. The foam expands creating pressure to raise the slab. After the injections, the lifted area of the slab will sit on the foam.

A. If the soil moisture content changes, the slab can move
B. This is a shallow repair system effected by active moving soil
C. Moisture changes from broken sewer or water pipes under the slab
D. Trees allowed to grow within close proximity of the slab
E. Poor drainage
F. Extreme seasonal moisture changes
G. Improper mixture of chemicals used to create expanding foam
H. Failure to control the amount of foam injected can cause upward bulging or cracking of the concrete slab
I. Prior to hardening, excessive water from an overly wet foam mix can seep into expansive soils, increase the moisture content, and cause local heave and out-of-levelness
J. Excessively high pressures from injected foam can cause damage due to blowouts
K. If underground utility lines have holes or cracks, the foam may possibly enter the holes and fully or partially plug the lines, particularly if the foam is pressurized

2.3 Spread Footings / Block and Base

Spread footings are most often wide, cast in place concrete pads, placed under a footing to spread the load and reduce pressure on the soil. On expansive soils, the bottom of a spread footing may be several feet below the surface. Block and base structures will typically have the floor held above the ground on wood or concrete supports placed on a wider base of precast concrete. There is usually an air space to allow ventilation for the wood components and access for maintenance.

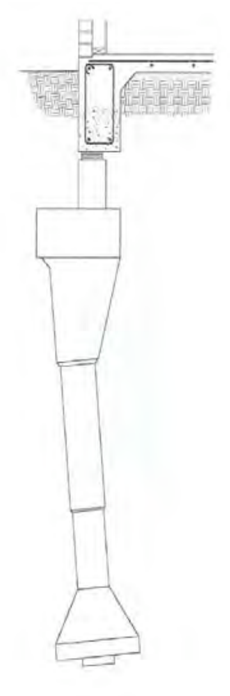
A. Foundation sagging may occur from blocks placed too far apart under the wood beams
B. Improper drainage or ponding of water under the house can allow the blocks to move, settle, or tilt
C. Incorrect material can be overloaded, overstressed, and break allowing sag of the supporting beam
D. Seasonal moisture changes
E. Improper depth and placement of base, supporting blocks, and shims
F. Improper size of base blocks



2.4 Drilled Piers

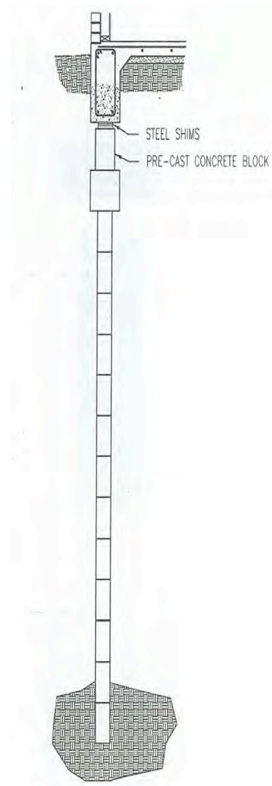
Drilled piers are a cast in place foundation support system. This system can be a drilled straight shaft, or a drilled straight shaft with an under ream at the base commonly referred to as a bell. A hole is excavated in the soil. Steel reinforcement is placed in the shaft. Redimix is placed in the hole and vibrated. The main support on this system comes from the bottom of the pier with some additional support obtained through skin friction on the sides of the shaft. The concept is to achieve support below the active zone of the soil to minimize future movement.

A. Improper depth of pier may allow pier to move
B. The soil condition is not conducive to construction of drilled piers
C. If required, improper bell at bottom of pier
D. Improper drainage may cause piers to move
E. Too much load on pier
F. The soil packs or shears at bottom of the pier
G. The cap or head of the pier is not thick enough and can allow the head to crack
H. Improper reinforcing steel placement may allow the pier to crack
I. Improperly shimmed
J. Seasonal moisture changes on shallow drilled piers may allow the pier to settle
K. Shaft installed at too steep of an angle can allow the pier to lay over or become overstressed when weight is applied
L. Failure to vibrate or low slump concrete may allow the concrete to honeycomb
M. Collapsing of the soils within the bell or sloughing of the soils along the shaft can result in the accumulation of loose fallen materials at the base of the pier, which could lead to settlement of the pier when these loose materials are compacted under the applied bearing pressures.
N. Any gaps that occur around the shaft of the pier (e.g. from drying action of the soil) can later serve as a free path for infiltrating water to reach the lower inactive zone of the soil and cause detrimental heaving of the pier
O. Sometimes the pier is installed properly, but blocks, pads, cylinders, or a rock on the beam bottom may shatter, allowing the beam to resettle.



2.5 Pressed Piles

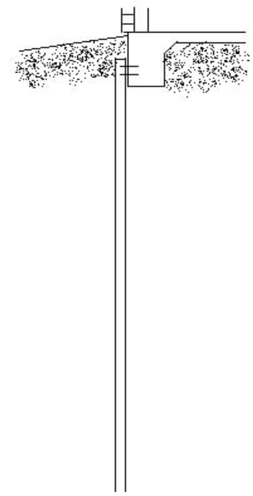
Pressed piles are precast concrete sections pressed into the ground using the weight of the structure being supported as ballast for resistance. Concrete sections are stacked one on top of the other and pressed into the ground until the pile stops going down and the structure begins rising. The main support in this system is the skin friction generated up the side of the pile as it is pressed into un-dug soil. Some support is also gained by the soil compressed under the base of the pile. The concept of this system is to achieve support below the active zone of the soil to minimize future movement.



A. The structural problems are not conducive to pressed pile, for example, insufficient steel in the beam/slab or not enough weight to allow driving pile to the proper depth
B. Pile caps or cylinders can break during installation. If not corrected further settling may occur
C. If the contractor did not shim immediately after driving pilings, the piling will spring back if bulb of pressure at the base of the pile is not maintained
D. Shallow piling not driven sufficiently below the moisture active zone can continue to settle as seasonal moisture changes occur
E. Drought can cause skin friction to release at shallow depths
F. Sometimes the piling will hold, but blocks, pads, cylinders, or a rock on the beam bottom may shatter, allowing the beam to settle
G. Improper moisture close to the pile from poor drainage or from leaking sewer or water lines can cause the pier to move
H. Water jetting may be used to install the piles through stiff sandy soils, which can locally introduce large amounts of water, increase the moisture content of expansive soils, and cause heave of the pile
I. The cylindrical segments may not be driven vertically due to misalignment when starting the first cylinder, or by being deflected laterally by tree roots, rocks, or calcareous nodules in the clay soils, which in turn could lead to a reduction in the vertical load capacity of the pile, or stress concentrations due to uneven load bearing that could result in crushing the concrete
J. Unreinforced pressed piles that do not have a central steel bar or cable running along the center of the pile are subject to potential differential lateral movements between the upper most concrete segments

2.6 Steel Pipe Piles

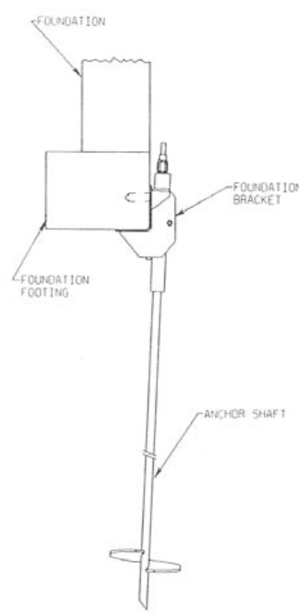
Steel pipe piles are pressed in the ground using the weight of the structure to be supported as resistance. Steel sections are pressed into the ground one on top of the other until the needed resistance is met or onto rock or other load capable strata. The main support with this system is skin friction generated up the side of the pipe sections unless pressed down onto rock or other load capable strata. This system attempts to gain support for the foundation down past the active moisture zone of the soil to minimize future movement.



A. If not pushed to refusal, the shallow pipe pile may move as seasonal moisture changes occur
B. If not pushed at the correct angle, the pipe pile may lay over or bend when the full load is applied
C. The pipe pile can be overloaded, causing the shaft to bend
D. Improper moisture close to the pile such as poor drainage or a leaking sewer or water line can cause the pier to move
E. The structural problems are not conducive to steel pipe pile - for example, insufficient steel in the beam/slab or not enough weight to allow driving pile to the proper depth
F. If the contractor did not clamp bracket immediately after driving pilings, the piling will spring back and loose some bearing pressure.
G. Drought can cause skin friction to release at shallow depths allowing the pipe pile to move

2.7 Helical Piers

The helical pier foundation system is an end-bearing anchor screwed into the soil by hydraulic drive equipment. Helical piers are steel pipe or solid square stock with helix plates attached at intervals. As the shaft is rotated, it screws into the soil. Anchors are driven to the required torque to obtain the needed capacity. Once installed, the bracket is used to transfer the weight of the structure from the foundation to the pier permanently.



A. Improper spacing may cause overload of the pier.
B. Piers not installed at the correct angle may allow the shaft to lay over when weight is applied.
C. Not installing the pier to the proper torque and depth below the active zone may allow the pier to move with seasonal moisture changes
D. Improper anchorage of the steel brackets that are used to support the foundation grade beams to the top of the helical piers can result in connection slippage and settlement of the grade beam

3.0 MAINTENANCE SYSTEMS

Maintenance systems should not be considered a repair; although they will enhance the performance of the foundation. Below is a description of some foundation maintenance systems and typical problems that may cause poor performance.

3.1 Root Barriers

Root barriers are designed to stop the growth and moisture draw of roots under and in the proximity of the foundation. A trench is dug in the intended area and a barrier of concrete, polyethylene, biocide treated mesh, or other material is placed to block the future growth of roots. Most barriers are 48" or less.

A. Soil expansion and contraction will occur with seasonal weather changes. The mesh root barrier will allow moisture to penetrate. The impervious barriers will not allow moisture to penetrate.
B. Caution should be utilized when water leaks occur between the impervious root barrier and the slab. The source of moisture may be from the plumbing lines, sprinkler systems, watering systems, downspouts, or poor drainage. The entrapped moisture can allow soil expansion and contraction to occur, allowing nearby foundations to move.
C. The root barrier may not be deep enough or may have soil above it thereby allowing roots to grow below or above it.
D. Property owner should remove wayward roots if they grow past the barrier
E. Root barriers that use chemicals to prevent root growth have a limited life and will eventually become ineffective after ten years.
F. Root barriers that consist of thin membranes such as polyethylene sheeting can be punctured during installation, allowing small root fibers to penetrate that can eventually grow into large roots
G. The root barrier installation may sever existing tree roots that already extend under the foundation, and as they wither and die, the surrounding soil could slowly regain its original moisture content, possibly causing local heave of the soil near the severed roots.
H. If the existing tree roots that already extend under the foundation are large in diameter, the decay of the severed roots caused by the root barrier installation could potentially lead to local settlement of the foundation.

3.2 Moisture Barriers

Moisture barriers can be either vertical or horizontal. Both types are designed to minimize changes in moisture content under the foundation. There are many different materials used in the construction of moisture barriers. A vertical moisture barrier is installed by excavating into the soil near the foundation and installing a waterproof barrier. The depth of the vertical moisture barrier would be determined by the soil type and site condition. A horizontal barrier is a waterproof barrier, which extends horizontally from the foundation and may be above or below grade, as site conditions require.

A. A water leak occurring between the barrier and the foundation beam may cause the slab to move The source of water may be plumbing lines, sprinkler systems, watering system downspouts, or poor drainage.
B. Property owner should visually monitor moisture or ponding water adjacent to the foundation.
C. The depth of a vertical moisture barrier may not be adequate, i.e., at or below the depth to constant suction.
D. The width of a horizontal moisture barrier may not be adequate
E. The barrier may not be properly sealed to prevent moisture penetration
F. The barrier may not have been properly designed
G. Thin membranes used for horizontal or vertical moisture barriers may be punctured or torn during installation and subsequently allow the transmission of water.

H. Horizontal moisture barriers such as polyethylene sheeting may be subject to degradation from ultraviolet radiation from sunlight if left uncovered.
I. Thin membrane vertical moisture barriers may be punctured by the growth of tree roots.
J. Horizontal moisture barrier systems can sometimes result in increasing the amount of moisture in the soil below the barrier through the process known as hydrogenesis (i.e. cooling of the ground during the night that causes water condensation of moisture-laden air in the soil on the underside of the barrier due to the temperature change), which in turn can cause heave.
K. Horizontal moisture barriers consisting of concrete pavement may be compromised by water that infiltrates through joints that are not properly sealed.
L. Horizontal moisture barriers may eventually "walk away" from the foundation as a result of cyclic movements due to alternate periods of shrinking and swelling of the supporting soils.
M. The installation of vertical moisture barriers can also cause severing of existing tree roots, which can lead to the same problems as described above for root barriers.
N. A vertical moisture barrier, which is normally designed to keep water from penetrating under the building, can also prevent water from migrating outward from underneath the building, which can amplify heaving action due to a leaking water line underneath the building.

3.3 Perimeter Watering Systems

Perimeter watering systems are used, particularly in expansive soils, to hydrate soils around a foundation to avoid drying and shrinking of the soils. Most systems emit a small amount of water on a regular basis to keep the moisture content even in the soils through nature's cyclic weather changes. Systems range from very simple manually operated, to highly complex automated systems.

A. Electric sensors may malfunction and cease to operate.
B. The system may allow water to run continuously, or not at all.
C. Over watering can cause adverse movement.
D. The soil type may not be conducive to a watering system.
E. Watering around the foundation tends to attract tree roots near the foundation so that cyclical soil movement is exasperated during periods of high and low moisture.
F. Lack of maintenance by the homeowner can make the system ineffective.
G. For automated systems, loss of power due to severed wires can cause failure of the system.
H. Dramatic foundation settlements could occur if the system is disabled.

FOUNDATION DESIGN OPTIONS

FOR

RESIDENTIAL AND OTHER LOW-RISE BUILDINGS

ON

EXPANSIVE SOILS

by
The Structural Committee

of
The Foundation Performance Association

www.foundationperformance.org

Houston, Texas

Document # FPA-SC-01-0

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0	30 Jun 04	For Website Publishing		Mari Mes
				George Cunningham
				John Clark
				Toshi Nobe Nicole Wylie Denis Hanys Jim Austin

PREFACE

This document was written by the Structural Committee and has been peer reviewed by the Foundation Performance Association (FPA). This document is published as FPA-SC-01 Revision 0 and is made freely available to the public at www.foundationperformance.org so all may have access to the information. To ensure this document remains as current as possible, it may be periodically updated under the same document number but with higher revision numbers such at 1, 2, etc.

The Structural Committee is a permanent committee of the Foundation Performance Association. Suggestions for improvement of this document should be directed to the current chair of the Structural Committee. If sufficient comments are received to warrant a revision, the committee will form a new subcommittee to revise this document. If the revised document successfully passes FPA peer review, it will be published on the FPA website and the previous revision will be deleted.

The intended audiences for the use of this document are engineers, builders, architects, landscape architects, owners, and others that may be involved in the design or purchase of foundations and building sites that are located in the southeast region of the state of Texas, and primarily within the City of Houston and the surrounding metropolitan area. However, many of the advantages and disadvantages discussed for each of the foundation and site design options may also apply to other geographical areas with expansive soils. Geographical areas that potentially have foundation problems similar to those in the Houston area may be identified as those having large surface deposits of expansive clays with a climate characterized by alternating wet and dry periods.

TABLE OF CONTENTS

1.0 INTRODUCTION

The scope of this document is to provide guidance in the selection of design options for residential and other low-rise building foundations, typically called light foundations, which are founded on expansive soils. Low-rise buildings are defined as one to four stories in height. These buildings include houses, garages, apartment and condominium buildings, restaurants, schools, churches, and other similar structures. Design options for foundation systems, foundation components, and moisture and vegetation control methods are reviewed and compared. There are no absolute design rules for choosing a design. This document provides a list of advantages and disadvantages for each of the many commonly used foundation design options to assist the designer in selecting the most suitable option.

A brief overview of the design problems associated with expansive soils is provided in Section 2.0, and general design considerations are presented in Section 3.0. The foundation design options are categorized into three separate sections. Section 4.0 covers foundation system design options considering the structural foundation system as a whole. The foundation systems are subdivided into two groups: deep support systems and shallow support systems. Section 5.0 addresses design options for various individual structural components of the foundation systems that are discussed in Section 4.0. Section 6.0 discusses site design options for moisture and vegetation control systems.

Foundation design options for heavily loaded structures such as mid- to high-rise buildings or large industrial structures that usually require deep foundations or thick large mat foundations are not addressed, nor are design options for lightly loaded structures that are not susceptible to significant damage due to differential vertical movements from soil moisture changes, such as relatively flexible light gage metal buildings with exterior metal siding and roofing and wide open interior spaces with no interior partition walls.

2.0 PROBLEM DEFINITION

The challenge with designing building foundations on moderate to highly expansive clay soils is the potential detrimental effects of differential movements of the foundation structural elements due to volumetric changes of the underlying and surrounding soils. In simple terms, expansive clay soils swell and can cause heave with increasing soil moisture, or can dry out and cause subsidence with decreasing soil moisture.

Movement of expansive soils is caused by fluctuations in the moisture content of soil particles. Because homogeneous expansive clay soils have very low permeability, fluctuations in the moisture content of the soils might normally be expected to occur over a very long period. However, permeability is increased with geotechnical phenomena such as ground faults, surface fractures due to desiccation of clays, and decomposition of tree roots which cause fissures and cracks that become widely disseminated over time.

Due to the repeated wetting, swelling, drying, and shrinking of the clay as it weathers, the fissures often fill with silt and sand, and create pathways for water that can exacerbate the infiltration process. Water can also easily move through naturally occurring sand strata, sand seams, and micro-cracks in clay soil caused by previous shrinkage. High negative pressures, also known as suction, in clay soils with low water content also increase the tendency for water to be absorbed into the clay.

Environmental factors other than climatic conditions can also affect expansive soils. Water extraction by trees and other vegetation, a process known as transpiration, can cause soil shrinkage. Swelling can be a result of water infiltration into the soil from lawn irrigation systems, broken water pipes, flooded and leaking utility trenches, poor drainage, or leaking swimming pools, or it can be a result of slow moisture replenishment and equalization after the removal of a tree. The combined effect and variability of all of these possibilities make it difficult to accurately predict expansive soil ground movements.

Foundation movements are considered problematic only if they result in negative phenomena that detrimentally affect the performance or appearance of the building. The negative phenomena are considered to be structural if the load carrying capacity of the superstructure or foundation elements are affected, or are considered to be cosmetic if only the appearance of the exterior cladding or interior wall, floor, or ceiling finishes are affected. Negative phenomena can also affect the serviceability the building, such as the opening or closing of doors.

Negative phenomena due to foundation movement typically occur because of differential movements between various parts of the building. Differential movements often lead to high internal stresses in building components resulting as distress in the form of cracks, splitting, bending, buckling, or separations in the exterior cladding systems such as brick, cement-board panels, or in the interior finishes such as gypsum drywall panels, wood paneling, and flooring.

3.0 GENERAL DESIGN CONSIDERATIONS

Aside from supporting the building loads, the goal of structural foundation design in expansive soil areas should be to economically mitigate the detrimental effects of foundation movement. This can be done by either isolating elements of the foundation system from potential soil movements or by utilizing design methods and details that help to control the effects of the movement of the soil.

Movements of expansive clay soils are generally restricted to an upper zone of soils known as the active zone. The lower boundary of this zone is commonly defined as the line of zero movement. The depth of the active zone varies from site to site. In the Houston area, this depth is thought to range from 8 to 20 feet. The depth of the active zone is an important design parameter used in the engineering design of foundations on expansive soils, particularly when planning to use deep foundations.

Another general design consideration is the effect of the magnitude of surcharge pressure on the degree of swell that can occur. Lightly loaded foundation components, such as concrete flatwork, pavements, and building slab-on-grade floors, are impacted more by expansive soil volumetric changes than are heavily loaded foundation components such as heavily loaded bearing walls. Heavy loads reduce the amount of swell than can occur.

Numerous foundation system design options are available that meet these goals to varying degrees. Many options are also available in the design and selection of components that make up these foundation systems; however, choices should be based upon an engineered geotechnical investigation. Different options are also available in the design of the site around the foundation and the selection of landscaping components. Advantages and disadvantages of these options are discussed in the following sections.

4.0 FOUNDATION SYSTEM DESIGN OPTIONS

This section discusses the various types of foundation systems that are commonly used for residential and other low-rise buildings in the Houston area where expansive soils occur. In this document, the foundation system is considered to include the structural floor framing system at or near grade level and all other structural components beneath the building. The building superstructure consists of all structural elements above the grade level floor.

The foundation systems are subdivided into two groups: deep support systems and shallow support systems. Each of these systems has an associated level of risk of damage that can occur to the building superstructure and architectural components due to differential foundation movements. Each of these systems also has an associated relative cost of construction. When comparing the various foundation systems, the level of risk is typically found to be inversely proportional to the level of cost. Higher risks are often accepted due to economic considerations. For example, shallow support systems typically have a relatively higher level of risk than deep support systems, but are often selected due to economics and affordability.

Because risk of damage and economic considerations are involved, building owners and/or developers need to be involved in the selection process of the foundation system. To assist in this selection, the foundation systems are generally listed in the order of increasing levels of associated risk and decreasing levels of construction cost.

4.1 DEEP SUPPORT SYSTEMS

Deep support systems are defined as foundations having deep components such as drilled piers or piles that extend well below the moisture active zone of the soils. They function to limit the vertical movements of the building by providing vertical support in a soil stratum that is not susceptible to downward movements caused by moisture fluctuations.

4.1.1 Isolated Structural Systems with Deep Foundations

Isolated structural systems are characterized as having a superstructure and a grade level structural floor system that are designed to be physically isolated from the effects of vertical movements of expansive soils. This is accomplished by providing sufficient space between the bottom of the floor system components and the top of the soil that will allow the underlying expansive soil to heave into the space or subside without causing movement of the floor system. The structural floor system usually consists of a reinforced concrete slab with a void forming system and series of grade beams. Other types of materials and framing systems can be used such as a crawl space, which is created by constructing the floor system above the ground.

4.1.1.1 Structural Slab with Void Space and Deep Foundations

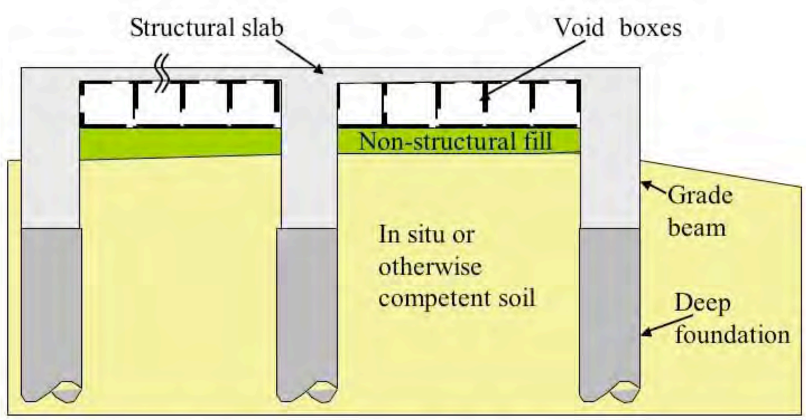

Figure 4.1.1.1 Structural Slab with Void Space and Deep Foundations

This foundation system typically consists of a structural reinforced concrete slab with cardboard carton forms that create a void space that separates the slab from the surface soils. The depth of the void forms ranges from four to eight inches and depends on the expansiveness of the soils. The more expansive the soil (i.e. the higher the plasticity index), the deeper the cardboard carton forms needed. The slab is called a "structural slab" because it spans between reinforced concrete grade beams that are supported entirely by deep foundations.

Because of the relatively small void space that is used with this system, the bottom portion of the grade beams are normally cast directly on the soil, even though they are designed to span between the deep foundations. The slabs typically range in thickness from four to eight inches. The reinforcement can consist of a single or double mat of rebar. The structural slab is designed in accordance with the American Concrete Institute (ACI) publication, *Building Code Requirements for Structural Concrete*, ACI 318.

Void forms serve as formwork for the placement of concrete by acting as a temporary platform that supports the weight of the wet concrete. Void forms typically are made of corrugated paper arranged in an open cell configuration. The exterior surface may be wax impregnated to temporarily resist moisture. The forms are specifically designed to gradually absorb ground moisture, lose strength, disintegrate over time, and leave a void between the expansive soils and the concrete slab. If the soil below the concrete heaves, it can expand into the space created by the void form without lifting the foundation.

TABLE 4.1.1.1 **STRUCTURAL SLAB WITH VOID SPACE AND DEEP FOUNDATIONS**		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Reduces vertical movements of slab-at-grade due to expansive soils provided a sufficient void is maintained under slab and the supporting deep foundations are founded sufficiently below active zone. 2. Usually outperforms any other type of foundation system. 3. Reduces, but does not eliminate, need for a foundation maintenance program. 4. More rigid than a timber framed floor with crawl space and deep foundations, resulting in less differential movement of superstructure. 5. Allows a void under approximately 80–90% of foundation when void cartons are not used under grade beams. 6. No need for select structural fill. Fill can be comprised of expansive or non-expansive soil. Fill need only be compacted to a density sufficient to support slab during setup.	1. Usually results in higher construction cost. 2. Can require additional engineering design effort than a slab-on-grade, and can result in higher engineering fees. 3. Extra time required to construct structurally isolated floor can lengthen overall construction schedule. 4. Improper carton form installation can result in void that is insufficient to provide for anticipated soil expansion. 5. Termites can be attracted to moist cardboard of carton forms. 6. Grade beams that are in contact with soil can heave due to swelling of expansive soils. 7. Depending on slab elevation, can allow water to collect below slab.	1. Slab is constructed about 4 to 8 inches above the soil using void carton forms. 2. Slab is designed to span between grade beams. Grade beams are designed to span between deep foundations. 3. Slab is more heavily reinforced than non-structural slab. 4. Vapor retarders such as polyethylene sheathing should not be placed below carton forms. Vapor retarders should be placed above carton forms in order to allow moisture to degrade void boxes. 5. Usually constructed with no carton forms below grade beams due to potential water infiltration into void and down shafts of deep foundations. 6. Installation of an expendable hard surface above carton forms such as Masonite sheeting will facilitate construction.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.1.1.2 Structural Floor with Crawl Space and Deep Foundations

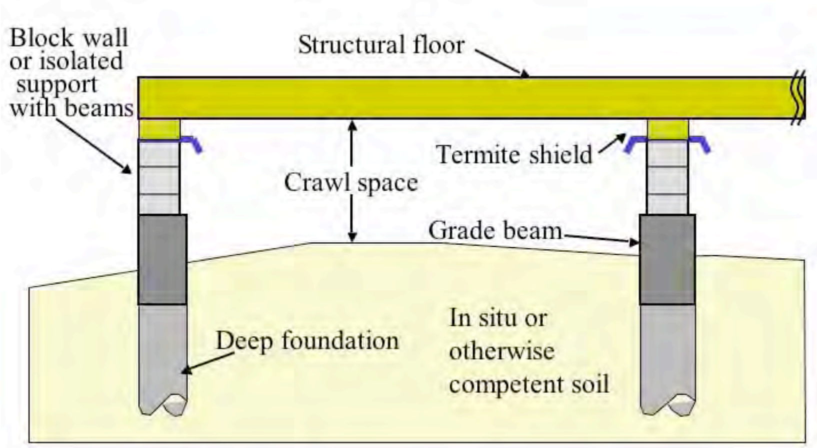

Figure 4.1.1.2 Structural Floor with Crawl Space and Deep Foundations

This foundation system is similar to the previous system, except that the vertical space used to isolate the floor system is much larger, usually at least 18 inches, which is sufficient to allow access underneath the floor, hence the name "crawl space". The structural floor system can be constructed utilizing any of the following common structural components: (a) wood subfloor and joists supported by wood, steel, or concrete beams; (b) concrete floor slab and joists supported by concrete beams; or (c) steel deck and open web bar joists or cold-formed sections supported by steel or concrete beams. Other combinations of these floor-framing components are possible, and other materials can be used such as precast concrete planks or T-sections.

TABLE 4.1.1.2	**STRUCTURAL FLOOR WITH CRAWL SPACE AND DEEP FOUNDATIONS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Reduces vertical movement of slab-at-grade due to expansive soils, provided sufficient crawl space is maintained under slab and supporting deep foundations are founded sufficiently below active zone. 2. Usually outperforms any other type of foundation system. 3. Reduces, but does not eliminate, need for foundation maintenance program. 4. Void cartons are not required under the floor. 5. No need for select structural fill. 6. Accommodates certain architectural styles with raised first floors. 7. Exposed below-floor plumbing is accessible. 8. More suitable for flood-prone areas since ground floor is generally higher than for other foundation systems. 9. Floor is easier to level than a slab-on-grade or structural slab with void space. 10. Helps to preserve nearby existing trees by allowing oxygen to root zones. 11. Allows a void under approximately 95% of foundation when void cartons are not used under grade beams, or nearly 100% when all beams are raised completely above grade. 12. Reduces settlement from soil shrinkage.	1. Usually results in highest construction cost. 2. Requires more extensive design effort, and will result in higher engineering fees. 3. Takes longer to construct because it is labor intensive. 4. Void below floor can collect water if nearby grade or other surrounding sites are at a higher elevation. 5. Less rigid than a stiffened slab, which can allow more differential movement of superstructure, causing more cosmetic distress. 6. Crawl space can allow sufficient oxygen for roots to grow, which can cause soil shrinkage. 7. Proper drainage must be provided in crawl space. 8. Exposed below-floor plumbing can freeze.	1. Ground floor is typically constructed 30 to 42 inches above grade, but can be greater. 2. Floor beams typically consist of steel, concrete, or wood beams spanning between piers over a 12–30-inch high crawl space. 3. Also known as Post-and-Beam, Block-and-Beam, Block-and-Base, or Pier-and-Beam. 4. Flooring typically consists of wood framing, steel framing, precast concrete planks, or precast double tees. 5. Crawl space should be ventilated to evaporate moisture, which accumulates due to natural soil suction, drainage problems, and plumbing leaks. 6. Usually constructed with no carton forms below grade beams due to potential water infiltration into void and down shafts of deep foundations. 7. Vapor retarders, such as polyethylene sheathing, are not recommended to be used to cover soils within crawl space.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.1.2 Stiffened Structural Slab with Deep Foundations

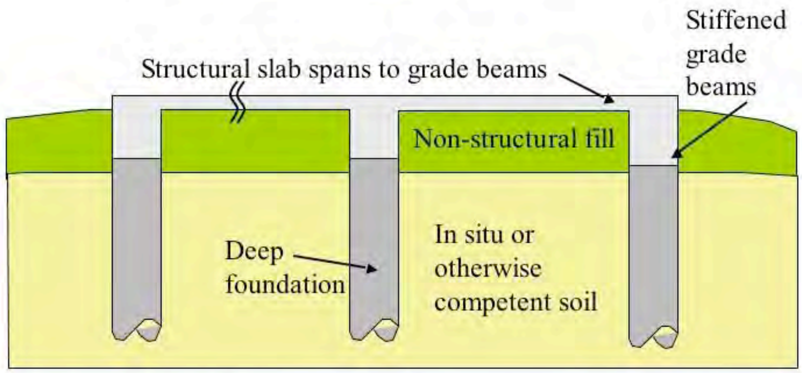

Figure 4.1.2 Stiffened Structural Slab with Deep Foundations

The stiffened structural slab with deep foundations is the same as the structural slab with void space and deep foundations with the following exception: the slab is placed, without a void, over the expansive soils and new fill, and the foundation must be designed to accommodate the pressures from the swelling soils. The foundation is designed as a "stiffened" slab. The grade beams form a grid-like or "waffle" pattern in order to increase the foundation stiffness and reduce the potential bending deflections due to upward movement of the foundation.

Using continuous grade beams in a grid-like fashion helps to reduce differential deflections. The deep foundations are used to minimize downward movement, or settlement, caused by shrinking soils. The stiffened structural slab with deep foundations should be designed to resist heave in accordance with the BRAB 33 (Building Research Advisory Board), Wire Reinforcement Institute (WRI) publication, *Design of Slab-on-Ground Foundations*; the ACI publication, *Design of Slabs on Grade*, ACI 360R, or the Post-Tensioning Institute (PTI) publication, *Design and Construction of Post-Tensioned Slabs-on-Ground*.

TABLE 4.1.2	**STIFFENED STRUCTURAL SLAB WITH DEEP FOUNDATIONS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Compaction of new fill below slab not as critical, and eliminates need for removing existing non-compacted fill. 2. Fill can be comprised of expansive or non-expansive soil. Fill need only be compacted to a density sufficient to support slab during setup. 3. Reduces settlement from soil shrinkage.	1. Does not limit heave that can occur. 2. Requires additional design effort and higher design and construction cost.	1. Slab is designed to span between grade beams. Grade beams are designed to span between deep foundations. 2. Slab is typically 4 to 8 inches thick, beam spacing is less, and slab is more heavily reinforced than for stiffened slab on fill. 3. Stiffening grade beams should be continuous across slab. 4. Slab is more heavily reinforced than non-structural slab. 5. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.1.3 Stiffened Non-Structural Slab with Deep Foundations

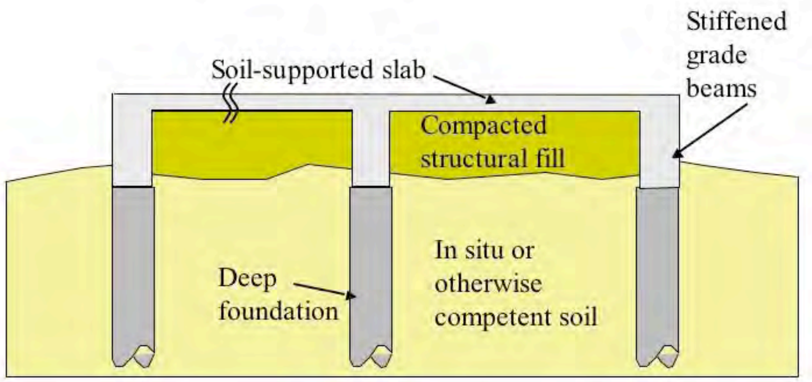

Figure 4.1.3 Stiffened Non-Structural Slab on Fill with Deep Foundations

This type of foundation system is a stiffened concrete slab that can bear on non-expansive select structural fill, with the stiffening grade beams spanning to deep foundations. Select structural fill can be defined as sandy clays with a plasticity index between 10 and 20, and a liquid limit less than 40. The fill acts as a buffer zone between the expansive soils and the slab, reducing the potential differential movement of the foundation. The foundation is designed as a ribbed mat that is "stiffened" with relatively deep and closely spaced grade beams. The grade beams are laid out in a grid-like or "waffle" pattern and are designed with sufficient stiffness to reduce the bending deflection caused by shrinking or swelling soils. See Section 4.1.2 for additional design information.

TABLE 4.1.3	**STIFFENED NON-STRUCTURAL SLAB WITH DEEP FOUNDATIONS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Usually less expensive than structurally isolated systems with deep foundations. 2. Slab thickness and reinforcing is usually less than that of structurally isolated systems. 3. Settlement from soil shrinkage is usually less than that of shallow supported foundations.	1. To resist potential uplift forces, grade beams may need to be deeper than those of a structurally isolated system.	1. Stiffening grade beams should be continuous across slab. 2. Select structural fill can be used to reduce potential vertical rise. 3. Subgrade and fill, if used, should be field-verified for conformance to geotechnical specifications. 4. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.1.4 Non-Stiffened Slab-on-Grade with Deep Foundations

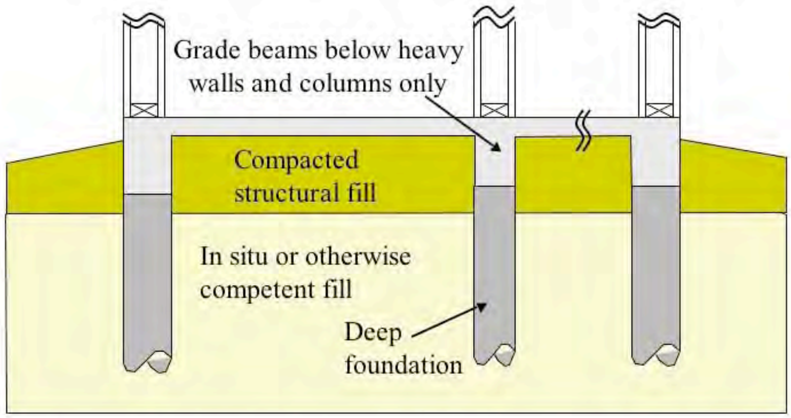

Figure 4.1.4 Non-Stiffened Slab-on-Grade with Deep Foundations

This system consists of a slab-on-grade with grade beams under load bearing walls supported by deep foundations. The foundation will move with the underlying soils. The foundation has little resistance to soil movement with this system. Perimeter grade beams are typically provided with this system to support the exterior wall system and to reduce undermining by erosion. They can also function as a root retarder or vertical moisture retarder. Interior grade beams are also usually provided under all interior load-bearing walls and shear walls. Interior columns are typically supported directly by deep foundations.

TABLE 4.1.4	**NON-STIFFENED SLAB-ON-GRADE WITH DEEP FOUNDATIONS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Comparatively easy and quick to construct. 2. Typically has fewer grade beams than stiffened slab foundation systems. 3. Construction joints and isolation joints can be used with this system to allow separate concrete placements.	1. Does not significantly reduce amount of differential vertical movement that can occur. 2. More distress to the superstructure may occur with this system. 3. Lack of grade beams may not provide sufficient stiffness for jacking if future underpinning is required.	1. Flat slab rests directly on underlying soil. 2. Warehouses, where interior slab movements can be tolerated, are often constructed using this method. 3. Select structural fill can be used to reduce potential vertical movements. 4. Subgrade and fill, if used, should be field-verified for conformance to geotechnical specifications. 5. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.2 SHALLOW SUPPORT SYSTEMS

Shallow support foundation systems are defined as foundations having shallow foundation components that do not extend below the moisture active zone of the soils and are subject to vertical movements due to volumetric changes of the expansive soils.

4.2.1 Grade-Supported Stiffened Structural Slab

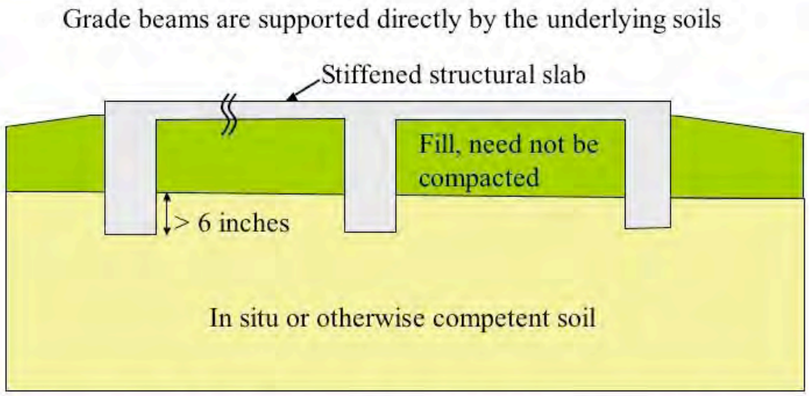

Figure 4.2.1 Grade-Supported Stiffened Structural Slab

This foundation system is similar to that discussed in Section 4.1.2, except that the grade beams are supported directly by the underlying soils instead of spanning to deep foundations. The key advantage of this system over that discussed in Section 4.2.2 is that the grade beams need only to penetrate a minimum of six inches into the competent natural soils or properly compacted fill. Fill placed between the grade beams is only required to be compacted enough to support the concrete during placement.

TABLE 4.2.1	**GRADE-SUPPORTED STIFFENED STRUCTURAL SLAB**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Compaction of new fill below slab not as critical, and eliminates need for removing existing non-compacted fill. 2. Usually performs better than other grade-supported slabs. 3. If fill below slab is loosely compacted, potential vertical rise can be reduced as compared to other grade-supported foundations. 4. Faster to construct than slabs with deep foundations.	1. May experience more vertical movement than stiffened slabs on deep foundations. 2. More expensive than slab-on-grade and non-structural systems due to more concrete and reinforcement. 3. Requires more design effort than non-structural slab systems.	1. Also referred to as a ribbed mat or "super slab". 2. Grade beams must be supported by competent soils. 3. Slab is designed to structurally span between grade beams. 4. Slab is typically 4 to 6 inches thick, depending on beam spacing. 5. Grade beams can be wider or more closely spaced than other grade-supported slabs. 6. Stiffening grade beams should be continuous across slab. 7. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.2.2 Grade-Supported Stiffened Non-Structural Slab

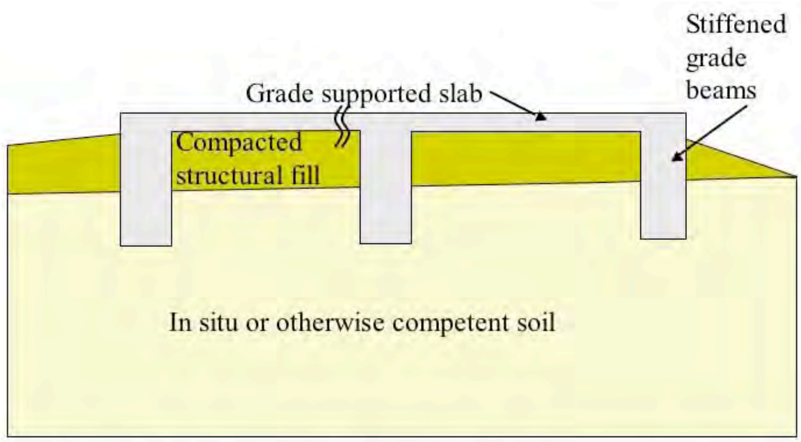

Figure 4.2.2 Grade-Supported Stiffened Non-Structural Slab

This foundation system is similar to that discussed in Section 4.1.3, except that the grade beams are supported directly by the underlying soils instead of spanning to deep foundations. It is also similar to Section 4.2.1 except that the entire stiffened slab is supported by the surface soils that are susceptible to the seasonal moisture fluctuations and movement. The foundation is designed utilizing continuous stiffening beams that form a grid like pattern. Grade-supported stiffened slabs should be designed in accordance with the WRI publication, *Design of Slab-on-Ground Foundations*, the ACI publication, *Design of Slabs on Grade*, ACI 360R, or the PTI publication, *Design and Construction of Post-Tensioned Slabs-on-Ground*.

TABLE 4.2.2	**GRADE-SUPPORTED STIFFENED NON-STRUCTURAL SLAB**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Most economical system used where expansive soils are present. 2. Faster to construct than slabs with deep foundations.	1. May experience more vertical movement than stiffened slabs supported on deep foundations.	1. Stiffened slabs are sometimes called “waffle” or “floating” foundations. 2. Grade beams must be supported by competent soils. 3. Most commonly used foundation system in Houston area. 4. Stiffening grade beams should be continuous across slab. 5. Subgrade and fill, if used, should be field-verified for conformance to geotechnical specifications. 6. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.2.3 Grade-Supported Non-Stiffened Slab of Uniform Thickness

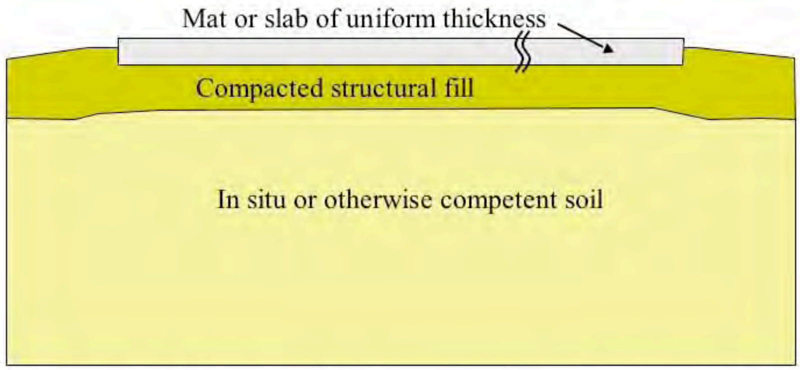

Figure 4.2.3 Grade-Supported Non-Stiffened Slab of Uniform Thickness

This system consists of a concrete slab-on-grade of uniform thickness with no deep support foundation components. The slab can be supported on in situ soils or compacted fill. This foundation system should be designed by the PTI method or other acceptable engineering methods to resist the potential bending moments induced by the differential deflections of the slab when subject to expansive soil movements.

TABLE 4.2.3 GRADE-SUPPORTED NON-STIFFENED SLAB OF UNIFORM THICKNESS		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Faster to construct than stiffened slabs and deeply supported foundations. 2. Eliminates digging of grade beams. 3. Lack of grade beams makes it easier to jack against if underpinning is later required.	1. May experience more vertical movement than stiffened slabs on deep foundations. 2. Potentially has more vertical differential displacement than stiffened slabs with the equivalent volume of concrete. 3. Allows roots to grow below foundation because there are no perimeter grade beams.	1. Also called a "California Slab". 2. Behaves similar to a mat foundation. 3. Flat slab rests directly on underlying soil. 4. May include a perimeter grade beam as a root retarder or to prevent erosion. 5. Typically reinforced with conventional deformed bar reinforcing or post-tensioned cable. 6. Suitable for deep sandy soil or foundations having consistent subsoil formations with low propensity for volumetric movement. 7. Subgrade and fill, if used, should be field-verified for conformance to geotechnical specifications. 8. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.1.1 to 4.2.3.

4.3 MIXED DEPTH SYSTEMS

Mixed depth systems are foundations that extend to different bearing depths. They are sometimes used to support concentrated loads. Although their use is discouraged for certain applications, mixed depth foundation systems are sometimes used. They can be used for new buildings with large plan areas located on a site with widely varying soil conditions, for new buildings on sites with a substantial amount of deep fill, for new buildings on a sloping hillside, for new buildings located next to a waterway or slopes greater than 5%, for existing buildings when adding a new addition, etc. When a new addition is added onto an existing building, consideration must be given to the depths of the new and existing foundation systems.

4.3.1 Mixed Depth System for All-New Building Construction

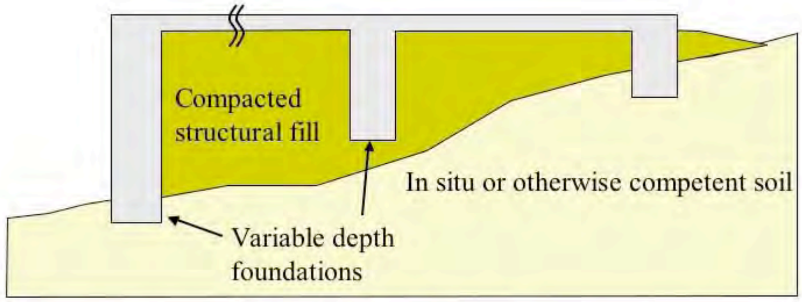

Figure 4.3.1 Mixed Depth System for All New Building Construction

Because of the increased possibility of differential movement, mixed depth systems are not often used for all-new construction except in areas of sloping grades and sloping strata.

TABLE 4.3.1	**MIXED DEPTH SYSTEM FOR ALL-NEW BUILDING CONSTRUCTION**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. More economical than uniformly deep foundation system	1. More likely to experience differential movement than foundations of uniform depth.	1. Pier depth, if included, can vary to follow bearing stratum or to address slope instability issues. 2. Often used for perimeter and point loaded commercial buildings. 3. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.3.1 to 4.3.3.

4.3.2 Mixed Depth System for Building Additions with Deep Foundations

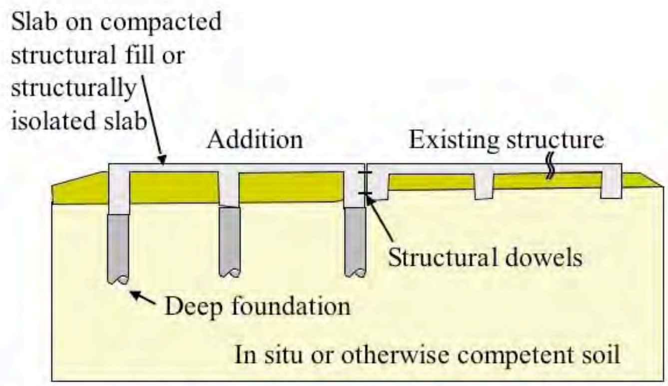

Figure 4.3.2 Mixed Depth System for Building Addition with Deep Foundations

Sometimes building additions are designed with deeper foundations than the original building in order to reduce movement of the addition. This is because the foundation of the older portion of the building has stabilized.

TABLE 4.3.2	MIXED DEPTH SYSTEM FOR BUILDING ADDITIONS WITH DEEP FOUNDATIONS	
ADVANTAGES *	DISADVANTAGES *	COMMENTS
1. Addition is more stable than a new grade-supported foundation due to less seasonally active soils at increased foundation depth.	1. More expensive than using a shallow foundation system for new addition.	1. When used in conjunction with a new structurally isolated slab (i.e. isolated from soil movement, and structurally connected to existing building) minimizes risk of differential movement. 2. Is not designed to prevent foundation tilt.

* Compared to other foundation systems as described in Sections 4.3.1 to 4.3.3.

4.3.3 Mixed Depth System for Building Addition with Shallow Foundations

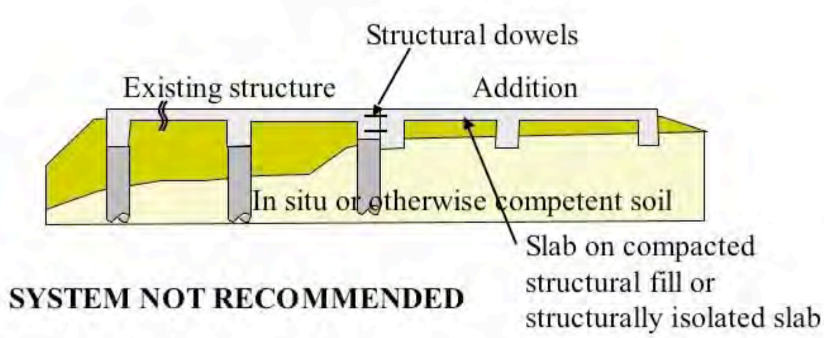

Figure 4.3.3 Mixed Depth System for Building Addition with Shallow Foundations

Sometimes additions are built with shallower foundations than the original building in order to reduce the cost of construction.

TABLE 4.3.3	**MIXED DEPTH SYSTEM FOR BUILDING ADDITION WITH SHALLOW FOUNDATIONS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. More economical than uniformly deep foundation system.	1. On expansive, compactable, or compressible soils, more shallowly supported addition is likely to move more than existing building.	1. Not recommended due to high probability of differential movement.

* Compared to other foundation systems as described in Sections 4.3.1 to 4.3.3.

5.0 FOUNDATION COMPONENT DESIGN OPTIONS

This section covers the advantages and disadvantages of common component design options for the systems that were discussed in Section 4.0. Components are referenced to other components in the same category.

5.1 DEEP SUPPORT COMPONENTS

This section discusses deep foundation support components that are commonly used in new construction for residential and other low-rise buildings. This includes drilled and underreamed piers, drilled straight-shaft concrete piers, auger-cast concrete piles, displacement piles, and helical piers.

5.1.1 Drilled and Underreamed Concrete Piers

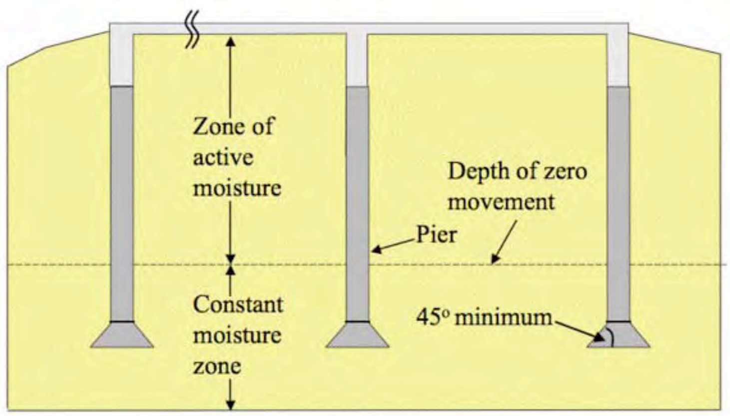

Figure 5.1.1 Drilled and Underreamed Concrete Piers

Drilled and underreamed piers are cast-in-place concrete foundation components with an enlarged bearing area extending downward to a soil stratum capable of supporting the loads. Drilled and underreamed concrete piers have also been referred to as drilled piers, drilled shafts, caissons, drilled caissons, belled caissons, belled piers, bell-bottom piers, foundation piers, bored piles, and or drilled-and-underreamed footings. The depth of the drilled pier should extend to a depth below the moisture active zone that is sufficient to anchor the pier against upward movements of swelling soils in the upper active zone.

TABLE 5.1.1 — **DRILLED AND UNDERREAMED CONCRETE PIERS**		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Has a long, successful track record, approaching a century of use. 2. Provides better lateral load resistance than other deep foundations with smaller projected surface areas. 3. Underreamed portion economically provides large end bearing capacity. 4. Most commonly used deep foundation component in Houston area. 5. Easier to install than displacement piles in very stiff sandy soils.	1. Installation requires a minimum of four different procedures: shaft drilling, underreaming, reinforcing steel placement, and concrete placement. 2. Requires removing excavated soils off-site. 3. Sloughing of soils at pier shaft and bell can create installation problems. 4. Difficult to confirm integrity of concrete placed under groundwater or slurry conditions. 5. May be difficult for some contractors to install drilled piers below a depth of 15 feet because of equipment limitations. 6. Requires waiting until concrete sufficiently cures before applying load. 7. Drilling piers below soil active moisture zone in Houston area often results in encountering water or sands.	1. Many contractors falsely believe underreams should be founded in certain color or stiffness clay rather than at depth shown on foundation engineering drawings. 2. Slump should be greater than 5 inches to prevent honeycombing. 3. Vertical reinforcement should be used to resist tensile forces due to friction on shaft from swelling soils. 4. Can be constructed in areas with high groundwater table by using slurry displacement method. 5. Can be installed through sandy layers by using retrievable casing.

* Compared to other deep supporting elements as described in Sections 5.1.2 to 5.1.5.

5.1.2 Drilled Straight-Shaft Concrete Piers

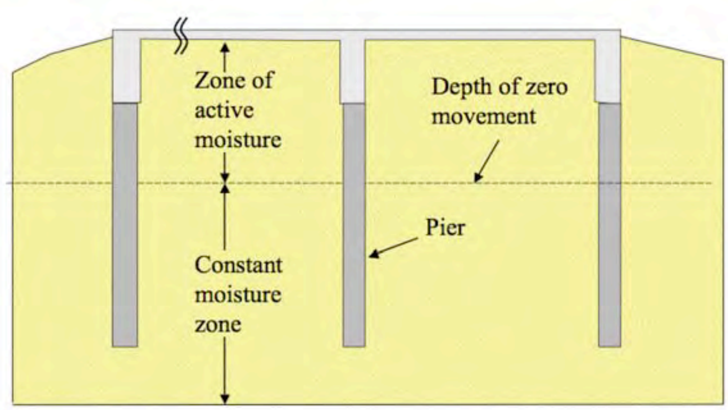

Figure 5.1.2 Drilled Straight-Shaft Concrete Piers

Drilled piers are cast-in-place concrete foundation components extending downward to a soil stratum capable of supporting the loads. Drilled straight-shaft concrete piers are not underreamed.

TABLE 5.1.2	**DRILLED STRAIGHT-SHAFT CONCRETE PIERS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Quality control is simpler than for drilled and underreamed piers. 2. Shafts are typically larger diameter than drilled and underreamed piers, and provide better lateral load resistance. 3. Easier to inspect than underreamed piers. 4. Has a long, successful track record, more than a century of use. 5. Easier to install than displacement piles in very stiff sandy soils.	1. Soil borings are required to be deeper than underreamed piers, which adds to cost. 2. Geotechnical reports do not routinely provide shaft allowable skin friction capacity values. 3. Requires removing excavated soils off-site. 4. Can require steel casing to drill through sandy soils. 5. Can require slurry or concrete to be pumped to bottom of hole when groundwater is encountered. 6. Requires waiting until concrete sufficiently cures before applying load. 7. Sloughing of soils can create installation problems. 8. Difficult to confirm integrity of concrete placed under groundwater or slurry conditions. 9. May be difficult for some contractors to install drilled piers below a depth of 15 feet because of equipment limitations. 10. Drilling straight-shaft piers often results in encountering water or sands. 11. Drilling piers below soil active moisture zone in Houston area often results in encountering water or sands.	1. Many contractors falsely believe underreams should be founded in certain color or stiffness clay rather than at depth shown on foundation engineering drawings. 2. Only recently used as an alternative to drilled and underreamed footings in light foundation industry. 3. Slump should be greater than 5 inches to prevent honeycombing. 4. Vertical reinforcement should be used to resist tensile forces due to friction on shaft from swelling soils. 5. Can be constructed in areas with high groundwater table by using slurry displacement method. 6. Can be installed through sandy layers by using retrievable casing.

* Compared to other deep supporting elements as described in Sections 5.1.1 and 5.1.3 to 5.1.5.

5.1.3 Auger-Cast Concrete Piles

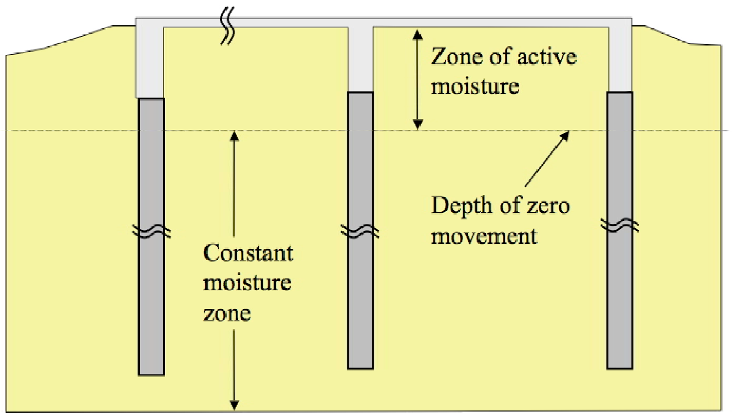

Figure 5.1.3 Auger-Cast Concrete Piles

Auger cast piles are installed by rotating a continuously-flighted hollow shaft auger into the soil to a specified depth. Cement grout is pumped under pressure through the hollow shaft as the auger is slowly withdrawn.

TABLE 5.1.3	AUGER-CAST CONCRETE PILES	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Can be readily installed through sand strata and below the water table. 2. Can be easily installed at angles other than vertical.	1. Reinforcing cage must be installed after auger is removed, which limits depth of reinforcing cage that can be installed and may result in inadequate concrete cover due to cage misalignment. 2. If singly reinforced, auger-cast piles do not provide significant bending resistance. 3. Higher mobilization costs than for other systems. 4. Fewer contractors are available that offer this system, making construction pricing less competitive. 5. Soil borings are required to be deeper than underreamed piers, which adds to cost. 6. Geotechnical reports do not routinely provide shaft allowable skin friction capacity values. 7. Requires removing excavated soils off-site. 8. Requires waiting until concrete sufficiently cures before applying load. 9. Difficult to confirm integrity of concrete.	1. Commonly utilized in situations drilling through collapsing soils and emerging free water. 2. Vertical reinforcement should be used to resist tensile forces due to friction on shaft from swelling soils. 3. Only recently used as an alternative to drilled and underreamed footings in light foundation industry. 4. Slump should be greater than 5 inches to prevent honeycombing.

* Compared to other deep supporting elements as described in Sections 5.1.1, 5.1.2, 5.1.4 and 5.1.5.

5.1.4 Displacement Piles

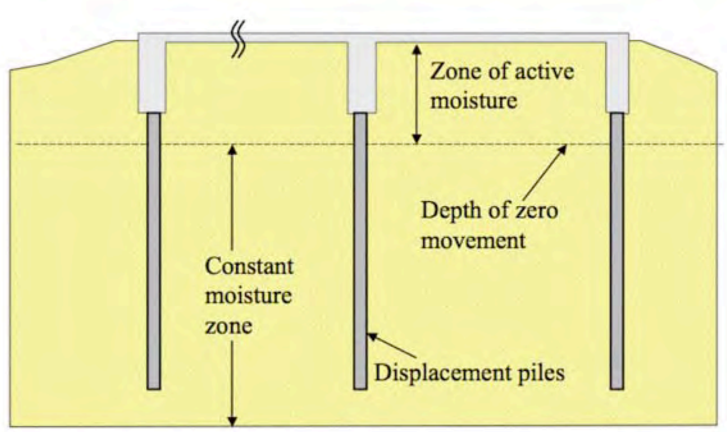

Figure 5.1.4 Displacement Piles

For the purpose of this document, displacement piles are defined as relatively long slender members driven, vibrated, or pressed into the soil while displacing soil at the pile tip.

TABLE 5.1.4	DISPLACEMENT PILES	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. No excavated soils to remove. 2. Only one trade typically involved during installation. 3. Easier to remove than drilled piers if future demolition is required. 4. Can be installed through soft soils and water-bearing strata.	1. Vibrations and noise that occur during installation can be a problem. 2. Difficult to install through stiff sand strata. 3. Because of relatively small diameters, grouping or clustering can be required, which can lead to other potential problems.	1. Typically used at shoreline locations, swamps, marshes, or other soft soil areas.

* Compared to other deep supporting elements as described in Sections 5.1.1 to 5.1.3, and 5.1.5.

5.1.5 Helical Piers

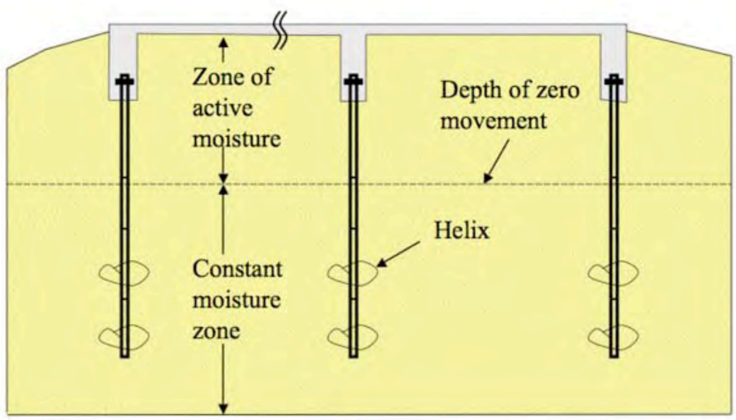

Figure 5.1.5 Helical Piers

Steel helical piers, also known as screw anchors or screw-in piles, have been used since the early 1950s as tie back anchors for retaining walls and as foundations for lighthouses, substations, towers, heavy equipment, and other similar applications. They are now gaining popularity for use in supporting heavier foundations such as residential and other low-rise buildings.

The anchor consists of a plate or series of steel plates formed into the shape of a helix to create one pitch of a screw thread. The shape of the plate permits easy installation, which is accomplished by applying torque to the shaft of the anchor and screwing it into the ground using rotary motors. The anchors can be used to resist a tensile or compressive load, which is accomplished by means of bearing pressure resistance on the area of each helix, and not by skin friction along the shaft. The plate helices of helical pier foundations are attached to a central high-strength steel shaft that can be segmented to facilitate construction and to allow various combinations of the number and diameter of helices used. The pier is screwed into the soils until the applied torque readings indicate that the necessary load capacity has been achieved or until the desired depth below the moisture active zone of the expansive soils is obtained. In new construction, the pier shafts are typically anchored to the grade beams by using fabricated brackets that are tied to the grade beam reinforcing before placing the concrete, and bolted to the top of the pier shafts. The bracket consists of a flat horizontal plate welded to a vertical square tube that slips over the shaft of the pier. The plate is embedded into the grade beam concrete.

TABLE 5.1.5	**HELICAL PIERS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Can be installed in low headroom applications in limited access areas. 2. Shaft of helical pier has small surface area, which limits amount of uplift or drag-down frictional forces that can occur due to vertical movements of expansive soils. 3. Only licensed contractors may install helical piers, which provide some means of assuring that contractors are tested and trained in all facets of helical pier construction. 4. Does not require excavation of soil for installation, thus providing minimal disturbance to site. 5. Can be installed in less time than drilled piers or auger-cast concrete piles. 6. Can be installed at a batter to resist lateral loads on foundation. 7. Shaft extensions can easily be added to install helices deep below soil moisture active zone. 8. Loads can be applied immediately after installation. 9. Only one trade typically involved during installation.	1. Fewer contractors are available that offer this system, and construction pricing is less competitive. 2. Usually requires on-site test pier to verify installability and load bearing capacity. 3. Although steel is often galvanized, corrosion can limit life expectancy. 4. In very soft soils with low lateral restraint, external concrete jacket is required to prevent buckling of small shaft under large loads. 5. Square shafts of helical piers disturb soil around shaft during installation to some extent, and can result in gaps occurring between soil and shaft along full length of pier. This gap can become a pathway for water to flow down around shaft and activate swelling of dry expansive soils in non-active zone, and can also allow air and moisture to speed up rate of steel corrosion. 6. Vertically installed helical piers provide little resistance to lateral forces because of their small shaft diameter.	1. Additional protective coatings (e.g., coal-tar epoxy) or cathodic protection can be used to control corrosion.

* Compared to other deep supporting elements as described in Sections 5.1.1 to 5.1.4.

5.2 SLAB AND GRADE BEAM REINFORCING

Since concrete is weak in tension, concrete slabs and grade beams are almost always reinforced with some type of steel reinforcing. The most common design options include post-tensioned reinforcing, deformed bar reinforcing, welded wire fabric reinforcing, and fiber reinforcing. Under special circumstances, unreinforced plain concrete can also be used. Advantages and disadvantages of these types of reinforcement for slab-on-grade and grade beam applications are discussed below.

5.2.1 Post-Tensioned Reinforcing

Post-tensioned concrete is a type of prestressed concrete in which the cables are tensioned after partial curing of the concrete has occurred. Pretensioned prestressed concrete, in which the cables are tensioned before placement of concrete around them, is not commonly used for slabs and grade beams in residential and other low-rise buildings. Post-tensioned reinforcing has become the norm for residential slabs in most Texas metropolitan areas.

Post-tensioned reinforcing consists of high-strength steel wire strands, typically referred to as tendons or cables, which are encased in plastic sheathing or ducts. When also used near the bottom of grade beams, the tendons are usually located near the top of the grade beam at the ends of the span and draped into the bottom portion of the grade beam near mid-span.

Tendons typically consist of 1/2-inch diameter high-strength seven-wire strands having yield strength of 270 ksi. The tendons are elongated by hydraulic jacks and held in place at the edges of the foundation by wedge-type anchoring devices. The type of tendons typically used in residential slabs and grade beams are unbonded tendons, in which the prestressing steel is not actually bonded to the concrete that surrounds it except at the anchored ends. Coating the steel strands with corrosion-inhibiting grease and encasing them in extruded plastic protective sheathing that acts as a bond-breaker will accomplish this. The tendons are typically fully stressed and anchored 3 to 10 days after concrete placement.

TABLE 5.2.1	**POST-TENSIONED REINFORCING**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Costs less than conventional steel rebar reinforcing. 2. Speeds up construction because there are fewer pieces to install than conventionally reinforced. 3. Controls the size of curing and shrinkage surface cracks in concrete slabs after tendons are stressed. 4. Can reduce required amount of control joints in slab. 5. Slab is designed as an uncracked section, therefore should require less concrete.	1. Requires specialized knowledge and expertise to design, fabricate, assemble, and install. 2. Geotechnical design parameters, such as y_m and e_m, are not consistently defined among geotechnical engineers. 3. Slab design can be compromised if cracks open before stressing and fill with debris. 4. Making penetrations into slab can be hazardous due to presence of tensioned cables. 5. Additional operations such as stressing, cutting, and grouting are required after concrete placement. 6. Cannot prevent cracks prior to stressing caused by plastic shrinkage, plastic settlement, and crazing at slab surface. 7. Tendon end anchorages, which are highly stressed critical elements of system, are located at exterior face of foundation where exposed strand ends and anchors can be susceptible to corrosion. 8. Post-tensioned reinforced foundations are susceptible to blowouts, in which sudden concrete bursting failure occurs during or after stressing. If a tendon or anchorage fails or a blowout occurs, additional operations are required for repair.	1. Normally used locally to reinforce stiffened slabs-on-grade but can be used for other configurations as well. 2. Post-tensioned foundations are typically designed using Post Tensioning Institute (PTI) publication *Design and Construction of Post-Tensioned Slabs-on-Ground*. 3. Compared to other types of post-tensioned construction, residential slabs are lightly reinforced with average concrete compression levels ranging only between 50 psi and 100 psi.

* Compared to other types of slab and grade beam reinforcing described in Sections 5.2.2 to 5.2.5.

5.2.2 Deformed Bar Reinforcing

Deformed bar reinforcing, commonly call rebar, typically consists of ASTM 615 steel having yield strength of either 40 or 60 ksi. Grade 40 rebar was more common in pre-1970 construction, and Grade 75 rebar is expected to become more common in the future. Deformed bar reinforcing is categorized as "passive" reinforcement since it does not carry any force until the concrete member deflects and cracks under applied loads. On the other hand, post-tensioned tendons are considered "active" reinforcing because they are prestressed and carry tensile force even when loads are not applied to the concrete member.

TABLE 5.2.2	**DEFORMED BAR REINFORCING**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Less technical installation operations than for a post-tensioned foundation system. 2. Post-construction slab penetrations are less hazardous to install than in slabs with post-tensioned reinforcing. 3. Field splices are easier to implement.	1. Requires specialized knowledge and expertise to design, fabricate, assemble, and install. 2. Costs more than post-tensioned reinforcing. 3. Slower to construct than post-tensioned reinforced slabs 4. Foundation performance is more sensitive to correct placement of slab reinforcement.	1. Some local building officials do not require deformed bar reinforced foundations to be engineered, even though design has similar difficulty as does post-tensioned foundations. 2. Deformed bar reinforced slab foundations are typically designed per American Concrete Institute (ACI) publication ACI 360R, *Design of Slabs on Grade*, charts from Portland Cement Association (PCA) publications *Concrete Floors on Ground* and *Slab Thickness Design for Industrial Concrete Floors on Grade*, Wire Reinforcement Institute (WRI) publication *Design of Slabs on Grade*, Building Research Advisory Board (BRAB) publication *Criteria for Selection and Design of Residential Slabs-on-Grade*, Post Tensioning Institute (PTI) publication *Design and Construction of Post-Tensioned Slabs-on-Ground,* or finite element methodology. 3. Local building officials may not require construction certification by engineers of foundations using only deformed bar reinforcing.

* Compared to other types of slab and grade beam reinforcing described in Sections 5.2.1 and 5.2.3 to 5.2.5.

5.2.3 Welded Wire Fabric Reinforcing

Welded wire fabric concrete reinforcing consists of cold-drawn wire in orthogonal patterns, square or rectangular, that is welded at all intersections, and is typically used in slab construction. Welded wire fabric (WWF) is commonly called "wire mesh", but mesh is a much broader term that is not limited to concrete reinforcement. Welded wire fabric can be made of smooth wire (ASTM A185) or deformed wire (ASTM A497), and can be manufactured in sheets (usually wire sizes larger than W4) or rolls (usually wire sizes smaller than W1.4).

TABLE 5.2.3	WELDED WIRE FABRIC REINFORCING	
ADVANTAGES *	DISADVANTAGES *	COMMENTS
1. Welded wire fabric rolls can be manufactured in any lengths, up to maximum weight per roll that is convenient for handling (100-200 ft). 2. Has greater yield strength than conventional deformed bar reinforcing, which can result in reducing required cross-sectional area of steel. 3. Development lengths are typically much smaller than for deformed bar reinforcing. 4. Concrete shrinkage cracks can be kept smaller due to confinement offered by welded cross wires, and microcracking is better distributed. 5. Labor costs to install welded wire fabric are less than to install conventional deformed bar reinforcing through elimination of tying reinforcing rods and faster placement of large sheets.	1. Welded wire fabric is difficult to position and hold in place within thickness of slab. Slab crack performance is more sensitive to correct placement of reinforcement. 2. Shipping restrictions as well as manufacturing limitations can limit maximum sheet size. 3. Application is generally limited to slabs only. 4. Heavy welded wire fabric reinforcement may not be readily available, and can require special order and/or long lead-time. 5. Practice of placing wire mesh on subgrade and using hooks to lift it, as workers walk on mesh, invariably results in large areas of mesh remaining at bottom of slab.	1. Welded wire fabric reinforced residential foundations are typically designed per Wire Reinforcement Institute (WRI) methodology. 2. Local building officials may not require engineering certification of foundations using only welded wire fabric.

* Compared to other types of slab and grade beam reinforcing described in Sections 5.2.1, 5.2.2, 5.2.4, and 5.2.5.

5.2.4 Fiber Reinforced Concrete

Fiber reinforced concrete consists of synthetic or steel fibers that help to control plastic shrinkage cracking, and plastic settlement cracks. Helps reduce bleeding and water migration to slab surface, which helps to control water-cement ratio and produce concrete with less permeability and improved toughness. Fiber reinforced concrete helps increase impact resistance and surface abrasion resistance of concrete.

Fiber reinforced concrete is not a substitute for structural reinforcing per ACI 544R-88. Therefore, advantages and disadvantages are not given.

5.2.5 Unreinforced Concrete

Unreinforced concrete, also known as "plain" concrete, is concrete without any reinforcing. Soil-supported concrete slabs can be designed as plain concrete, as well as continuously supported grade beams. Unreinforced concrete should not be used for structural foundations on expansive soils and is not recommended for use in slabs subject to movement unless cracking is not objectionable. Therefore, advantages and disadvantages are not given.

5.3 VOID SYSTEMS UNDER GRADE BEAMS AND PIER CAPS

Voids used by some engineers under grade beams and concrete caps for deep foundations are commonly created by using the same type of wax-impregnated corrugated cardboard forms described in Section 4.1.1.1. If the grade beams are constructed by using the trenching method in which concrete is cast directly against the soil in the excavated trenches, self-disintegrating void forms must be used.

An alternate method of grade beam construction is to form the sides of the concrete grade beams, particularly for foundation designs that use non-expansive backfill that elevate the slab above the surrounding grade for drainage purposes. In this case, it is possible to use removable forms to create the voids under the grade beams, and to install strip forms along the bottom edges of the grade beams to keep the soil from filling the voids when backfilling against the sides of the grade beams.

Carton forms are not typically used below grade beams due to potential water infiltration into void and down shafts of deep foundations.

5.4 VAPOR RETARDERS

A vapor retarder (sometimes misleadingly called a vapor barrier) is sheeting material, usually polyethylene film, which is placed under a ground level concrete slab in order to reduce the transmission of water vapor from the soils below the foundation up through the concrete slab. Vapor retarders are commonly used where moisture can migrate upward from below the slab and cause damage to floor coverings, household goods, or stored materials. Vapor retarders should be overlapped at least 6" at the joints, and should be carefully fitted around pipes and other service penetrations through the slab.

In typical southeast Texas area slab-on-grade construction, the vapor retarder is normally placed directly on top of the finish-graded in situ clay or sandy clay soil, or when fill is added, on top of the non-expansive select fill. If cardboard void cartons are used, the vapor retarder should be placed above the void forms in order to allow moisture to degrade the void boxes.

For residential construction in areas having expansive soils, concrete is most commonly placed directly on the vapor retarder. At grade beam locations, the vapor retarder may be draped down into the excavated trench and be continuous around the exterior surface of the grade beam.

5.5 GRADE-BEAM-TO-PIER CONNECTIONS

Traditionally, drilled pier shafts in new construction have been tied to the grade beams with hooked or long straight rebar anchorage to create a connection. The recent trend over the last decade in the Houston area residential construction market is to allow the grade beams to float on top of the deep foundation components with no vertical restraints. This eliminates stresses due to fixed pier-to-beam connections.

5.5.1 Grade-Beam-to-Pier Connections with No Restraints

Grade-beam-to-pier connections with no restraints implies that the foundation grade beams are cast atop the already cured drilled piers, which are flat and allow the grade beam to translate relative to the pier in all directions except vertically downward.

TABLE 5.5.1	GRADE-BEAM-TO-PIER CONNECTIONS WITH NO RESTRAINTS	
ADVANTAGES *	DISADVANTAGES *	COMMENTS
1. Allows foundation to rise more uniformly if underlying soils swell. 2. Easier to clean mud from top of pier caps. 3. Pier steel does not interfere with trencher during grade beam excavation.	1. Foundation can move laterally due to swelling soils or sloping sites. 2. After concrete is placed, cannot easily verify if reinforcing steel was installed in piers.	

* Compared to other types of grade-beam-to-pier connections described in Sections 5.5.1 to 5.5.3.

5.5.2 Grade-Beam-to-Pier Connections with Horizontal-Only Restraints

Grade-beam-to-pier connections with horizontal-only restraints implies that the foundation grade beams have a positive connection to the piers in any lateral direction, but the grade beams are allowed to translate vertically upward, relative to the piers.

TABLE 5.5.2	GRADE-BEAM-TO-PIER CONNECTIONS WITH HORIZONTAL-ONLY RESTRAINTS	
ADVANTAGES *	DISADVANTAGES *	COMMENTS
1. Horizontal foundation movement is limited. 2. Allows foundation to rise more uniformly if underlying soils swell.	1. Requires additional labor and material.	1. Bond-breakers, such as sleeved deform bars, non-deformed bar dowels, and shear keys can be used. 2. Recommended at sloping sites where lateral resistance is required.

* Compared to other types of grade-beam-to-pier connections described in Sections 5.5.1 to 5.5.3.

5.5.3 Grade-Beam-to-Pier Connections with Horizontal and Vertical Restraints

Grade-beam-to-pier connections with horizontal and vertical restraints implies that the foundation grade beams are connected to the top of the piers in such a way that there can be no relative translation in any direction.

TABLE 5.5.3	GRADE-BEAM-TO-PIER CONNECTIONS WITH HORIZONTAL AND VERTICAL RESTRAINTS	
ADVANTAGES *	DISADVANTAGES *	COMMENTS
1. May provide uplift resistance from swelling soils if piers are adequately anchored. 2. Horizontal foundation movement is limited.	1. Impedes or prevents jacking of a foundation that must be lifted. Pier reinforcing must be severed before lifting. 2. Does not allow grade beams to freely lift off piers if upper strata heave occurs. This can cause distress in slab because it is more flexible than grade beams. 3. Beams, beam-to-pier connections, and slab must be designed for uplift forces due to swelling soils.	1. Connection typically is an extension of pier shaft vertical deformed reinforcement. 2. Recommended at sloping sites where lateral resistance is required.

* Compared to other types of grade-beam-to-pier connections described in Sections 5.5.1 to 5.5.3.

6.0 FOUNDATION SITE DESIGN OPTIONS

The various types of mitigation options to reduce the damaging effects of soil movement due to improper drainage and transpiration of trees and shrubbery that are discussed in this guide can be categorized into two basic groups: (1) moisture control systems, and (2) vegetation control systems.

6.1 MOISTURE CONTROL SYSTEMS

Moisture control systems mitigate damage by controlling the amount of water and moisture that enter into the site soils. This includes methods to control stormwater runoff and methods of providing irrigation to site vegetation.

6.1.1 Site Drainage Systems

Three methods of controlling site drainage include site grading, French drains, and area drains. These systems reduce vertical movements of building foundations by moderating the effects of seasonal moisture changes.

6.1.1.1 Site Grading

Site grading causes excess water to flow away from the foundation via surface sloping and drainage swales. Adequate surface drainage slopes are essential to minimize foundation movement and damage. Current International Residential Code requires 6" minimum fall the first 10' out from and perpendicular to building walls, and 2% minimum elsewhere to drain off lot. Because current building practices sometimes have homes built closer than 10’ to the adjacent structure or lot line, it is necessary to have greater slopes so that the 6” minimum is maintained.

TABLE 6.1.1 **SITE GRADING**		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Fill materials are readily available. 2. Less maintenance required afterwards. 3. Inadequate drainage is easier to detect. 4. Most economical.	1. Improper grading can cause water to shed to adjacent properties. 2. Fill materials require proper compaction and material.	1. Materials should consist primarily of clay. Do not use bank sand or clayey sand or silts. 2. Can require excavation in addition to fill.

* Compared to other types of site drainage systems described in Sections 6.1.1.2 and 6.1.1.3.

6.1.1.2 French Drains

French drains are subsurface drainage systems that are used around the perimeter of a foundation to remove free water in the subsoil.

TABLE 6.1.1.2	**SUBSURFACE DRAINAGE SYSTEMS (FRENCH DRAINS)**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Helps reduce moisture infiltration from underground water sources. 2. Suitable when site grading is not an available option.	1. If not draining effectively, can recharge the surrounding soil with moisture, causing damage to the foundation. 2. Not very effective in expansive clay soils because soil suction is high when soil permeability is very low. 3. Can cause erosion of surrounding soil, causing settlement of foundation if too close. 4. Requires some maintenance but is difficult to monitor. 5. French drain fabric membranes can tear or be punctured. 6. If used in conjunction with a vertical moisture retarder on one side, sufficient geotechnical or geophysical testing is required to determine natural flow direction and source of groundwater. 7. Most expensive system.	1. Must have functioning outfall to storm drain system. 2. Commonly used when site is too flat to accommodate proper grade slopes. 3. Usually consists of a 4” or larger PVC perforated pipe, covered with sand and gravel and sloped to a positive outlet, but must comply with International Residential Code. 4. Utilized in removing moisture behind retaining and basement walls.

* Compared to other types of drainage systems described in Sections 6.1.1.1 and 6.1.1.3.

6.1.1.3 Area Drains

Area drains (catch basins) with non-perforated pipe are surface collection systems used around the perimeter of a foundation to remove surface water by gravity flow or mechanical lifting.

TABLE 6.1.1.3	**AREA DRAINS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Suitable when site grading is not an available option. 2. Water from downspouts can be discharged into area drainage system.	1. A clogged drain can cause localized flooding. 2. If used in conjunction with a vertical moisture retarder on one side, sufficient geotechnical or geophysical testing is required to determine to natural flow direction and source of surface water. 3. A leaking pipe can cause soils to erode or swell.	1. Commonly used in back yards when site is too flat to accommodate proper grade slopes. 2. Usually consists of a 4” or larger PVC non-perforated pipe. 3. When used, sump failure can cause flooding and can require periodic maintenance. 4. Can be used in conjunction with site grading. 5. A mechanical lift should be used if gravity outfall is not possible.

* Compared to other types of drainage systems described in Sections 6.1.1.1 and 6.1.1.2.

6.1.2 Moisture Retarder Systems

Moisture retarder systems are used to reduce moisture transfer to the soils underneath foundations. Such systems include horizontal moisture retarders and vertical moisture retarders. These systems help moderate effects of seasonal changes on foundation movements.

6.1.2.1 Horizontal Moisture Retarders

Horizontal moisture retarders usually consist of materials of low permeability. These systems extend outward around the edges of the foundation. Sidewalks, driveways, or parking lots can be multifunctional, also serving as moisture retarders.

TABLE 6.1.2.1 HORIZONTAL MOISTURE RETARDERS		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Readily inspected and maintained 2. Requires less slope than green space (e.g., only 1/8" / ft.) to achieve positive drainage away from foundation. 3. Can double as pavement for parking or sidewalk.	1. Requires larger area to be effective. 2. Heaving of retarder can occur due to soil hydration. 3. Fabric membrane retarders more prone to tear or puncture. 4. Unsecured rigid retarders can "walk-away" from building and allow water infiltration through gap created at edge of building if seal is not maintained. 5. Owner or future owner can remove it, not realizing it serves a design purpose, and inadvertently eliminate benefit it provides. 6. Only retards surface moisture. 7. May not be aesthetically acceptable.	1. Usually concrete or asphalt pavement. 2. Horizontal moisture retarders slow root growth by reducing oxygen transmission to roots. 3. Doubles as root retarder.

* Compared to other type of moisture retarder system described in Section 6.1.2.2.

6.1.2.2 Vertical Moisture Retarders

Vertical moisture retarders usually consist of materials of low permeability that extend downward from grade level around the perimeter of the foundation.

TABLE 6.1.2.2 VERTICAL MOISTURE RETARDERS		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. More effective in retarding lateral moisture migration. 2. Controls vertical movements better. 3. Also functions as a root retarder.	1. Higher cost. 2. Requires severing tree roots and can compromise tree health. 3. Is difficult to inspect and to know when to repair or replace. 4. Fabric membrane retarders more prone to tear or puncture. 5. Can retain moisture from under slab leaks and exacerbate heaving.	1. Usually consists of concrete, steel, polyethylene or fabric sheets, or bentonite clay.

* Compared to other type of moisture retarder system described in Section 6.1.2.1.

6.1.3 Watering Systems

Watering systems are usually used to induce moisture into the soils and to water vegetation around the foundation, thereby attempting to provide a constant and uniform moisture condition. During droughts, water can be rationed, preventing use of these systems. A soil moisture sensor with automatic controls is recommended with these watering systems.

6.1.3.1 Sprinkler Systems

An irrigation system consists of below grade piping and above grade sprinkler heads.

TABLE 6.1.3.1	**SPRINKLER SYSTEMS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Can attract tree roots away from foundations if system is properly drained, zoned, and set. 2. Provides moisture during dry periods, thereby reducing movement due to soil shrinkage. 3. Can provide more uniform moisture content to site.	1. Can cause uneven soil moisture, moving and damaging the foundation. 2. If not properly monitored, can result in excess watering, resulting in heave or loss of soil bearing. 3. Leaks may not be detected thereby causing localized foundation movement. 4. Requires more maintenance than other watering systems. 5. Overspray onto superstructures can occur. 6. Results in more waste of water by runoff and evaporation.	1. Only enough irrigation should be applied to sustain vegetation, so that there is no ponding or algae buildup. 2. It is essential that drainage slopes comply with International Residential Code.

* Compared to other types of watering systems described in Sections 6.1.3.2, 6.1.3.3 and 6.1.3.4.

6.1.3.2 Soaker Hose Systems

Soaker hoses are permeable water conduits resembling garden hoses normally used to water localized areas.

TABLE 6.1.3.2	**SOAKER HOSE SYSTEMS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Properly maintained soaker hoses apply water more slowly to soil than sprinkler systems. 2. Can be used to provide moisture to vegetation. 3. Easiest to install.	1. Can cause excessive and uneven soil moisture, moving, and damaging the foundation. 2. Hoses can be subject to premature deterioration. 3. Sensitive to damage from freezing. 4. Can attract roots toward foundation if used around perimeter of foundation.	1. Normally limited to garden and foundation applications. 2. Can be buried to reduce evaporation and avoid damage from lawn equipment.

* Compared to other types of watering systems described in Sections 6.1.3.1, 6.1.3.3 and 6.1.3.4.

6.1.3.3 Under-Slab Watering Systems

Under-slab watering systems are installed under slabs to provide moisture directly below the foundation. These systems typically consist of a network of piping, wells, and moisture sensors, which are intended to function together to maintain a uniform level of moisture in the soil beneath the structure.

TABLE 6.1.3.3	**UNDER-SLAB WATERING SYSTEMS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Low to medium cost although high-cost systems are also available. 2. Minimizes evaporation.	1. Can cause excessive and uneven soil moisture, moving and damaging the foundation. 2. Requires strict monitoring and maintenance. 3. Can take a long time to stabilize vertical building movements. 4. Difficult to install underneath existing buildings. High-cost system requires cutting holes through slab-at-grade to install system. 5. Soil irregularities and discontinuities can limit effectiveness of system. 6. Dramatic subsidence could occur if system is disabled. 7. Attracts roots toward foundation, which can increase dependency on this system. 8. Monitoring program should be included that entails regular geotechnical testing and foundation level distortion surveys. 9. During droughts, water can be rationed, preventing its use. 10. Moisture sensors are subject to frequent replacement and performance can be unreliable. 11. Desired moisture content under slab is difficult to determine.	1. Low cost foundation watering systems that do not include a continual soil moisture content monitoring system that is connected to moisture release valves is considered an unacceptable system. 2. It is essential that drainage slopes comply with International Residential Code.

* Compared to other types of watering systems described in Sections 6.1.3.1, 6.1.3.2 and 6.1.3.4.

6.1.3.4 Drip Watering Systems

Drip irrigation slowly applies water to soil under low pressure through emitters, bubblers, or spray heads placed at each plant.

TABLE 6.1.3.4	**DRIP WATERING SYSTEMS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Offers increased watering efficiency and plant performance when compared to sprinkler irrigation. 2. Can be installed without excavation.	1. Can cause excessive and uneven soil moisture, moving and damaging the foundation. 2. Requires strict monitoring and maintenance. 3. Not permanent. 4. Frost sensitive. 5. Can attract roots toward foundation if irrigation is excessive near building.	1. Drip irrigation slowly applies water to soil under low pressure through emitters, bubblers, or spray heads placed at each plant. 2. Normally limited to garden and foundation applications. 3. A moisture meter is recommended with this type of system.

* Compared to other types of watering systems described in Sections 6.1.3.1, 6.1.3.2 and 6.1.3.3.

6.2 VEGETATION CONTROL SYSTEMS

Vegetation control systems mitigate damage by providing some control over the growth of roots that can penetrate into unwanted areas and cause shrinkage of foundation soils by means of water withdrawal through the transpiration process.

6.2.1 Root Retarder Systems

Root retarder systems are typically physical or chemically induced barriers that limit the growth direction of the roots of trees, shrubbery, and other large plants.

6.2.1.1 Vertical Root Retarders

Vertical root retarders are vertical barriers that are installed in the ground adjacent to the perimeter of a foundation or around a tree or other large plant.

TABLE 6.2.1.1 VERTICAL ROOT RETARDERS		
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Minimal disturbance of landscaping. 2. Impervious type root retarder also doubles as moisture retarder, which can prevent excess moisture resulting from drainage problems from reaching foundation.	1. Requires severing tree roots, necessitating tree pruning or other treatment when using near existing trees. 2. Can compromise tree health. 3. Can compromise tree stability. 4. Will only help to control vertical foundation movements, not stop it. 5. Limited warranty and limited period of effectiveness (for biocide systems). 6. Requires special details for penetration of building utility lines. 7. Very difficult to inspect and to know when to repair or replace. 8. Sometimes difficult or impractical to install below deepest lateral tree roots. 9. Usually aesthetically required to be installed completely below grade, which can allow roots to grow over retarder. 10. Impervious type of root retarder also acts as a moisture retarder, which can interrupt existing natural below-grade moisture movement due to soil suction or gradients. 11. Extensive geotechnical and geophysical testing can be required to ensure that installation of impervious retarders is not detrimental to foundation. 12. For cases where retarder is installed adjacent to structure, foundation support can be compromised in order to install retarder deep enough to be useful.	1. Can be an impervious type (e.g., plastic, concrete or sheet metal piling) or a biocide type. 2. Vertical root retarders slow root growth below a foundation by forcing roots to grow to a greater depth. 3. Should be professional installed to minimize root damage. 4. Non-impermeable retarders require more maintenance.

* Compared to other types of vegetation control systems described in Sections 6.2.1.2 and 6.2.2.

6.2.1.2 Horizontal Root Retarders

Horizontal root retarders are horizontal barriers that are installed on top of the ground adjacent to the perimeter of a foundation or around a tree or other large plant.

TABLE 6.2.1.2	**HORIZONTAL ROOT RETARDERS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Doubles as pavement for parking or sidewalks. 2. Its presence prevents planting of trees near foundation. 3. Doubles as a horizontal moisture retarder. 4. Quick to install. 5. Relative low cost.	1. Not as architecturally pleasing, especially for residences. 2. Needs steel reinforcement and expansion joints to prevent cracking due to ground movement. 3. Expansion joints and cracks must be sealed to retard oxygen transfer to soil. 4. Joint seals require maintenance, which is easily forgotten by Owner. 5. Owner or future owner can remove it, not realizing it serves a design purpose. 6. Tree roots can lift and break retarder causing vertical offsets.	1. Normally concrete pavement. Horizontal root retarders slow root growth by minimizing oxygen flow to roots. 2. May not be effective in preventing root growth.

* Compared to other types of vegetation control systems described in Sections 6.2.1.1 and 6.2.2.

6.2.2 Root Watering Wells

Root watering wells are installed near trees to provide moisture below grade. These systems typically consist of a drilled hole filled with coarse material. Piping can be inserted in the holes in order to maintain a clear path for water access.

TABLE 6.2.2	**ROOT WATERING WELLS**	
ADVANTAGES *	**DISADVANTAGES ***	**COMMENTS**
1. Minimal excavated material. 2. Minimal disturbance of landscaping. 3. Can be beneficial to health of trees by helping them establish roots at a greater depth. 4. Can help keep new tree roots from growing under buildings provided wells are installed away from foundation. 5. Less likely to damage existing tree roots during installation process.	1. Effectiveness of reducing moisture withdrawal from under building is questionable. 2. Beneficial effects can take a long time to materialize. 3. Require maintenance. 4. Require ongoing operating costs and water usage. 5. Can hit utility line or tree root when drilling. 6. Movement of water through unfractured clays is extremely slow. 7. Chlorinated water directly applied to deep roots can be detrimental to tree health.	1. If used, root watering wells should be installed on side of tree opposite foundation.

* Compared to other types of vegetation control systems described in Sections 6.2.1.1 and 6.2.1.2.

6.3 TREE AND PLANT SELECTION

When doing the initial site landscaping design, the proper selection of site vegetation with regard to tree and plant moisture requirements can directly affect future foundation performance. Vegetation selection can also be a deciding factor in the selection of other moisture and vegetation control system design options.

TABLE 6.3	**TREE AND PLANT SELECTION**	
ADVANTAGES	**DISADVANTAGES**	**COMMENTS**
1. Proper tree and plant selection can also be aesthetically pleasing. 2. Proper tree and plant selection can increase property value.	1. Limits available vegetation for landscaping design. 2. Owner or future owner can remove trees and shrubbery with low-water-requirements, not realizing they serve a design purpose.	1. An example of site vegetation control is Xeriscape landscaping, defined as quality landscaping that conserves water and protects environment, by using plants and trees with low-water requirements. 2. Selecting plants and trees with low-water requirements can reduce potential for problems caused by vegetation water demands. 3. Using vegetation with low water requirements means less run-off of irrigation water that can carry polluting fertilizers and pesticides to nearby streams or lakes, and less permeation of irrigation water into ground that can leach nutrients deep into soil away from vegetation and increase chances of polluting groundwater.

DISTRESS PHENOMENA OFTEN MISTAKENLY ATTRIBUTED TO FOUNDATION MOVEMENT

by
The Structural Committee

of
The Foundation Performance Association

www.foundationperformance.org

Houston, Texas

Document # FPA-SC-03-1

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1	1 May 2004	Issued for Website Publishing		

PREFACE

This document was first peer reviewed and published as Revision 0 on 1 December 2000. After obtaining additional feedback, it has been updated by the Structural Committee to Revision 1 and again been peer reviewed by the Foundation Performance Association (FPA).

This document is based on experience gathered by engineers in Houston, working primarily in the southeast Texas area. It is intended to be used in southeast Texas by homeowners, building owners, builders, foundation repair contractors, inspectors, engineers, architects, and others involved with structural inspection, forensic assessment, and/or monitoring of residential and other low-rise buildings. Information contained herein is provided as a guide only, and caution should be exercised when applying the information to other types of buildings or outside the geographic area for which it was intended.

This document is made freely available to the public through the Foundation Performance Association at www.foundationperformance.org so that engineers, inspectors, owners, builders, realtors, and other interested parties may have access to the information. To ensure this document remains as current as possible, it has been and will continue to be periodically updated under the same document number but with new revision numbers such as 2, 3, etc. If sufficient comments are received to warrant a revision, the Structural Committee will form a new subcommittee to revise and reissue this document in accordance with the FPA Technical Paper Peer Review Procedure. If the revised document successfully passes FPA peer review, it will be published on the FPA website and the previous revision will be deleted.

INTRODUCTION

Architectural and structural distress in residential and other low-rise buildings in areas of Texas having expansive soils is a common occurrence and may be the result of differential foundation movement. However, not all observed distress results from foundation movement. The purpose of this document is to provide the user with a tabulation of phenomena that may occur in buildings or their foundations that are sometimes incorrectly attributed to foundation movement. For each distress or otherwise negative phenomenon, an attempt was made to provide a more probable cause of the phenomenon. Possible reasons or events that may have led to the direct cause of distress are also provided along with repair recommendations.

NEGATIVE PHENOMENON	PROBABLE CAUSE OF PHENOMENON	POSSIBLE REASON FOR CAUSE	REPAIR RECOMMENDATION
1. Masonry cracks above long lintels (e.g., garage door). *a. Cracks at midspan.*	a. Lintel beam undersized or overloaded.	a. Gravity load is causing lintel to deflect excessively.	a. Reinforce or replace lintel, and patch cracks or replace brick.
b. Cracks at ends.	b. Insufficient bearing surface.	b. Excessive lintel deflection or masonry is crushing.	b. Reinforce or replace lintel, and patch cracks or replace brick.
c. Grout cracks at first brick of soldier course on either side.	c. Insufficient bond of mortar to brick.	c. Thermal stresses can cause initial bond to break.	c. Patch crack with non-shrink mortar or fine grout.
2. Vertical cracks in masonry running full height of wall with uniform crack width from top to bottom of wall and usually located near center of wall or openings.	Thermal contraction and expansion of masonry wall.	Insufficient number, spacing, or width of expansion joints provided when masonry was constructed, or expansion joints were made ineffective during construction by excessive mortar obstructing joints.	Saw cut vertical expansion joints into brick as required, and repair existing ineffective expansion joints.
3. Surface cracks in post-tensioned slabs prior to tendon stressing (typically perpendicular to long dimension).	Concrete shrinkage during curing process.	As water naturally evaporates out of slab, voids are left causing shrinkage cracks.	Cracks typically close after stressing. No repair is usually required.
4. Cracks, or buckling, at top of walls supporting sloping roofs.	Inadequate horizontal restraint at top of walls, causing walls to bow outward and drywall to separate as ridge settles and sloping roof members push wall laterally.	Inadequate design or construction, missing collar ties, or overloading of roof or attic.	Add collar ties or reinforce ridge and eliminate any overloading.
5. Spalled concrete at corner of foundations supporting brick walls.	Brick bonds to grade beam when wall thermally expands and contracts.	Lack of bond breaker between brick and brick ledge.	Spalled area can be cosmetically patched with a bonding agent and non-shrink grout.
6. Cracks along gambrel and other vaulted ceilings, usually on exterior walls.	Ceiling joists are supported by rafters, and rafters expand and contract more than ceiling joists due to temperature changes.	Ceiling joists and rafters may not be adequately nailed together or supported.	Provide flexible material, such as caulking, at drywall intersections of sloping roof and ceiling.
7. Sags and cracks in drywall ceiling.	a. Excessive deflection of beams and stud wall plates, causing drywall to crack.	a. Undersized wood beams, improper support of wood beams, or overloaded attic roof members.	a. Analyze wood support beams and reinforce as required. Investigate whether there is adequate support for beams.
	b. Inadequate nailing.	b. Improper construction or loss of nails due to popouts resulting from wood shrinkage.	b. Add nails.
8. Cracks and spalls in first story brick, when second story is wood-framed and second story exterior walls are not clad with brick.	Excessive gravity loads on brick resulting from inadequate support of second story.	Improper engineering, construction or overloading; second story may be supported by brick that may not have been designed to be load-bearing.	Adequately support second story walls, and eliminate any overloading.

NEGATIVE PHENOMENON	PROBABLE CAUSE OF PHENOMENON	POSSIBLE REASON FOR CAUSE	REPAIR RECOMMENDATION
9. *Ground floors are noticeably out of level, but there is no apparent evidence of distress.*	Slab may have been originally constructed out of level.	Formwork not level during construction.	Begin slab elevation monitoring program on a quarterly basis to confirm there is no abnormal movement.
10. *Nails popping out of walls and ceilings in structures.*	a. Shrinkage of wood members.	a. Wood members may have become wet during construction.	a. Wait approximately two years for wood shrinkage to subside, and then repair.
	b. Infiltration of moisture into wall or ceiling.	b. There may be a roof, wall, or plumbing leak.	b. Repair leaks.
	c. Structural framing movement.	c. Flexure due to wind.	c. Consult structural engineer.
11. *Crown molding separation in newly constructed structures.*	Shrinkage of wood members or caulking.	Wood members may have become wet during construction, and shrunk while drying.	Wait approximately two years for shrinkage to subside, and then repair.
12. *Drywall separation around tape seals and corner beads in newly constructed residences.*	a. Shrinkage of wood members.	a. Wood members may have became wet during construction, and shrunk while drying.	a. Wait approximately two years for shrinkage to subside, and then repair.
	b. Insufficient bonding of corner beads at joints.	b. Insufficient nailing of corner beads.	b. Add more nails or screws, and then refloat.
13. *Miscellaneous cracking in concrete foundation with no apparent distress in superstructure.*	a. Concrete may have impurities in cement matrix.	a. Deleterious chemical reactions may cause cracking of concrete.	a. Wait until cracking appears dormant, and then repair as needed.
	b. Concrete shrinkage during curing process.	b. As water naturally evaporates out of slab, voids are left causing shrinkage cracks.	b. Wait until cracking appears dormant, and then repair as needed.
	c. Surface shrinkage cracks due to excess water during foundation construction.	c. Over vibration during foundation construction with excessive bleeding of concrete.	c. Wait until cracking appears dormant, and then repair as needed.
	d. Crack around reinforcement.	d. Moisture penetrated concrete and caused rebar corrosive expansion, resulting in concrete spalling.	d. Wait until cracking appears dormant, and then repair as needed.
14. *Short horizontal brick cracks at window lintels.*	Lintel corrosion expanding and lifting brick.	Inadequate lintel corrosion protection.	Remove rust, clean, paint, and seal exposed lintel ends.
15. *Doors that bind at bottom, but are easy to open or close if lifted by knob.*	a. Door hinge is loose.	a. Screws are too small or too short for weight of door.	a. Tighten or replace hinge with longer screws.
	b. Door hinge is too small.	b. Hinge is too weak or not enough hinges.	b. Replace or add additional hinges.
	c. Door has sagged or racked.	c. Glue that binds door components together has deteriorated.	c. Replace door.
16. *Masonry cracks adjacent to garage door jamb 1-ft to 2-ft above slab, frequently with outward displacement of brick at crack, and frequently with misaligned garage door jamb.*	Veneer or jamb damaged by horizontal impact load.	Veneer or jamb hit by passing auto bumper.	Repair brickwork and wooden jamb.

NEGATIVE PHENOMENON	PROBABLE CAUSE OF PHENOMENON	POSSIBLE REASON FOR CAUSE	REPAIR RECOMMENDATION
17. Cracks in individual floor tiles, but cracks do not connect to other cracks in adjacent tiles and there are no mortar cracks.	Cracks are inherent in tile material.	Cracks passed inspection during tile manufacturing process or tile was damaged by installation or fallen object.	Replace tile if not cosmetically acceptable.
18. Continuous cracks in grout lines and/or hard surface floor tiles in newly constructed residences.	a. Concrete continued to shrink during curing process after floor tiles were installed, leaving small cosmetic hairline cracks.	a. Cracks due to continued concrete shrinkage cause reflective cracking in tiles that are bonded to concrete floors.	a. Wait until cracking appears dormant, and then replace tiles (typically 1 to 2 yrs). When replacing tiles, provide an elastomeric membrane between tiles and concrete that prevents bonding.
	b. Internal overstress in tile resulting from inadequate gap between edge of tile and wall, cabinets, bathtubs, or other fixtures that are installed before tile is set.	b. Expansion of tile or shrinkage of concrete substrate may lead to compressive stress buildup in tile if edges of tile are restrained from horizontal movement. Inadequate clearance may be due to improper design or construction.	b. Remove grout adjacent to walls and fill with caulk.
19. Doors that open or close on their own (ghost doors).	Door is not plumb.	Poor installation.	Slightly bend hinge pin, or add shims to realign.
20. Doors do not latch.	Keeper is out of alignment.	Poor installation.	Move keeper.
21. Dips or humps in floor system at second floor.	Floor system was not constructed level.	Floor system was constructed out of level or has deflected excessively due to overloading.	Consult structural engineer, and eliminate any overloading.
22. Buckled trim boards around window sills and other areas where moisture can occur.	Trim boards became wet and swelled.	Materials used are simulated wood products sensitive to moisture changes.	Repair or replace with wood or other product less sensitive to swelling.
23. Wrinkling in vinyl floor sheathing.	Inadequate bonding.	Inadequate adhesive, floor preparation, and/or water migration.	Remove and replace vinyl flooring and eliminate any moisture migration.
24. Loose carpet seams.	Inadequate carpet installation.	Adhesive not sticking, carpet cross-laid, and/or bad edge cutting.	Repair carpet.
25. Wrinkled carpet.	a. Carpet became wet.	a. Carpet stretched when wet.	a. Dry and re-stretch carpet.
	b. Carpet is loose.	b. Carpet is not adequately attached to tack strips.	b. Re-stretch and secure carpet.
26. Buckles in wood floor planks.	Wood planks became wet and swelled.	Wood products are sensitive to moisture changes.	Eliminate moisture migration.
27. Cracking and buckling of drywall around HVAC vents.	Local overstress due to thermal expansion and contraction.	Difference in expansion and contraction characteristics of drywall and metal.	Allow for thermal expansion between HVAC vent and drywall.

RECOMMENDED PRACTICE

FOR

GEOTECHNICAL EXPLORATIONS AND REPORTS

by
The Structural Committee
of
The Foundation Performance Association
Houston, Texas

Document # FPA-SC-04-0

ISSUE HISTORY (Only includes issues outside the Structural Committee)

<table>
<tr><th>Rev #</th><th>Date</th><th>Description</th><th>Subcommittee Chair</th><th>Subcommittee Members</th></tr>
<tr><td>D</td><td>19 Jul 00</td><td>Issued for Comments to FPA Geotechnical Engineers</td><td rowspan="5">Ron Kelm</td><td rowspan="5">Jon Monteith
George Wozny
Michael Skoller
Moyeen Haque</td></tr>
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<tr><td>H</td><td>4 Jan 01</td><td>Re-Issued for Comments to Local Geotechnical Engineers</td></tr>
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PREFACE

This document has been developed by a group of Structural Engineers in southeast Texas with the goal to attain the geotechnical information they believe is necessary to adequately perform their structural designs. Their need for this document has been prompted by a large number of residential and light commercial foundation problems, some of which have been the subject of litigation. As a result, this document has been prepared specifically for the Structural Engineers' use. However, it is made freely available to the public through the Foundation Performance Association at www.foundationperformance.org so others may have access to the information and may adapt it to their work as they see fit. To ensure the document remains as current as possible, it is intended to be periodically updated under the same document number but with new revision numbers.

This document is a recommended practice only and is therefore intended to be neither comprehensive nor a substitute for engineering judgment or for local or standard codes and practices. The user should recognize that there is always the possibility this recommended practice might not be fully adaptable to the site being investigated and in those cases, the use of engineering judgment will be paramount. The intent of this document is to detail certain minimum requirements recommended for the geotechnical exploration and report, thereby ensuring that the Structural Engineer receives the information needed to perform an adequate foundation design. Thus, Geotechnical Engineers preparing proposals for a geotechnical exploration and report in accordance with this recommended practice must all follow certain minimum proposal requirements, which can help ensure a more uniform selection process during the procurement of their services.

In requiring the use of this recommended practice, the participating Structural Engineers understand that the request for the information specified herein would most likely increase the cost of the geotechnical work, since there is no intent to delete any of the work currently being executed in the geotechnical industry. However, they should also realize that this additional cost is necessary in order for them to better understand the soil characteristics of the site on which they plan to design a foundation.

When using this recommended practice, it is expected that the Client will provide a description of the foundations and structures proposed for the site. In addition, the Client should provide site plans that show the foundation outline(s), the foundation location(s), and the location and species of any trees that are planned to be removed and that have trunk diameters equal to or greater than 6 inches. If the lots are Wooded Lots, it is intended that the Client will provide a tree survey to the Geotechnical Engineer, showing the location, sizes, species and condition of the trees on each lot. The Client should not discount this requirement as something less than a necessity. Though not recognized locally to be a problem as recently as ten years ago, trees in this area are now known or at least suspected to be the main contributor in the majority of foundation problems in the local market. Therefore the recommendations addressing trees should not be taken lightly.

This recommended practice addresses a geotechnical report prepared specifically for foundation design and construction. In new subdivisions, the type of geotechnical report that addresses the streets and utilities is not acceptable as a substitute for the work specified herein. Preferably, the borings for a new subdivision should be taken after the streets are cut and the lot's fill is compacted. If however, the geotechnical exploration is made before the streets are cut, then it is the intent of this recommended practice that a separate exploration will later be procured in order to verify the required density, moisture content, and Atterberg limits for the fill material.

This recommended practice is written specifically for use in Houston and the general southeast area of Texas. Therefore, it should be used with caution if utilized elsewhere or if adapted for foundations other than those supporting residential or light commercial structures.

The main purpose of this recommended practice is to bring certain minimum requirements together into one document for local Structural Engineers to use in part or in whole, as they see fit. It is not meant to imply that problems will not occur if geotechnical explorations and reports comply in part or in whole with this recommended practice. The Foundation Performance Association and its members make no warranty regarding the recommendations contained herein and will not be liable for any damages, including consequential damages resulting from the use of this document.

DEFINITIONS

For the purpose of this document, the following definitions apply:

Builder – The general contractor responsible for performing the construction of the foundation, including the site work.

Client – The person or company using this recommended practice in the procurement of the geotechnical exploration and report.

Geotechnical Engineer – The engineer or engineering firm responsible for performing the geotechnical exploration and for providing a report of the results.

Structural Engineer – The engineer or engineering firm responsible for performing the structural design of the foundation.

Wooded Lot – A lot that contains at least one tree per thousand square feet (1 per 1000 SF) of lot area, with those trees having trunk diameters greater than or equal to 6 inches. Note that the trunk diameter measurement is intended to be made at approximately chest-height above the ground level. Although the proper term for tree stem diameter in arboriculture is "caliper", that term is purposely not used herein because it is sometimes confused with "circumference" when measuring trees.

TABLE OF CONTENTS

1.0 MINIMUM CRITERIA

The following subsections outline the Geotechnical Engineer's minimum requirements in accordance with this recommended practice.

1.1 General

The Geotechnical Engineer should provide the Structural Engineer and Builder with sufficient information to enable them to: (a) design a structural foundation that is appropriate for the site conditions and is capable of adequately supporting the given building design loads, (b) provide a safe foundation design that meets local code and professional standards, and (c) carry out the site work and foundation construction in a safe and efficient manner. The Geotechnical Engineer should advise the Client if any requirements herein are in direct conflict with local codes and professional standards. In addition, the Geotechnical Engineer should carry out the work and prepare the report with the assumption that the Structural Engineer will never actually see the site for which the foundation will be designed.

1.2 Site Exploration

It is the Geotechnical Engineer's responsibility to investigate the site as required to comply with local codes, professional standards and the requirements of this recommended practice. In addition, the site exploration should include the following minimum criteria:

1.2.1 Area Reconnaissance

The Geotechnical Engineer should check the area of the building site and the area surrounding the site for any anomalies such as streams, ponds, fill, dumps, existing above-grade structures, escarpments, slopes, poor draining areas, seeps, outcrops, large trees, tree stumps, erosion, structures, roadways, railways, areas that appear to be wetlands, or anything else that will help the Builder and Structural Engineer understand the prior and present land use of and around the site. Representative color photos of the site should be recorded at the time of the site exploration and should be included in the final report. Where possible, the photos should include portions of the properties adjacent to the site. All anomalies, including trees and tree stumps with trunks equal to or greater than 12 inches diameter should be located on the boring plan. On Wooded Lots the Geotechnical Engineer should superimpose the data from the Client-supplied tree survey on his plans.

1.2.2 Boring Quantities and Locations

a. For new residential subdivisions that are anticipated to have grade-supported foundations, at least one boring per 5 lots is recommended, but not less than one boring every two acres.
b. For new residential subdivisions that are anticipated to have pier-supported foundations, at least one boring per lot is recommended.
c. For individual residential lots or for properties with light-commercial buildings, one boring for every 2,500 square feet of building ground floor slab, but a minimum of two

borings for the lot. The borings should be taken inside the projected building perimeter or as close as possible, if obstructions exist.

d. For lots with predominately cohesive soils and with trees growing within 25 feet (even if located on adjacent property) of the proposed foundation, one boring should be taken within 10 feet, or as close as possible, of the largest tree having a trunk size equal to or greater than 12 inches diameter, even if this dictates an additional boring for the lot.
e. For additions of less than 1000 square feet, one boring is adequate, except that the boring should be taken within the proposed foundation area and the recommendations in Paragraph 1.2.2d are still applicable.

1.2.3 Boring Depths

Boring depths below are measured from the grade existing at the time of the site exploration.

a. The minimum depth of every boring should be 20 feet. However, if the upper 10 feet are predominately cohesionless, then the minimum depth may be reduced to 15 feet.
b. On Wooded Lots, or lots containing one or more trees with trunks equal to or greater than 12 inches diameter, and if these lots contain predominantly cohesive soils, borings should be a minimum of 25 feet depth.
c. For sloped lots, where the proposed foundation is to be situated at a height H above the toe of an embankment (or estimated toe if submerged), if the horizontal distance from the foundation to the toe is less than 4H, then the depth of borings recommended should be a minimum of 2H.

1.2.4 Sampling Frequency

a. Undisturbed samples should be taken at a minimum of 1-ft, 2-ft, 4-ft, 6-ft, 8-ft, 10-ft, 12-ft, 16-ft, 20-ft depths, and thereafter at a maximum of 5-ft intervals.
b. If fill is known to have been placed on the site, sampling frequency should be increased to one sample per foot in the fill regions.
c. A sample should also be taken at the bottom of the borehole.

1.2.5 Field Testing and Logging

a. Each sample should be visually classified and logged during retrieval.
b. Existence and depth of roots should be noted.
c. Hand penetrometer testing should be done and reported on all cohesive samples.
d. Standard penetration testing should be done and reported on all cohesionless samples.
e. After the borehole is complete, measurements of the free water surface should be made and logged at completion of the borehole, and then again upon completion of the sitework, with that time interval being reported.

1.3 Laboratory Testing

The Geotechnical Engineer should perform sufficient laboratory testing to comply with the requirements of standard codes and local practices. However, the following laboratory testing is recommended as a minimum:

a. Existence and depth of roots or root fibers should be observed and reported for each soil sample.
b. Moisture contents should be performed on all samples retrieved.
c. In cohesive soils, Atterberg testing should be performed on a minimum of one third of the samples retrieved, with emphasis towards the upper strata. Reports should include plastic limits, liquid limits, and plasticity indices.
d. In cohesive soils, the percentage of clay (minus 2 microns) should be tested using a hydrometer.
e. Cohesive samples at the recommended foundation bearing depths may be tested using the torvane testing device provided a baseline test is made using unconfined compression tests for comparison.
f. For lots that have predominately cohesive soils, testing at each boring location should be done to determine soil suction. Soil suction tests should be conducted using the transistor psychrometer method, the filter paper method or other methods that give similarly reliable suction values. Where suction testing is recommended, it should be done at sample depths of 2-ft, 4-ft, 6-ft, 8-ft, 10-ft, 12-ft, 16-ft and 20-ft, but may be terminated earlier at the depth of constant suction, if determined.

1.4 Reporting

The Geotechnical Engineers may use their own standard reporting techniques. However, the final report should also contain the following where applicable:

a. A statement confirming that the geotechnical exploration and report are in accordance with the requirements of this recommended practice. If any exceptions are taken, the report should note each exception and the reason for taking the exception.
b. A general description of the site and surrounding properties, specifically addressing the anomalies as discussed in Paragraph 1.2.1.
c. Color photos of the proposed building site and where possible, adjacent properties. A minimum of two photos is recommended, but the total number of photos should at least be equal to the total number of borings.
d. A plot plan showing the approximate location of borings, tree trunks equal to or greater than 12 inches diameter and all anomalies as described in Paragraph 1.2.1. In the case of trees, include species where known, the condition of the tree if not healthy (i.e., “dying,” or “dead”) and show trunk diameters, measured at chest-height. If the site personnel are unable to identify the tree species, then an attempt should be made to classify them into categories that help the user to estimate the potential water usage of the tree. For example a tree could be classified as either a hardwood or pine. Alternatively, it could be classified either as a broadleaf or conifer.
e. Boring logs that include all field and laboratory tests results, unless particular data is presented on other charts or tables.
f. Descriptions and classification of the materials encountered.
g. Elevation of the water table, if encountered. If no water was encountered, the report should state that the holes were “dry”.
h. Provisions to mitigate the effects of expansive soils.
i. Recommendations on earthwork stabilization requirements (including requirements for slope stability) needed to prepare the site before the foundation can be constructed.

j. A discussion on foundation maintenance required in order to maintain the design.
k. Combined (i.e., for the various borings) plots of moisture content profiles and plastic limit profiles vs. boring depths.
l. Plots or tables showing the percentage of clay in cohesive samples as determined from hydrometer testing.
m. A discussion of the degree of saturation or desiccation of the site as compared to the estimated equilibrium moisture contents of the samples. This can be presented graphically depending on the method used (e.g., a graph of moisture content minus plastic limit vs. depth).
n. Combined (i.e., for the various borings) plots of suction values vs. boring depths.
o. Interpreted output from the suction testing including the moisture active depth, the movement active depth, the edge moisture variation distance and the probable vertical movement, both up and down, of the ground surface.
p. Specific discussion of trees to be removed before construction and of trees that are to remain after construction is complete, if known.

2.0 SPECIAL REQUIREMENTS

In addition to the general minimum criteria discussed above, there are some specific requirements that may be applicable to the Geotechnical Engineer, depending on the Client's needs. These requirements are as follows:

2.1 Slab-On-Grade Design Parameters

Regardless of the type of building foundation planned, if design recommendations are provided for slab-on-grade foundations, then the Geotechnical Engineer should provide recommendations as outlined in both (a) WRI's "Design of Slab-on-Ground Foundations," latest edition and (b) PTI's "Design and Construction of Post-Tensioned Slabs-On-Ground," latest edition.

2.2 Drilled Piers Design Parameters

If design recommendations are requested or made for drilled piers such as those recommended for slab-on-piers (at grade), suspended (structural) slabs, or structural floor-on-piers (i.e., with a crawl space) foundations, then the Geotechnical Engineer should provide recommendations for pier depth that takes into account possible upward and lateral movements as well as the normal downward movement due to gravity loads. Upward movement should be addressed if the soil is predominately cohesive. Lateral movement should be addressed if the site has pronounced slopes or if substantial fill is planned or has already been placed. In addition, the Geotechnical Engineer should also provide similar design recommendations as specified in Paragraph 2.1. Further, allowable design loads and recommended depths should be provided for both (a) drilled and under-reamed piers and (b) drilled straight shaft (skin friction) piers, in order to give the Client an opportunity to perform or obtain a cost/benefit study.

2.3 Suspended Slab Design Parameters

If design recommendations are requested or made for the entire slab to be suspended above the soil using a structural slab system, the report should prescribe the recommended void box height. In this case, the report should also advise the maximum possible heave the surface of the soil could experience if the site is exposed to an unlimited source of moisture.

2.4 Select Fill Parameters

On sites that require select fill be added to reduce the expansiveness of the in-situ soil, the Geotechnical Engineer should provide several options (e.g., different thickness of select fill/natural soil removal combinations versus potential heave/subsidence) that will allow the Client an opportunity to perform or obtain a cost/benefit analysis.

FOUNDATION MAINTENANCE AND INSPECTION GUIDE FOR RESIDENTIAL AND OTHER LOW-RISE BUILDINGS

by

The Structural Committee

of

The Foundation Performance Association

www.foundationperformance.org

Houston, Texas

Document # FPA-SC-07-0

ISSUE HISTORY

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<tr><td>A</td><td>20 Jul 01</td><td>For Committee Comments</td><td rowspan="7">James Austin</td><td rowspan="7">John Clark
Ron Kelm
Bill Polhemus
Taylor Sealy
Michael Skoller
Jack Spivey
George Wozny</td></tr>
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<tr><td>E</td><td>3 Jan 03</td><td>For Committee Comments</td></tr>
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PREFACE

This document has been developed by a group of foundation design engineers in Southeast Texas with the goal to educate homeowners, low-rise building owners, and tenants of their duties to maintain their foundations. Foundations, like other parts of a home or building, require a certain amount of maintenance to avoid premature deterioration.

The need for this document has been prompted by a large number of residential and low-rise building foundation problems in Southeast Texas, some of which might have been avoided had owners and tenants properly understood the foundation engineer's design basis and provided the required foundation maintenance. As a result, this document has been prepared and made freely available to the public through the Foundation Performance Association at www.foundationperformance.org so that owners, tenants, realtors, builders, inspectors, engineers, architects, repair contractors, and others involved with residential and other low-rise building foundations may benefit from the information it contains.

A properly engineered foundation will be designed to support its superstructure and buffer it from ground movements and other loadings so that the structural and architectural components do not show significant distress. As with any engineering design, there will be design assumptions made which limit the scope of design so that a foundation can be constructed which is not only strong, meeting the requirements of the Building Code and accepted standard engineering practice, but is also affordable.

The foundation design engineers will want building owners to be satisfied with the performance of the foundations for their buildings. However, building owners must be practical about the nature of the area in which they have built. They must expect and accept a certain amount of foundation movement. With proper maintenance, this movement can be minimized. This document details the responsibilities that foundation design engineers need and expect from the owners and tenants. By following the recommendations contained within this document, the maintainer can greatly increase the probability that the foundation will perform as originally designed.

This document was written specifically for use in the southeast region of the state of Texas and primarily within the City of Houston and the surrounding metropolitan area. Therefore, it should be used with caution if used elsewhere, or if adapted for foundations other than those supporting residential or low-rise structures. The Foundation Performance Association and its members make no warranty regarding the information contained herein and will not be liable for any damages, including consequential damages, resulting from the use of this document.

DEFINITIONS

For the purposes of this guide, certain terms are defined as follows:

Foundation is defined as a composite of soil, concrete, steel, wood, plastic, and other materials that are designed to work together to provide a stable base that supports a superstructure.

Superstructure is defined as the building components above the foundation such as the structural framing and the architectural coverings for the floor, walls, ceilings, and roof.

Foundation Design Engineer is defined as a licensed professional engineer that designs foundations (also called Engineer of Record).

Maintainer is defined as the person or group responsible for monitoring and maintaining the condition of the foundation, usually the owner or tenant.

Sand is defined as soil particles that are at least 0.06 millimeters but less than 2.0 millimeters in diameter. (Note: 1 millimeter = 0.039 inch)

Silt is defined as soil particles that are at least 0.002 millimeters (2 microns) but less than 0.06 millimeters in diameter.

Clay is defined as microscopic soil particles measuring less than 0.002 millimeters (2 microns) in diameter.

Non-Expansive is defined as a property of soil, indicating the soil particles have little potential to swell when moisture is absorbed by them and little potential to shrink when moisture is extracted from them.

Expansive is defined as a property of soil, usually clay, indicating the soil particles have a potential to swell when moisture is absorbed by them and to shrink when moisture is extracted from them. The shrink-swell movements can be in all six directions but the directions of most concern in this guide are the vertical upward (heave) and the vertical downward (subsidence) movements, as defined below.

Settlement is defined as downward movement of underlying supporting soils under load, taking with it the foundation and superstructure, and is due to the immediate elastic compression and distortion of granular or clay soil particles, and the long-term consolidation resulting from gradual expulsion of pore water from voids between saturated clay soil particles. Settlement may occur in all types of soils.

Subsidence is defined as downward movement of underlying supporting expansive soils, taking with it the foundation and superstructure, and is due to the extraction of moisture from the expansive soil particles, and consequently, shrinkage of the expansive soil particles.

Heave is defined as upward movement of underlying supporting expansive soils, taking with it the foundation and superstructure, and is due to the addition of moisture to the expansive soil particles, and consequently, swelling of the expansive soil particles.

TABLE OF CONTENTS

APPENDICES
(CHECKLISTS)

1.0 INTRODUCTION

This foundation maintenance guide is divided into several parts, summarized as follows. In Section 2, initial survey requirements are discussed. These initial survey requirements are targeted for use by the maintainer. The intent is that they will be implemented upon initial occupation, whether the structure is new or used. In Section 3, the regular maintenance requirements are summarized, and the requirements for maintenance of the foundation are detailed. In Section 4, a foundation design philosophy is presented, giving the user an understanding of typical assumptions made by the foundation design engineer to design a foundation. Finally, the appendices contain checklists for maintainers to print and use in surveying and maintaining their foundations.

2.0 INITIAL SURVEY REQUIREMENTS

2.1 General

Soon after the initial purchase of the structure is complete, the maintainer may use the checklists in Appendices A and B to determine what action may be required to help ensure that the original foundation design philosophy will not be compromised. The items in the checklists apply whether the purchase is for a new or a used structure.

If there are any signs of distress, the maintainer should employ an experienced consultant, such as a licensed structural inspector to determine if the distress is indicative of a foundation movement problem. It the structure is newly built, the original builder and or foundation design engineer of record should be contacted since they will already be familiar with the structure and usually have some responsibility to ensure its performance.

If the distress is found to be the result of foundation movement, the inspector/builder/engineer of record may recommend that a forensic engineer or other forensic consultant then be hired to investigate the cause of movement. The inspector/builder/engineer of record should also provide a level distortion survey of the foundation. Even if they do not find a cause for concern, the level distortion survey and other documentation provided will be a valuable baseline for a forensic engineer or other consultant to use in case there is foundation movement in the future.

If there are signs of distress for which the maintainer is unsure if they are attributable to foundation movement, it may be helpful to refer to the Foundation Performance Association's technical paper # FPA-SC-03, *Distress Phenomena often Mistaken for Foundation Movement*, freely available to the public at www.foundationperformance.org.

Since foundations on expansive soils have maintenance requirements that differ from or are additional to those for foundations on non-expansive soils, the maintainer should determine if the building is located in an area where expansive soils exist or are likely to exist. One sure way to make this determination is to engage a geotechnical engineer to sample and test the soils at the building site, but this method may cost more than the maintainer is willing to spend. A less expensive but less accurate method is to obtain the soil survey booklet from the U.S. Department of Agriculture for the county where the building is located. Alternately, the maintainer may contact the applicable City or County engineering office and question the building inspectors about their knowledge of expansive soils in the area.

2.2 Exterior Survey

Soon after the initial purchase of the structure is complete, the maintainer should make a reconnaissance-type survey of the site and exterior of the structure for evidence of problems affecting the foundation design. Photographs or other documentation should be made of any obvious or suspected distress or other anomalies observed. For later comparison purposes, it is also recommended to take photographs of all exterior perimeter walls to document their initial condition. A typical checklist that may be followed for this survey is shown in Appendix A.

2.3 Interior Survey

Soon after initial occupation, the maintainer should make a reconnaissance-type survey of the interior of the structure for evidence of problems that are indicative of past unusual movement of the foundation. Photographs or other documentation should be made of any obvious or suspected distress or other anomalies observed. A typical checklist that may be followed for this survey is shown in Appendix B.

3.0 REGULAR MAINTENANCE REQUIREMENTS

3.1 General

During the entire period of ownership, the maintainer should regularly provide maintenance in accordance with the checklists in Appendices C and D to help ensure that the foundation performance and original foundation design philosophy will not be compromised. An interval of twice per year is recommended to monitor the site and structure for required maintenance. The best time is after extreme dry or wet weather periods.

During the surveys outlined in the following subsections, the maintainer should review initial survey photographs and other documentation for changes that have occurred. If there are any new signs of obvious or suspected distress, the maintainer should, after first referring to technical document # FPA-SC-03 (see Section 2.1), seek the services of a forensic engineer or other forensic consultant to determine the cause of distress.

3.2 Exterior Maintenance

Approximately every six months, the maintainer should make a reconnaissance-type survey of the site and exterior of the structure for evidence of new or reoccurring problems affecting the foundation performance. More photographs should be taken or other documentation should be made of any new obvious or suspected distress or other new anomalies that are observed. A checklist that may be used for this survey is shown in Appendix C.

3.3 Interior Maintenance

Approximately every six months, the maintainer should make a reconnaissance-type survey of the interior of the structure for evidence of new or reoccurring problems affecting the foundation performance. If any new phenomena have developed, do not repair them without first having a forensic engineer or a forensic consultant investigate the cause of the distress. More photographs or other documentation should be made of any new distress observed. A checklist that may be followed for this survey is shown in Appendix D.

4.0 FOUNDATION DESIGN CONSIDERATIONS

4.1 General

Depending on its type, location, and the year in which it was constructed, the foundation for the home or low-rise building may have been engineered by a professional engineer who was licensed by the state. In the cases where this was done, the foundation design engineer would most probably have required that a soil exploration be carried out and reported by a geotechnical engineer, also licensed by the state. However, even if it was not an engineered foundation, this section should still give the user insight to the typical foundation design philosophy so that the reasons for providing the maintenance discussed above may become apparent.

4.2 Foundation Types

The foundation's primary function is to provide a stable support for the superstructure, keeping superstructure distress to a minimum. A description of the types of foundations commonly used in the Southeast Texas area follows:

A. **Slab-on-Grade**: This foundation type is the most commonly used and consists of cast-in-place concrete slabs stiffened with grade beams and supported by the surface soils (includes both post-tensioned and conventionally reinforced foundations).

B. **Slab-on-Piers**: This foundation is similar to Slab-on-Grade with a soil-supported slab, except the grade beams are supported on drilled cast-in-place concrete piers. The grade beams are designed to span between the piers, but the slab is normally not designed to span between the grade beams.

C. **Structural Slab on Piers**: This foundation consists of cast-in-place reinforced concrete slabs and beams supported on drilled cast-in-place concrete piers, and that is *not* supported by, but rather usually spans a few inches above, the surface soils.

D. **Pier-and-Beam**: The first floor framing system for this type of foundation is built well above the ground, creating what is commonly called a "crawl space". Typically, there is no concrete slab in this system, and the first floor is framed with wood. The "beam" is part of the framed first floor and spans from pier to pier. The "pier" in this case is an aboveground support, typically cast-in-place concrete or reinforced masonry columns. The aboveground piers are in turn typically supported on drilled cast-in-place concrete piers or cast-in-place concrete spread footings.

   Following are some variations of Pier-and-Beam:

   1) **Crawl Space on Grade Beam**: This foundation system consists of a framed first floor supported on framed walls, which are supported on cast-in-place concrete grade beams.

   2) **Crawl Space on Piers**: This foundation system is similar to Crawl Space on Grade Beams, except the grade beams are in turn supported on drilled and cast-in-place concrete piers.

For a more complete listing of foundation types and their various components, and the reasons for using or not using each, please see the Foundation Performance Association's technical paper # FPA-SC-01, *Foundation Design Options for Residential and Other Low-Rise Buildings on Expansive Soils*, at www.foundationperformance.org (scheduled to be published and freely available late 2003).

4.3 Superstructure Types

The superstructure or framing scheme is the "skeleton" that supports the building enclosure and finishes. The most common types of superstructures used in the area are:

A. **Beam and Column**: This framing system concentrates the building weight to the foundation in small areas or "points" located at the bases of the columns. Typically, a drilled and underreamed cast-in-place concrete pier or cast-in-place concrete spread footing is placed at this point load. The rest of the foundation carries less load than do these points.

B. **Joist and Wall**: This framing system spreads the building weight more uniformly on the foundation, and this affects the location of grade beams and piers or spread footings. The walls may be wood stud, cold-formed steel stud, masonry, or concrete.

4.4 Architectural Coverings

Different types of finishes respond differently to movement. Brittle surfaces, such as stucco, masonry, gypsum board (drywall), glass, and tile, cannot tolerate as much movement as flexible surfaces, such as carpet, wood, and vinyl. This affects the design of the superstructure as well as the foundation.

4.5 Soil Types

Of the various materials making up a foundation, the soil is the one of most concern. Because the soil has the ability to expand, contract or settle, it can load the superstructure as well as support it. The area contains various soil types, which include sand, silt, and clay. Much of the clays in the Houston area, especially those south of Interstate 10, are expansive. Movements in these soils may result in loss of support or the exertion of tremendous upward pressure on foundations, causing unsightly distress to the superstructure.

Unless they have been penetrated by tree roots or have widely disseminated fissures and cracks due to desiccation, the clays in the area often have low permeability, and when found a few feet below a granular surface material such as silt and sand, they can cause perched water tables to occur. When this happens, water is trapped in the silts and sands above. This saturation can cause the soil to lose much of its bearing capacity, consequently causing settlement of the foundation and superstructure.

The area also has active faults, which can cause severe foundation and superstructure distress. However, sites with active faults are usually obvious and well defined, so that foundations today are rarely constructed over them. Many other conditions exist that cause foundation movement, such as instability of sloping soil near a bayou or creek, or improper compaction of fill. Their discussion is beyond the scope of this document.

4.6 Soil Moisture

If expansive, the foundation support soils expand and contract due to changes in moisture content. Changes in moisture content can cause very large changes in soil volume when going from a dry to a saturated condition, and vice versa. This movement does not mean the foundation is improperly designed or that it has failed. The foundation design engineer cannot control the moisture content of the soil, but often the owner/tenant can. Uniformity is the key: uniform moisture content in the soil, uniformly maintained in all areas around the foundation.

If changes in moisture content are uniform, then movement of the foundation will be uniform and less distress will be created in the structure. If changes in moisture content are non-uniform, then there may be differential movement in the foundation. Differential movement can cause greater (and more obvious) distress in the structure.

Leaking pools, leaking plumbing lines, leaking drains, dripping faucets, dripping air conditioning condensate lines, and misdirected water from clogged and broken gutters and downspouts can cause local high moisture contents that can result in differential movement in areas of expansive soils. These conditions should be remedied as soon as possible.

Trees in or near the footprint of the foundation, either removed or planted during construction, cause the majority of foundation problems requiring repair in this area. Trees removed during construction tend to cause heave of expansive soils during the first few years, with initial distress often evident at the time of move-in. Trees planted during or after construction tend to cause subsidence of expansive soils. However, significant subsidence distress will usually not occur for ten to twenty years as the trees mature.

4.7 Site Drainage

It is extremely important, particularly in areas of expansive soils, that water drains away from the foundation and not be allowed to pond against or near the foundation. The soil around the foundation should be graded to an obvious slope (two to five percent). Fill in any low spots with select fill (sandy clay) and level off any high spots.

4.8 Site Vegetation

Avoid the use of metal edging or other damming devices within five feet of the foundation, particularly if the soils are expansive. The roots of trees and large plants remove large quantities of water from the soil. If these trees and shrubs are near the foundation and if sufficient water is not supplied, the soils may shrink if expansive, causing subsidence in the foundation. During dry periods, enough water should be supplied to trees to minimize shrinking of expansive soils around them. Most of the irrigation water should be applied well away from the foundation to attract the tree roots in that direction. When trees mature to the point of shading the entire lot, regular pruning will be needed to reduce their water uptake.

Landscaping (plants, shrubs, flowers, etc.) should not trap water against the foundation. Provide a slope in soils below landscape bedding and in the bedding away from the foundation. Alternatively, provide swales around and through the landscaping to drain water away. Provide uniform ground cover around the foundation. This will help keep the moisture evaporation rate uniform. In areas that are not planted, use mulch. Extend the ground cover at least five feet from the foundation.

4.9 Climate

During periods of dry weather, the soil around the foundation should be irrigated if the building is located in an area where expansive soils are known to occur. The most commonly used irrigation system is aboveground timed sprinklers with a manual override so they can be turned off in rainy weather. An automatic belowground irrigation system that senses the moisture content of the soil may also be used.

Tend to keep the irrigation system set on "manual", and only use it in drier periods when wilting of the lawn grasses and other vegetation occurs. The irrigation should be done at least one to two feet away from the foundation, and then lightly so that tree roots are not attracted there. Do not allow sprinklers to spray water against the structure. In extended dry periods, should the soil crack and pull away from the foundation, do not water directly into the gap.

4.10 Initial Soil Movement

Due to the changes in the environment and the load to the soils around a new foundation, the soils have to be allowed to adjust and reach a new equilibrium. This will result in some movement in the soils, foundation, and superstructure. The soils normally stabilize within the first one or two years after construction. However, this initial movement should not cause more than hairline cracking in the superstructure and is usually undetectable to the common building owner that is not looking for distress.

If more than hairline distress is observed, the maintainer should contact the builder/inspector/engineer of record to determine if the distress is due to abnormal foundation movement. If the observed movement is believed to be abnormal, then a forensic engineer or forensic consultant should be contacted to determine the cause of movement.

APPENDIX A - INITIAL EXTERIOR SURVEY

Date________

Category	Items to Check (at the time of purchase or move-in)	√
Cracks & Separations	Check that there are no cracks or separations in the walls if the structure is new.	
	Check that the observed cracks or separations are no more than hairline if the structure is used and is less than 10 years old.	
	Check that the observed wall cracks or separations are no more than 1/8" wide if the structure is more than 10 years old.	
	Check that vertical expansion joints in brick are uniform in width.	
Drainage	Check that water does not pool near the foundation after a heavy rain. If it does, bring in fill and re-grade or add an underground drainage system with area drains.	
	Check that the grade slopes away from the foundation at least 1 inch vertical per foot horizontally for the first 5 feet all around the perimeter (may be less where paving occurs). If necessary, revise the grade with sandy clay (not sand alone) fill or add underground drainage.	
	Check that where paving occurs near the structure, that it positively drains away from the foundation. If not, add underground drainage with area drains or re-pave.	
	Check that downspouts and gutters are clean and water from downspouts is directed away from the foundation.	
	Check that gutters and downspouts exist and that downspouts are tied directly into an underground drainage system or at least have aboveground extensions (e.g. flexible plastic pipe or long concrete splash block) to carry the water at least five to ten feet away from the building before it is allowed to run onto the soil. (Does not apply if the soil is known to be predominately non-expansive.)	
Vegetation	Check that there is no broadleaf tree (e.g., oak, ash, tallow, pecan, hackberry, etc.) closer to the foundation than a distance equal to the height of the tree, even if the tree is on an adjacent property. (Does not apply if the soil is known to be predominately non-expansive.)	
	Check that there is no conifer tree (e.g., pine) closer to the foundation than a distance equal to the radius of its canopy, even if the tree is on an adjacent property. (Does not apply if the soil is known to be predominately non-expansive.)	
	Check that there are no trees of any kind and no large shrubs growing next to the foundation. (Does not apply if the soil is known to be predominately non-expansive.)	
Water Leaks	Check that there are no leaks near the foundation, such as a faucet drip or a condensate drip from an air conditioning unit. If found, repair as needed.	
	Check that the automatic sprinkler system (if applicable) is properly functioning. Change settings as required to keep watering uniform but to a minimum (as needed to support the vegetation), particularly around the foundation. Set the cycle times to purposely water trees away from the structure in an effort to establish their roots away from the foundation.	
	Check that swimming pools, ponds, and fountains hold water without leaking.	

APPENDIX B - INITIAL INTERIOR SURVEY

Date_________

Category	Items to Check (at the time of purchase or move-in)	√
Cracks & Separations	Check that there are no cracks or separations in the coverings for the walls, ceilings, or floors if the structure is new.	
	Check that the observed cracks or separations in the coverings for the walls, ceilings, or floors are no more than hairline if the structure is used, but is less than 10 years old.	
Water Leaks	Check that all plumbing works properly, and that there is no stoppage or leaks. If a problem is found, repair as needed.	
Miscellaneous	Check that each door hangs properly, i.e., it does not stick, swing open, or shut on its own, and that there is no appreciable gap between the top of the door and its doorframe header above.	
	Check that there are no uncomfortable floor slopes, easily noticed by walking each room.	
	Check that wood rafters (where applicable) in the attic are not pulled away from ridge members.	
	Check that there is no evidence of past drywall or other architectural repairs.	

APPENDIX C - REGULAR EXTERIOR MAINTENANCE Date________

Category	Items to Check (at six-month intervals)	√
Cracks & Separations	Check for new or changed cracks or separations in the walls. If some have developed, do not repair them without first having a forensic engineer or a forensic consultant investigate the cause of the distress.	
	Check that masonry expansion joints are of uniform width top to bottom and the mortar joints are aligned.	
Drainage	Check that water does not pool near the foundation after a heavy rain. If found, correct the grade slope or add underground drainage.	
	Check the automatic sprinkler system (if applicable) for proper settings to give the site vegetation sufficient moisture to keep it from wilting, but without over-watering it. As part of the check, look inside each underground valve box and in the main water meter valve box to make sure they are dry. If the valves are submerged, suspect over-watering and stop watering in those zones until they are again dry or until the vegetation begins to wilt.	
	Check that patios and flatwork around the structure are providing positive drainage away from the foundation.	
	Check that fences, flowerbeds, or edging are not blocking drainage.	
	Check that downspouts and gutters are clean, and water from downspouts is directed away from the foundation.	
	Check for clogs or leaks in any existing downspout extensions, area drains, or underground drainage pipes, and clean and repair as required.	
Vegetation	Check that there is no broadleaf tree (e.g., oak, ash, tallow, pecan, and hackberry, etc.) closer to the foundation a distance equal to the height of the tree, even if the tree is on an adjacent property. If such is the case, begin a pruning program to keep the tree's canopy at that size for the rest of its life. A reasonable pruning interval would be every 2 - 3 years. (Does not apply if the soil is known to be predominately non-expansive.)	
	Check that there is no conifer tree (e.g., pine) closer to the foundation a distance equal to the radius of its canopy, even if the tree is on the adjacent property. If such is the case, begin a pruning program to keep the tree's canopy at that size for the rest of its life. A reasonable pruning interval would be every 2-3 years. (Does not apply if the soil is known to be predominately non-expansive.)	
	Check that there is no new tree of any kind coming up next to the foundation. If found, remove it.	
	Check that there are no shrubs next to the foundation that have grown to the point where they approach a one story roof in height. If found, cut them back to window height or replace them with a smaller variety. (Does not apply if the soil is known to be predominately non-expansive.)	
Water Leaks	Check that no leaks have developed near the foundation, such as a faucet drip or a condensate drip from an air conditioning unit, particularly from its emergency overflow pipe. If found, repair as needed.	
	Check that the underground drainage system (if applicable) is properly functioning. If it does not drain freely, investigate and clean as needed to achieve normal flow.	
	Check that swimming pools, ponds, fountains, etc. are holding water without leaking. If suspected of losing water below grade, have a pool-leak-detection company investigate, isolate, and repair the leak.	

APPENDIX D – REGULAR INTERIOR MAINTENANCE Date________

Category	Items to Check (at six-month intervals)	√
Cracks & Separations	Check that there are no new or changed cracks or separations in the coverings for the walls, ceilings, or floors.	
Water Leaks	Check that all plumbing works properly, and that there is no stoppage or leaks. If found, repair as needed.	
Miscellaneous	Check that there are no uncomfortable floor slopes by walking each room. If the maintainer is the tenant, perhaps ask someone else to check this, as it is easy to become accustomed to slopes that have gradually changed over time.	
	Check that wood rafters (where applicable) in the attic are not pulled away from ridge members.	
	Check that each door hangs properly (or as it did before), i.e., it does not stick, or swing open, or shut on its own, and there is no appreciable gap between the top of the door and its doorframe header above.	
	Check interior countertops for levelness, and check cabinet doors and drawers for proper operation.	

DESIGN, MANUFACTURE, AND INSTALLATION GUIDELINES OF PRECAST CONCRETE SEGMENTED PILES FOR FOUNDATION UNDERPINNING

by

The Structural Committee

of

The Foundation Performance Association

Houston, Texas

Document # FPA-SC-08-0

ISSUE HISTORY (Some internal issues omitted)

Rev #	Date	Description	Subcommittee Chair	Subcommittee Members
A	24 Jan 01	First Issue for Committee Comments	Karl Breckon	George Wozny Ron Kelm Michael Skoller Bill Polhemus Dan Jaggers Lowell Brumley Mari Mes David Dorr Nicole Wylie Bob Newman
Q	20 May 05	Issued for FPA Peer Review		
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PREFACE

This document was written by the Structural Committee and has been peer reviewed by the Foundation Performance Association (FPA). This document is published as FPA-SC-08 Revision 0 and is made freely available to the public at www.foundationperformance.org so all may have access to the information. To ensure this document remains as current as possible, it may be periodically updated under the same document number but with higher revision numbers such at 1, 2, etc.

The Structural Committee is a permanent committee of the Foundation Performance Association. At the time of writing this document, the Structural Committee was chaired by Ron Kelm, P.E. and 20 to 25 members were active on the committee. The committee sanctioned this paper and formed a subcommittee to write the document. The subcommittee chair and members are listed on the cover sheet of this document.

Suggestions for improvement of this document shall be directed to the current chair of the Structural Committee. If sufficient comments are received to warrant a revision, the committee will form a new subcommittee to revise this document. If the revised document successfully passes FPA peer review, it will be published on the FPA website and the previous revision will be deleted.

The intended audiences for the use of this document are engineers, foundation repair contractors, segmental pile manufacturers, builders, owners, and others that may be involved in the design, manufacture, and installation of foundation underpinning and underpinning components.

This document was created with generously donated time in an effort to improve the performance of foundations. The Foundation Performance Association and its members make no warranty, expressed or implied, regarding the accuracy of the information contained herein and will not be liable for any damages, including consequential damages, resulting from the use of this document. Each project should be investigated for its individual characteristics to permit appropriate application of the material contained herein.

TABLE OF CONTENTS

1.0 INTRODUCTION

The scope of this document is to provide guidance and information for projects that use precast concrete segmented piles for underpinning the foundations of existing residential and other low-rise structures. This type of foundation underpinning system is usually not suitable for high-rise buildings or other similarly heavily loaded structures due to the limited load capacity of the pile sizes commonly used and the driving depths typically achieved. Most applications involve lifting existing foundations, but some only intend to stabilize a foundation from future downward movements.

The foundation underpinning system is also known by generic names such as precast concrete piles, pre-cast pilings, hydraulically driven concrete piling, pressed piles, pressed pilings, driven cylinders, and others. Most of the available segmented pile systems utilize cylindrical concrete segments, but rectangular and other shapes may be used.

Piles consist of precast concrete segments, usually manufactured cylinders, which are installed one by one on top of one another, pressed into the ground by hydraulically jacking against the underside of the existing structure. The weight of the structure is used to create the reactive force that allows the pile segments to be driven into the soil. These piles may be categorized as driven displacement piles, which displace and force aside the surrounding soil as they are driven. The piles transfer load to the foundation soils primarily through skin friction along the length of the pile, although some end-bearing load transfer also occurs.

This system is mainly utilized in clay soils where the driving resistance is small enough to allow the weight of the structure to be used to develop the driving force necessary to obtain sufficient pile penetration. Dense granular soils may offer too much driving resistance, making the piles more difficult to install with the available weight of the structure as the driving force. For similar reasons, precast concrete segmented piles are also difficult to install in clay soils with stiff sandy clay or clayey sand layers. However, a high-pressure water injection technique, called jetting, or other methods, such as pre-drilling, may be used to break up the soil and allow additional pile penetration.

This document addresses both interconnected and non-interconnected precast concrete segmented piles. Means of interconnection may consist of steel bar(s), threaded rod, or cable that is inserted into a hole preformed through the center of each of the concrete segments along their longitudinal axis. These elements are typically used to align and/or hold the segments together. An alternative method of interconnection may be to bond the segment ends using epoxy or other adhesive. It is recommended that interconnected piles be used.

2.0 GENERAL DESIGN CONSIDERATIONS

In deciding which type of foundation underpinning system to specify, consider the following in using interconnected and non-interconnected precast concrete segmental piles:

1. Non-interconnected precast concrete segmental piles without reinforcement are typically less expensive.

2. Precast concrete segmental piles are not able to resist significant bending moments due to lateral loads.

3. Depending on the type of interconnecting system used and when the interconnecting element is installed, the interconnection may help to avoid detrimental vertical misalignment of the pile while being driven.

4. When interconnected, a properly designed and installed concrete segmental pile is more likely to resist the uplift forces due to swelling of expansive soils transmitted via friction along the shaft. A pile will not resist the uplift forces to the foundation.

5. Depending upon the soil uplift forces, interconnected precast concrete segmental piles that are connected to the existing foundation system may not be able to provide resistance against foundation uplift if the soil is in contact with the existing foundation system.

6. For most projects, the final depth of each pile will vary from pile to pile. As a result of using only the weight of the structure to drive the pile, the precast concrete segmental pile system has a depth of refusal that varies depending upon the tributary weight and stiffness of the structure above the pile being driven. If the pile cannot attain sufficient penetration into stable soils, then it may not be anchored against potential movements that occur due to swelling or shrinking of the soils in the moisture active zone.

7. Should a void exist under the slab subsequent to the lifting process, treatment of the void should be determined on an individual basis.

8. Geotechnical investigation and structural analysis could be of value for the design of the foundation repair.

3.0 FOUNDATION UNDERPINNING REPAIR GUIDELINES

The purpose of this section is to provide a guideline to specifying foundation underpinning repair, including remedial precast concrete segmental piles for foundation repair, foundation stabilization against movements, and foundation lifting.

3.1 DEFINITIONS

Segment: Precast concrete units that are typically cylindrical in shape and about one-foot long, although other shapes and lengths may be used.

Reinforced Precast Concrete Segments: Precast concrete pile Segments reinforced with steel, fiberglass, or other materials.

Pile: The Pile or Piling includes the following elements: multiple Segments that are driven into the ground, Pile Head, Shims, and Interconnections (if specified).

Pile Head: The uppermost section of the Pile. The Pile Head typically consists of a rectangular or trapezoidal shaped concrete block placed on top of the last driven Segment along with two additional Segments or other spacer elements placed on top of the rectangular or trapezoidal shaped concrete block.

Foundation Underpinning: The process of adding additional supporting elements under an existing foundation system.

Shim: Metal or other material used to fill the space between the Pile Head and the bottom of the foundation system.

Precast Concrete Segmented Pile: The installed assembly of Segments, Pile Head, Shims, and interconnection system (if applicable).

Non-Interconnected Precast Concrete Segmented Pile: Precast concrete segmented piles that are driven into the ground without any physical connecting elements between the pile segments.

Interconnected Precast Concrete Segmented Pile: Precast concrete segmented piles, with a center hole in the longitudinal axis, held together by a central steel reinforcing bar, a steel rod or rods, threaded ends and coupling nuts, steel cable, or other similar reinforcement. The Segments may be interconnected by inserting the connecting element through the center hole of each Segment. An alternative method of interconnection may be with adhesives, such as epoxy, between the ends of the Segments.

Foundation Elevation Adjustment: The process of raising or lowering the foundation elements in order to obtain a new vertical position in an effort to reduce distress, deflection, and/or tilt to the superstructure.

Foundation Stabilization: The process of underpinning the foundation to help prevent future downward foundation movement without appreciably performing Foundation Elevation Adjustment.

Refusal: Refusal is defined as the point when the structure is lifted 1/4 inch to 1/2 inch above its elevation at the location of the pile during the driving process. The Pile is said to reach refusal and the driving operation is stopped before the occurrence of significant vertical movement of the building that may cause damage to the superstructure.

3.2 REFERENCES AND STANDARDS

The contractor shall follow all applicable codes and standards. The following standards are identified by issuing authority, authority abbreviation, designation number, title or other designation established by the issuing authority. The contractor shall follow the current version of the applicable standards from the American Society for Testing and Materials (ASTM) list below :

ASTM A29/A29M Steel Bars, Carbon and Alloy, Hot-Wrought and Cold Finished.
ASTM A36/A36M Carbon Structural Steel.
ASTM A153 Zinc Coating (Hot Dip) on Iron and Steel Hardware.
ASTM A416 Steel Strand, Uncoated Seven-Wire for Pre-stressed Concrete.
ASTM A615 Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.
ASTM A706 Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement.
ASTM A767 Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement.
ASTM A775 Epoxy-Coated Reinforcing Steel Bars.
ASTM A992 Structural Steel.
ASTM C33 Concrete Aggregates.
ASTM C39 Test Method for Compressive Strength of Cylindrical Concrete Specimens.
ASTM C150 Portland Cement.
ASTM C494 Chemical Admixtures for Concrete.

3.3 DESIGN REQUIREMENTS

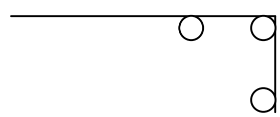
The owner shall have direct involvement in specifying the project objective. Design of the precast concrete segmented pile for commercial applications shall be prepared by an Engineer (for the purposes of this paper, defined as “Licensed Professional Engineer”)**.** In residential projects, an Engineer should be engaged to prepare a design or analyze a pile layout that has been prepared by a contractor.

The following are general piling placement guidelines in evaluating a typical one story and two story residential structures. The purpose of these guidelines is to provide general outlines when preparing a foundation repair plan.

1. **<u>Exterior Pilings</u>** - Exterior piling supports shall have a maximum spacing as follows: (porches and garages see below)
    a. 8 feet center to center on a 1-story wood or brick structure.
    b. 7 feet center to center on a 1-1/2 story (i.e. a 1-story with a partial 2nd-story) and 2-story with brick on the 1st story only.
    c. 6 feet center to center on 2-story brick structures.
    d. If there is no exterior grade beam present, then a structural member designed by an Engineer shall be installed between the piling and the slab in order to reduce the punching shear stresses and to increase the pile driving force.

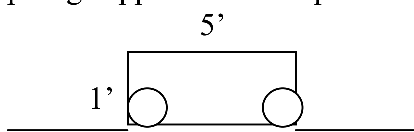
2. **<u>Interior Pilings</u>** - Interior pilings shall have a maximum spacing as follows:
    a. 8 feet center to center on 1-story structures.
    b. 7 feet center to center on 2-story structures.
    c. If an interior room is 16 feet wide or greater, piling supports shall be placed in the middle of the room (excluding garages). If there is no interior grade beam present then a structural member designed by an Engineer shall be installed between the piling and the slab in order to reduce the punching shear stresses and to increase the pile driving force.

3. **<u>Corners</u>** Piling supports shall be placed at each corner where an exterior wall changes direction. When lifting a corner provide at least one piling support on each side of the corner pile as depicted below.

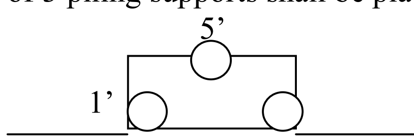


4. **<u>Fireplaces</u>** - The following are examples of piling support placement for fireplaces.

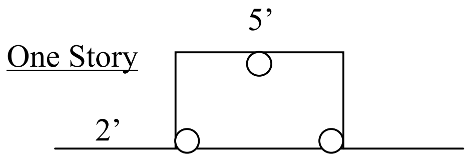
    a. 1 foot by 5 feet prefabricated fireplace with wood exterior a minimum of 2 piling supports shall be placed as shown below.

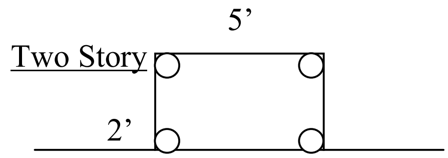


    b. 1 foot by 5 feet prefabricated fireplace with brick veneer exterior a minimum of 3 piling supports shall be placed as shown below

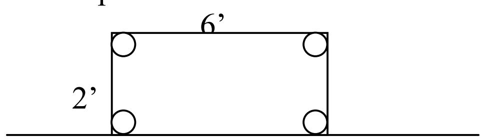


c. 2 feet by 5 feet fireplace with brick veneer exterior a minimum of 3 piling supports shall be placed as shown below for a one story and 4 pilings shall be used on a two story.

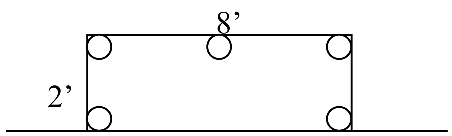




d. 2 feet by 6 feet fireplace with brick exterior a minimum of 4 piling supports shall be placed as shown below.

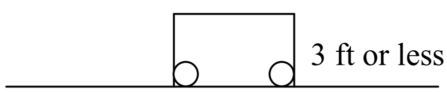


e. If the fireplace is 8 feet or greater in length then an additional piling shall be placed at the center on the exterior wall as depicted below.

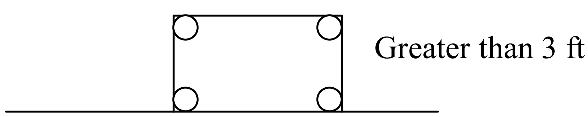


5. **<u>Porches</u>** - The following are examples of piling support placement for porches.

a. If the inset of the porch wall is 3 feet or less then a minimum of 2 piling supports shall be placed as illustrated below.



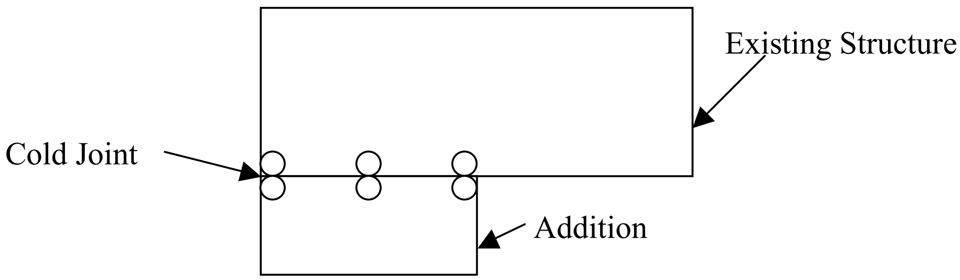
b. If the inset of the porch wall is greater than 3 feet then a minimum of 4 piling supports shall be placed as illustrated below.



c. On extended one-story porches piling supports shall be placed at a maximum distance of 12 feet center to center or shall be placed directly under the overhang column supports.

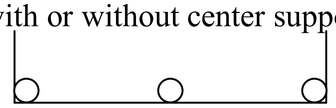
6. **<u>Wing Walls</u>** - If the wing wall is monolithic and greater than 3 foot in length then a piling shall be placed at the end of the wing wall.

7. **Additions** – At additions, the drawing shall show the concrete joint and separation dimensions. At the interface of the addition and the original structure double-piling supports shall be used and spaced per the exterior and/or interior requirements stated above.



8. **Garages** – A minimum of one piling support shall be used at the mid-point of a garage door opening for a 2-car garage. A piling support shall be placed under garage support columns. Examples are illustrated below.

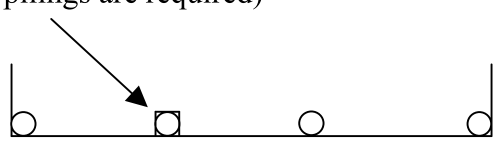
    a. Two car garage with or without center support.



Garage Door Opening

    b. Three car garage.

Piling located under concentrated load (Engineer to determine if additional pilings are required)



1 car door opening 2 car door opening

3.4 SUBMITTALS

The contractor shall submit the following items to the owner or his designated representative in accordance with conditions of the contract and the applicable submittal procedures in the above References and Standards (Section 3.2):

Product Data: Submit product data for the specific type of precast concrete segmental pile used, showing pile configuration including, but not necessarily limited to, reinforcing type (if any), details of pile toe and pile head, manufacturing and fabrication details, installation instructions, product components, and related accessories.

Pile Location Drawings: Submit drawings showing the proposed location, and total number of precast concrete segmental piles.

Qualification Data: If requested, the contractor shall submit a list of completed projects with project names and addresses, names and addresses of owners, and other information to demonstrate their capabilities and experience.

Engineering Analysis: For a commercial project or if requested for a residential project, a foundation analysis and a foundation repair design shall be prepared by an Engineer. When a design is submitted by the contractor for review, it should be analyzed by an Engineer.

Pile-Driving Equipment: Submit data sheets that indicate the type, make, rated load capacity, and manufacturer's recommended calibration frequency of the hydraulic ram that will be used to drive the segmental piles. Submit the date of the ram's last calibration or purchase date of the pressure gauge. Submit data sheets that indicate the type, make, and pressure capacity of any water jetting equipment that will be used. Include a description of how the water jetting operations, (if used), will be integrated with the pile driving operations.

Foundation Lifting Equipment: Foundation lifting equipment typically consists of hydraulic jacks with a capacity in the range of 20 to 50 tons.

Pile-Driving Records: Submit to client or his representative the pile-driving records within seven days of driving the pile. The pile-driving records shall indicate depths of exterior piles from existing grade and depths of interior piles from the bottom of the slab. The submitted records shall include the force or pressure used to drive the segmental piles. Records shall indicate if water jetting was utilized and the range over which water jetting was engaged. These records shall be forwarded to the Engineer in order to verify that sufficient pile depths and other design requirements have been achieved.

Prequalification Test Reports: If a prequalification test is required to observe the behavior of the building's superstructure and the pile under actual driving conditions, then a test pile shall be driven at an outside corner of a building. If an outside corner is not available then the test pile shall be driven in another area where the driving force (structure's weight and stiffness) is minimized. Test reports for prequalification test piles shall be submitted to the Engineer for review and comment prior to the installation of additional Piles.

Safety Program: Submit Contractor's standard safety requirements for working in excavated trenches, tunnels, and confined spaces that meets the requirements of OSHA standards.

Initial Elevation Survey: Along with the Pile Location Drawings, submit the initial foundation elevation survey clearly showing the location of the reference datum. A symbol, such as a dot, shall be used to show the location of each elevation survey point.

Final Elevation Survey: After installing the Piles, submit the final foundation elevation survey. The reference datum used in the initial elevation survey shall be used for the final elevation survey.

Material Test Reports or Material Certificates: Submit material test results from a qualified testing agency, or material certificates from the manufacturer, indicating compliance with concrete and reinforcing (if applicable) materials.

3.5 QUALITY ASSURANCE / QUALITY CONTROL

The following quality assurance / quality control requirements shall be submitted to the owner or his designated representative:

Contractor Qualifications: Contractor shall submit evidence of experience in performing this type of work. Include list of completed projects with project names and addresses, names and addresses of owners, and other pertinent information.

Licensed Professional Engineer Qualifications: If an Engineer is retained for the project, evidence of experience in performing this type of work shall be submitted.

Pre-installation Meeting: If requested by the owner, contractor shall conduct a pre-installation meeting with owner or his designated representative to verify project requirements, soil conditions, contractor/manufacturer's installation plan and instructions, contractor/manufacturer's warranty, and other requirements of this document or contract.

Precast Concrete Segment Manufacturing Tolerances: Each precast concrete segment shall be within the following tolerances:

- Cylinder Diameter: Tolerance for the diameter of each of the precast concrete cylinders shall be plus or minus (±) 1/4 inch.
- Cylinder Length: Tolerance for the length of the each of the precast concrete cylinders shall be plus or minus (±) 1/2 inch.
- Segment Ends Concentricity: Tolerance for the concentricity of each of the precast concrete segment about the central axis shall be plus or minus (±) 1/16 inch per foot of segment length.
- Hole Location (if applicable): Tolerance for the location of the central hole through each segment shall be plus or minus (±) 1/8 inch from true center, and measured at each end.
- Hole Diameter (if applicable): Tolerance for the diameter of the central hole through each segment shall be plus or minus (±) 1/16 inch.
- End-Bearing Surface Flatness: Tolerance for flatness of end-bearing surfaces shall be plus or minus (±) 1/16 inch throughout the area of each end-bearing surface.
- End-Bearing Surface Cant: The cant of the end-bearing surface from a true perpendicular surface to the longitudinal axis of the cylinder shall be plus or minus (±) 1/2 degree.

Quality-Control Testing: Owner may employ, at owner's expense, an independent testing agency to evaluate the precast concrete segment manufacturer's quality control and testing methods. Owner's testing agency shall have access to material storage areas, concrete production equipment, concrete placement, and curing facilities. Contractor shall cooperate with Owner's testing agency and provide samples of materials and concrete mixes as may be requested for additional testing and evaluation. Owners testing agency shall provide contractor a written report of findings.

Defective Work: Strength of precast concrete segments will be considered deficient if units fail to comply with requirements. Discard precast concrete segments that do not comply with requirements, including strength, manufacturing tolerances, and finishes.

Verification of Equipment Performance: Calibrate hydraulic ram using method and frequency in accordance with hydraulic jack manufacturer's recommendations.

3.6 DELIVERY, STORAGE, AND HANDLING

Deliver pile components to the Project site in such quantities and at such times to ensure continuity of installation and to meet Project schedule requirements. Handle and store pile materials at the Project site to prevent breaks, chips, cracks, or other physical damage to the precast concrete segments and to any protective coating that may be used on reinforcement, if applicable.

3.7 WARRANTY

Submit contractor's standard warranty document executed by authorized company official. Contractor’s warranty is in addition to, and not a limitation of, other rights Owner may have under the contract document and / or law. The warranty period shall be a minimum of ten (10) years commencing on date when the contractor substantially completed the work. Warranties shall be transferable.

During the warranty period, if differential deflections occur due to downward movement (upward movement is not normally covered unless stated otherwise in the warranty) in any repaired area that have caused damage to the architectural finishes, then the warranty shall cover re-lifting of the foundation to as near the original as-lifted condition as possible, including any tunneling, concrete, decking, or pavement repair as required. If cosmetic repairs and landscaping were not included in the initial scope of work, they will not be covered when warranty foundation repairs are made.

3.8 PRODUCTS

Compressive Strength: The concrete shall have a minimum 28-day compressive strength of 5,000 psi in accordance with ASTM C39.

Reinforcement: The central longitudinal segmented pile reinforcement shall conform to the following, if applicable:

Reinforcing Bars: ASTM A615, Grade 60, deformed.
Low-Alloy-Steel Reinforcing Bars: ASTM A706.
Galvanized Reinforcing Bars: ASTM A767, Class II, hot-dip galvanized.
Epoxy-Coated Reinforcing Bars: ASTM A775.
Steel Cable: ASTM A416, Grade 250, galvanized, seven-wire, low-relaxation strand.
Steel Rods: ASTM F1554.

Pile Heads: Pile heads shall be precast concrete having the same strength and properties of the precast concrete pile segments.

Steel Reinforcement Corrosion Protection: The reinforcement, if applicable, shall be provided with corrosion protection as follows:

- Steel deformed bar reinforcement (rebar) shall be encased in epoxy or epoxy grout until the annulus fills.
- Steel cable reinforcement and threaded rods shall be galvanized.
- Smooth steel rod reinforcement shall be coated with epoxy paint.

Accessories: Shims shall have a minimum thickness of 1/8 inch. Shims and other accessories shall be of sufficient size, strength and durability to match the load capacity of the precast concrete segments.

3.9 INSTALLATION

Install Precast Concrete Segmental Piles in accordance with the following steps:

1. **Safety:** Meeting OSHA and other work safety requirements is the contractor's sole responsibility.

2. **Work Plan:** Contractor shall submit a work plan for excavation and backfill related to the underpinning work with a complete written description that identifies details of the proposed method of construction and the sequence of operations relative to excavation and backfill activities. The descriptions, and/or supporting illustrations, shall be sufficiently detailed to demonstrate that the procedures meet the applicable requirements.

3. **Manufacturer's Product Data:** Contractor shall comply with manufacturer's product data, including product technical bulletins.

4. **Site Examination:** Before installing any piles, the contractor shall examine the site to verify the existing conditions are as indicated on the contract drawings. Notify the owner's designated representative of any discrepancies found between the existing site conditions and the contract drawings.

5. **Piling Placement:** Use a method that will not cause damage to nearby structures. Remove concrete as required to create sufficiently sized access holes through existing slabs-on-grade. Remove and salvage any landscaping plants that occur at exterior access locations. Remove and store plants in accordance with good practice. Excavate soils as required at pile locations to obtain clearance under existing foundation elements sufficient to install piles. Provide a flat bottom surface in the excavated pit at the pile location under the existing foundation. Examine the underside of the existing foundation element, and, if required, chip concrete at bottom surface or add grout as required to provide a smooth bottom surface of the existing foundation at the pile location.

6. **Plumbing Leak Test:** Contractor should employ a licensed plumber to perform a hydrostatic leak detection test of the under-slab sanitary sewer lines and under-slab water supply lines before and after the work is performed. The plumber shall provide the results in a written report.

7. **Utilities:** Obtain utility company approvals when required before digging. When working near known buried utility lines, excavations shall be made using hand tools to avoid disturbing or damaging the utility lines.

8. **Groundwater Control:** If necessary, contractor shall provide groundwater control including dewatering of water-bearing soil layers to remove seepage water from excavations.

9. **Surface Water Control:** Contractor shall provide surface water control to divert water away from excavations through the use of dikes, ditches, curb walls, pipes, sumps, or other means.

10. **Access Holes:** Excavated holes (or pits) for precast concrete segmented piles for installation shall not be permitted beyond the depth required to obtain personnel and equipment access clearances necessary for the pile driving operation.

11. **Existing Drilled Piers:** If existing drilled piers are tied into the existing foundation, sever the top of the existing drilled piers subsequent to driving new precast concrete segmental piles from the bottom of the grade beams. When the tops of the existing drilled pier shafts are connected to the grade beams, the connection shall be severed by chipping to remove concrete and cutting reinforcement bars.

12. **Locate Piles:** Piles shall be positioned as indicated on the approved pile location drawings. Pilings shall be located not more than twelve (12) inches from design location on the Pile Location Drawing, unless approved otherwise by the contractor's design Engineer.

13. **Obstructions:** Remove any encountered obstructions, or add/relocate pile and adjacent piles as required by the pile layout designer.

14. **Stockpile Segments:** Stockpile a sufficient number of concrete segments at each pile location to obtain the anticipated pile depth, and have a sufficient number of extra segments readily available to obtain the pile length necessary to achieve the anticipated depth to refusal. The stockpiling requirement is to ensure that the anticipated pile depths can be obtained without having to stop the pile driving process to obtain more segments. Stopping the pile driving process could potentially cause early thixotropy (soil freeze up).

15. **Concrete Segment Size:** Unless approved otherwise, use the same concrete segment size for all segments within each pile. Contractors may use cylindrical concrete segments of varying diameters within a given single pile, increasing the diameter in a step-wise fashion to effectively taper the pile from bottom to top. An example would be to start

driving with 4-inch diameter segments, switching to 6-inch diameter segments, and then switching again to 8-inch diameter segments.

16. **Driving Records:** Maintain accurate driving records for each pile recording, as a minimum, the information shown on the Segmented Repair Piles checklist of FPA-SC-10 "Quality Control Checklists for Foundation Inspection of Residential and Other Low-Rise Buildings", current revision found at www.foundationperformance.org.

17. **Axial Alignment:** Establish and maintain axial alignment of all cylinders within each pile so that all cylinders remain concentric and vertical during the driving operation.

18. **Lubrication:** Water may be added to the bottom of the excavated hole (pile pit) at the pile location during pile driving for lubrication purposes. The amount of water used shall be minimized to avoid excessive water accumulation in the soils that could lead to additional swelling of expansive soils.

19. **Interruptions:** Drive each pile continuously until refusal. Avoid interruptions in the driving process that may cause soil freeze-up resulting in early refusal.

20. **Load Sharing:** Drive one pile at a time to avoid load sharing of the tributary building weight at the pile location to more than one pile. If piles are driven simultaneously the piles being driven shall be a minimum of 25 feet apart.

21. **Reinforcing:** If applicable, install steel reinforcing bars, rods, or cable through the central hole in each precast concrete pile segment during the pile installation. If steel cables are used, ensure that the cable is not slack.

22. **Epoxy:** If applicable, install epoxy or epoxy-grout to fill void around reinforcement. Some designs may also specify epoxy between the pile segments.

23. **Refusal:** Drive piles to the point of Refusal. In order to allow some time for clay soil to remold without significantly rebounding upward, at Refusal, maintain the hydraulic jack pressure for a minimum period of 5 minutes before removing the jack.

24. **Water Jetting:** Water jetting shall not be used unless specifically permitted by the Engineer and / or the building official. If used, jetting shall be carried out in such a manner that the capacity of the existing foundations and structures shall not be impaired.

25. **Pre-drilling:** Pre-drilling of holes at the pile locations will not be permitted unless otherwise approved by an Engineer.

26. **Hydraulic Jack Pressure:** Monitor the hydraulic jack pressure while driving each pile segment. If a sudden significant loss of pressure occurs along with any abnormal sound from the pile (that may indicate crushing of a concrete segment), the pile shall be considered defective and shall be abandoned and a new pile shall be added. Concrete crushing of a pile segment that occurs during the driving process is considered to result in

a defective pile since compressive load transfer between the segments can no longer be assured and further driving of the pile can result in misalignment of the pile.

27. **Pile Depth:** Install piles to the specified minimum depth if determined by a geotechnical engineer. The contractor shall provide piles capable of withstanding the pile driving stresses and design loads, and capable of being driven to refusal at or below a minimum design depth if specified by the Engineer. Piles that reach refusal before attaining the minimum required depth as specified shall be subject to the following:

    a. Terminate pile at depth obtained with approval of Engineer, or
    b. Replace pile with pile having a smaller cross sectional area, installed at a location at least 6 pile diameters from the terminated pile, or
    c. Implement water jetting or pre-drilling with approval of Engineer.

    If more than three consecutive piles cannot be driven to the minimum specified depth, notify the Engineer and obtain further instructions from the Engineer as to how to proceed. Do not drive any more piles until receiving instructions from the Engineer.

28. **Cap Pile:** Immediately after removing the hydraulic ram, and after completion of the driving process, temporarily cap and shim off the pile to prevent pile rebound. Install specified pile cap horizontally on top of the driven pile segment and install shims.

29. **Adjacent Piles:** Proceed to drive adjacent piles using the same steps as outlined above. Do not over-lift (See “Refusal”) the structure during the underpinning process since it may cause damage to the structure and / or architectural finishes.

30. **Defective Pile Segment:** Withdraw damaged or defective pile segment and reinstall new pile segment.

31. **Defective Pile:** Abandon damaged or defective pile and install new pile in alternate location (See “Locate Piles”). Fill hole left by abandoned pile using the excavated soils or alternate fill materials. Place and compact in lifts not exceeding 8 inches. Record locations of abandoned pile on the as-built drawings.

32. **Adjust Pile Caps:** After all piles are installed, adjust all pile caps and shims as required to correct any shims that may have been dislodged during the driving of adjacent piles to provide full contact bearing at pile locations. Maintain horizontal alignment of the pile cap on top of the driven pile segments.

33. **Foundation Lift:** Lift foundation system in a systematic manner using jacks. The lifting process shall be performed in a manner that curtails damage to the structure. Attempt to close any masonry and / or drywall cracks as much as possible. Test doors and windows to ensure they operate as intended. If a primary structural member, such as a grade beam, is damaged during driving or lifting, an Engineer must be consulted for its repair and the member shall be repaired in accordance with the Engineer’s repair specification, at the contractor’s expense.

34. **Reinforcement Splices:** Splices in the pile's central steel reinforcing, if used, shall not be permitted except as approved by an Engineer.

35. **Final Elevation Survey:** After the installation is complete, the contractor shall perform a final elevation survey of existing foundation floor twelve feet beyond all locations where piles were installed. This will establish a benchmark survey that may be used for warranty purposes. The results of the survey shall be documented on a sketch of the foundation repair plan, showing the location of the reference datum, and a symbol, such as a dot, indicating the location of each elevation survey point and submitted to the Engineer and / or owner for review.

36. **Restore Landscaping:** Contractor shall restore any landscaping plants that were salvaged during the preparation stage to their original locations and restore lawn to its original condition. Owner shall be responsible for replacement of landscaping that was not salvageable. After backfilling, restore building interior slab and exterior pavement where access holes were required. Match finishes as close as possible.

37. **Excess Material:** Haul off excess excavated materials and clean finished surfaces.

QUALITY CONTROL CHECKLISTS FOR FOUNDATION INSPECTION OF RESIDENTIAL AND OTHER LOW-RISE BUILDINGS

by
The Structural Committee

of
The Foundation Performance Association

www.foundationperformance.org

Houston, Texas

Document # FPA-SC-10-0

ISSUE HISTORY

Rev#	Date	Description	Subcommittee Chair	Subcommittee Members
A	02 Oct 01	For Subcommittee Comments	Jack Spivey	Ron Kelm Jon Monteith Michael Skoller Terry Taylor Mari Mes Mike Palmer Lowell Brumley George Wozny Dan Jaggers Toshi Nobe
B	19 Sep 02	For Subcommittee Comments		
C	14 Oct 02	For Subcommittee Comments		
D	26 Nov 02	For Subcommittee Comments		
E	10 Dec 02	For Subcommittee Comments		
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H	28 Apr 03	Issued for Committee Comments		
I	10 Jul 03	For FPA Peer Review		
0	09 Oct 03	FPA Web Site Publishing		

PREFACE

The following documents are the results of two years of work completed in the late nineteen nineties by the Inspections Subcommittee of the Foundation Performance Committee. Jack Spivey chaired this committee and his fellow members were:

MR. MICHAEL SKOLLER P.E.

MR. JOE EDWARDS

MR. LOWELL BRUMLEY P.E.

MR. DEAN EICHELBERGER

Meetings took place on a monthly basis and were attended by many interested parties. Special recognition should be given to Mr. Jim Dutton of Du-West Foundation Repair and Mr. Dan Jaggers of Olshan Foundation Repair. Their assistance with the foundation repair sections was invaluable. The topics for discussion have followed a general outline, which was established at the onset of the meetings. It was determined that our basic intent would be to establish a set of standards and procedures for the inspection of foundation construction and foundation repairs. These standards were to be incorporated into an inspection document, which would be thorough in its scope, but also easy to use. It was established early on in our discussions that the best form for our purposes would be a simple checklist, which would fully cover the subject of the inspection. It was also determined that keeping the checklist to one page would afford the most user-friendly instrument for our purposes. Once these parameters were established the subjects of the inspections were taken in the following order:

FOUNDATION MAKE-UP -- POST TENSION
STRESSING POST TENSION
FOUNDATION MAKE-UP -- CONVENTIONAL/REBAR
CONCRETE PLACEMENT
CONSTRUCTION PIERS
REPAIR PIERS
SEGMENTED REPAIR PILES

These topics were judged to represent the major types of foundation construction and foundation repairs found in the Houston area. They are certainly not inclusive of every inspection situation or construction method in use, but they do offer a basic set of standards for the majority of inspections that would be encountered in typical residential construction.

They are also designed to be used by anyone who has some knowledge of foundation construction. It was our intention that they would serve field inspectors, builders, builders' superintendents, municipal inspectors, or anyone with an interest in quality foundations.

The first order of business worked on by the subcommittee was to establish a heading format for each inspection. This portion of the form is meant to establish a context for the inspection. The basics of the site such as, the builder, subdivision, address, lot and block, are all set out at the top of the form. The next section is meant to establish the parameters that will govern the rest of the inspection. The most important of these, deals with the plans. No inspection should be undertaken without a set of plans, which should include the name of the engineer, the date of the plans and the detail sheet. Other pertinent details of the site that are covered in this section are the date, the time, the weather, and whether there is a detached garage.

The above guidelines were followed on each form, with the following variations dictated by the context of the inspection:

- For the Concrete Placement Form there is specific reference to the Foundation Make-Up Form, and the items in need of repair.
- In the Stress Form, there is an added reference to the cable count, the concrete placement date, and the post tension construction company.
- On the Construction Piers Form, there is a reference to the Geotechnical Engineer, and on the Repair Piers and Segmented Repair Piles Forms, there is reference to the design documentation and the municipal permit.

Once the context is established in the heading, the form moves on to sections relating to different aspects of each inspection. In general, these sections are documented by simply checking the item to show that it has been correctly completed. **The checkmark *(✓)* serves to show that the item has been considered and complies with the plans, whereas an x *(X)* denotes that the item does not comply with the plans.** In some sections, direct questions are asked that should be answered. Finally, the lower sections of the forms generally have reference to a drawing of the slab, the piers or piles, or the foundation being repaired. The drawings further document the conditions specific to the site and the foundation and allow the inspector to orient the data being described in the conclusion of the inspection.

Each of these forms represents an attempt to document the events related to a specific foundation project or a specific foundation repair. It should be remembered that all the answers and data reported are typically the only documentation of what actually happened during this phase of construction. For this reason, every item is pertinent and should be given careful consideration during the inspection. Though many of the items listed are fairly common knowledge to the typical inspector or builder, it is the sequencing and nuances of certain questions and items listed, which are the greatest advantage of using the forms. The committee felt that all major items such as beam size, tendon counts, plan dates, etc., were adequately covered in each form.

It should be noted that the Repair Piers and Segmented Repair Piles Forms contain information that is not found in any established sources or specifications. This is particularly true of the Segmented Repair Piles Form. It was generally agreed that these items are rarely inspected by an independent inspector.

This document is made freely available to the public through the Foundation Performance Association at www.foundationperformance.org so engineers, architects, inspectors, contractors, and other professionals involved in the quality control of foundations systems for residential and low-rise buildings may have access to the information. To ensure the document remains as current as possible, it will be periodically updated under the same document number but with new revision numbers. Please direct suggestions for improvement to the current chair of the structural committee.

QC Checklists

1. **POST-TENSION SYSTEM FOUNDATION MAKE-UP**
2. **CONCRETE PLACEMENT**
3. **POST-TENSION SYSTEM STRESSING**
4. **CONVENTIONAL (REBAR) FOUNDATION MAKE-UP**
5. **CONSTRUCTION (BUILDERS) PIERS**
6. **REPAIR PIERS**
7. **SEGMENTED REPAIR PILES**

CLIENT__________________ **QUALITY CONTROL COMPANY______________________________**

QC Checklist #1 - POST-TENSION SYSTEM FOUNDATION MAKE-UP

Builder___________________________ Subdivision______________________________ Date__________ Time__________
Site Address_______________________ Lot______ Blk______ Sec______ Plan site specific Yes ☐ No ☐
Plan #:_________ Cable Count________ Design Engineer______________________ Superintendent______________________
Plan provided at site Yes ☐ No ☐ Weather_____________ Plan Date_________ Detail Sheet Date ______________
Concrete Contractor__________________ Detached Garage Yes ☐ No ☐ Permit #:____________________

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

SITE

Subdivision Lot _____ Other ______________________
Lot Description_______________________________
Fill on site Yes ☐ No ☐
Compaction verified by Geotechnical Engineer:
Yes ☐ No ☐ Date___________
Will foundation make up drain: Yes ☐ No ☐
Trees removed________________________________
Are trees within 20' of foundation Yes ☐ No ☐

FORMS

☐ Forms secure
☐ Floats installed
☐ Proper clearance at floats
☐ Garage front closed

ADDITIONAL REVIEWS

Date_____ Time_______

SLAB

☐ Thickness_____________(in)
☐ Measured: Screeds_____ Stringline______ Other_______
☐ Describe Pad Material ___________________________
☐ Level and Firm Yes ☐ No ☐

BEAMS

☐ Design Depth: ______(in) Exterior_____ Interior______
☐ Actual Depth:______(in) _____(in) _____(in) _____(in)
☐ Design Width:______(in)
☐ Actual Width:______(in) _____(in) _____(in) _____(in)
☐ Average depth into undisturbed soil_______________(in)
☐ Clean of soil & debris
☐ Water in beams Yes ☐ No ☐ Average Depth_____(in)
☐ Will water drain Yes ☐ No ☐
☐ Plumbing obstructions accommodated ______________
☐ Pier tops clean

TENDONS

Count: L to R______ F to B________ Garage____________
Total ____ Variance ___Explain______________________
Number of tendons left on site_______ Rebar_____________
☐ 1/2" tendons __________________ Other
☐ No tendons spaced over 6'-0"
☐ 20D nails used at castings
☐ Live ends stripped of plastic not over 1" or taped
☐ Cathead clamps all tight
☐ All intersections tied
☐ All tendons supported at intersections
☐ Dead ends have 3/4" clearance to forms
☐ All S Hooks crimped
☐ Beam tendons draped and secured by #3 stakes or rebar concrete bricks
☐ Ample chairs all tied

Tendon grid secured for concrete placement Yes ☐ No ☐

POLYETHYLENE SHEETING

☐ 6-mil.Lapped and Taped ☐ Seated in bottom of beams secured at sides ☐ Mastic/tape applied at plumbing

REINFORCING STEEL

SLAB SECTION

WWF: (Mesh)Size_______ Roll_______ Sheet_________ **OR** ☐ #3 @_____________(in.) on center both ways
☐ All WWF (mesh) seams lapped 6" ☐ #3 Lapped per plans ☐ All edges 2" from forms
☐ No rebar or WWF (mesh) touching forms

BEAM SECTION

Rebar: grade ____ Clearances per plan: Sides ☐ Bottom ☐ Top ☐
☐ Splices lapped per plan
☐ Corner rebar installed at corners & dead ends
Typical Rebar/Exterior Beams ________________________ continuous
Typical Rebar/Interior Beams ________________________ continuous
Corner bars installed at dead ends Yes ☐ No ☐
Bay Windows or Porches ________ Rebar ________ Stirups ________
Extra Rebar Added ______________________________
Diagonal Rebar at Re-entrant Corners ☐ No. of Corners____________
Nose Bars @ _________________ Construction Joints ____________
Anchor bolts on site Yes ☐ No ☐ Diameter ______(in) Length_____ (in)
Other Fasteners_____________________

IS FOUNDATION READY FOR CONCRETE? Yes ☐ No ☐

Sketch

CHANGES NEEDED:____

____

____

Quality Controller's Signature **Superintendent's Signature**

CLIENT__________________ **QUALITY CONTROL COMPANY**______________________________

QC Checklist #2 - CONCRETE PLACEMENT

Builder____________________________ Subdivision______________________ Date__________ Time_______________
Site Address________________________ Lot_____ Blk_____ Sec_____ Plan #:________ Cable Count__________
Design Engineer_____________________ Superintendent___________________ Q.C. Arrival Time___ Departure Time____
Copy of Foundation Makeup Report Provided Yes ☐ No ☐ Date of Copy___ Items Repaired Yes ☐ No ☐
Concrete Contractor__________________ Detached Garage Yes ☐ No ☐ Permit #:__________________________

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

SITE
Subdivision Lot _____ Other ________
Lot Description___________________
Are there obstructions at the site which would prevent access for concrete trucks Yes ☐ No ☐
Explain ____

FORMS
☐ Forms secure
☐ Floats installed
☐ Proper clearance at floats
☐ Garage closed in

ADDITIONAL REVIEWS
Date_____ Time_______

WEATHER
Weather conditions START:______________________________ FINISH:______________________________
Will temperature rise above 40° F for five hours ____
Forty-eight hour forecast: HIGH TEMPERATURE:_______________ LOW TEMPERATURE:___________________

CONCRETE
Concrete Company ______________________ Batch Plant __________________________ Tickets on site? Yes ☐ No ☐
Delivered by truck over what distance _________ Was a pump used Yes ☐ No ☐ Pump Co._______________________
Mix : ____________ psi _____________ psi "pump mix"– Pump Prime Placed outside of form Yes ☐ No ☐
Sack Mix: _______ 4 _______ 5 _______ 6 **OR** Strength Mix Yes ☐ No ☐ Strength ________________________
Additives: ________________________________ **NO CALCIUM CHLORIDE**–APPLIES TO POST TENSION SLAB
Fly Ash: Type C? Yes ☐ No ☐ ________________%
Slump as ordered from plant _________________ (in)
Explain (Discrepancies if slump is different): ____
Was concrete consolidated by vibrator Yes ☐ No ☐ Other ☐ ____
Test Cylinders Taken Yes ☐ No ☐ Testing Company ______________________________________
Slump Test Taken Yes ☐ No ☐ Testing Company ______________________________________
If water is added at the jobsite, show the amounts over ten gallons and give a visual estimate of the final slump

Truck #	Time Poured Out	Gallons Added	Placement Location	Est. Slump	Tested Slump	Temp.

Draw a diagram of the slab below showing the locations of each load by truck number

Anchor bolts on site Yes ☐ No ☐ Diameter ______(in) Length________ (in)
Other Fasteners____
Describe provisions for curing______________________________________

SKETCH

ADDITIONAL COMMENTS: ____
____
____
____

______________________________ ______________________________
Quality Controller's Signature **Superintendent's Signature**

CLIENT____________________ QUALITY CONTROL COMPANY________________________________

QC Checklist #3 – POST-TENSION STRESSING

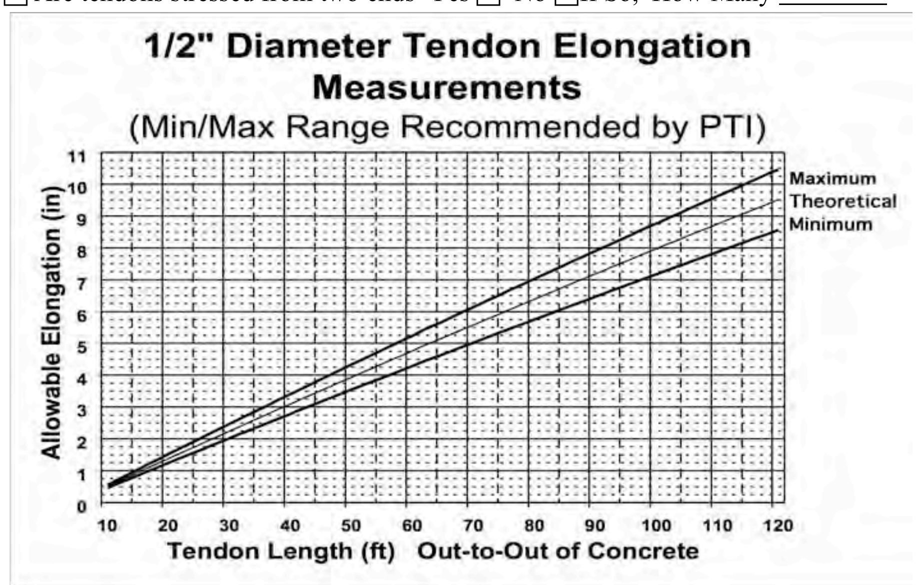
Builder____________________________ Subdivision________________________________ Date__________ Time__________
Site Address________________________ Lot________ Blk________ Sec____________ Plan site specific Yes ☐ No ☐
Plan #:_________ Cable Count__________ Design Engineer__________________________ Superintendent__________________
Plan provided at site Yes ☐ No ☐ Weather______________ Plan Date__________ Detail Sheet Date _________________
Concrete Placement Date_______________ Stress Date ____________________________ Partial Stress Date________________
Post Tension Company____ Permit #:_______________________

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

☐ Are there any cracks in the surface of the slab Yes ☐ No ☐ Describe___________________
____
____

ADDITIONAL REVIEWS
Date_____ Time_______

Estimate size and locate on the sketch below
☐ Are elongations specified on the plans Yes ☐ No ☐
☐ Are the tendons painted at the edge of the slab Yes ☐ No ☐
☐ What is the predetermined distance between the mark and the edge of the slab ____(in)
☐ Are the wedges placed in a vertical position Yes ☐ No ☐
☐ Is there evidence of gripper marks on the gripper end of all tendons Yes ☐ No ☐ **(If no show location on sketch below)**
☐ Are tendons stressed from two ends Yes ☐ No ☐If So, How Many ________



USE CHART IF ELONGATIONS ARE NOT LISTED ON PLAN, OR MULTIPLY TENDON LENGTH IN FEET BY 0.08 TO CALCULATE APPROXIMATE ELONGATION IN INCHES FOR LENGTH OVER 30 FEET.

SKETCH
Draw a simple sketch of the foundation configuration noting all tendon locations and their elongation measurements. Also note any problems which you have observed, particularly blowouts at corners or the garage entry and cracks.

FOLLOWING STRESS VERIFICATION
☐ Are the tendon ends cut inside the pocket former
☐ After stressing are the nails cut
☐ Are the tendon ends grouted with a non-shrink grout

Quality Controller's Signature **Superintendent's Signature**

CLIENT________________ **QUALITY CONTROL COMPANY**______________________________

QC Checklist #4–CONVENTIONAL (REBAR) FOUNDATION MAKE-UP

Builder__________________________ Subdivision____________________________ Date__________ Time__________
Site Address_____________________ Lot________ Blk________ Sec________ Plan site specific Yes ☐ No ☐
Plan #:________ Design Engineer_____________________________ Superintendent____________________
Plan provided at site Yes ☐ No ☐ Weather____________ Plan Date___________ Detail Sheet Date _____________
Concrete Placement Date_____________ Detached Garage Yes ☐ No ☐ Permit #____________________

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

SITE
Subdivision Lot _____ Other ____________________
Lot Description_____________________________
Fill on site Yes ☐ No ☐
Compaction verified by Geotechnical Engineer:
Yes ☐ No ☐ Date__________
Will make up drain: Yes ☐ No ☐
Trees removed_______________________________
Are trees within 20’ of foundation Yes ☐ No ☐

FORMS
☐ Forms secure
☐ Floats installed
☐ Proper clearance at floats
☐ Garage front closed

ADDITIONAL REVIEWS
Date_____ Time_______

SLAB
☐ Thickness____________(in)
☐ Measured: Screeds_____ Stringline______ Other_______
☐ Describe Pad Material ________________________
☐ Level and Firm Yes ☐ No ☐

BEAMS
☐ Design Depth: ______(in) Exterior____ Interior_____
☐ Actual Depth:______(in) _____(in) _____(in) _____(in)
☐ Design Width:______(in)
☐ Actual Width:______(in) _____(in) _____(in) _____(in)
☐ Average depth into undisturbed soil_______________(in)
☐ Clean of loose soil & debris
☐ Water in beams Yes ☐ No ☐ Average Depth_____(in)
☐ Will water drain Yes ☐ No ☐
☐ Plumbing obstructions accommodated _____________
☐ Pier tops clean Yes ☐ No ☐

POLYETHYLENE SHEETING
☐ 6-mil.Lapped and Taped ☐ Seated in the bottom of beams secured at sides ☐ Mastic/tape applied at plumbing

CONSTRUCTION PIERS
Number of piers _______ Are pier tops clean of debris Yes ☐ No ☐

REINFORCING STEEL
Grade of Steel ________

BEAM SECTIONS
Exterior Beams: Steel size_______ Number top _______ Bottom _______ Stirrup size _______ Spacing _______(in)
Interior Beams: Steel size_______ Number top _______ Bottom _______ Stirrup size _______ Spacing _______(in)
Extra Beam depth Yes ☐ No ☐ Additional steel required ____________________________
Proper Clearance: Bottom _______ (in) Sides ________(in) Top _______(in) Support System _______________
Continuity: Splices lapped per plan Yes ☐ No ☐ Corner bars installed Yes ☐ No ☐
Rebar clean of mud and excessive rust Yes ☐ No ☐
Void Boxes in bottom of beam Yes ☐ No ☐ Height _____(in) Condition_____________________________

SLAB REINFORCING
Mesh: Size_______ Roll_______ Sheet_________ **OR** ☐ #3 @_____________(in.) on center both ways
☐ All mesh seams lapped 6” ☐ #3 Lapped per plans ☐ All edges 2” from the forms
☐ No rebar or mesh touching forms
Void Boxes Yes ☐ No ☐ Height ___(in) Poly covering void boxes Yes ☐ No ☐

ADDITIONAL REINFORCING
Diagonals: Size ___________________ Number in slab_____________
Fireplace pads: Size of steel _________ Placement________________
Bay windows: Size of steel _________ Placement________________
Other projections: ___________________ Control joints______________
Construction joints: ______________________________________
Anchor bolts on site Yes ☐ No ☐ Diameter ______(in) Length________ (in)
Other Fasteners______________________

IS THE FOUNDATION READY FOR CONCRETE PLACEMENT? Yes ☐ No ☐ **SKETCH**
CHANGES NEEDED: ____
____
____

Quality Controller’s Signature **Superintendent’s Signature**

CLIENT__________________ **QUALITY CONTROL COMPANY**______________________________

QC Checklist #5 – CONSTRUCTION (BUILDER'S) PIERS

Builder____________________________Subdivision______________________________Date__________Time__________
Site Address________________________Lot_____Blk_____Sec______ Plan site specific Yes ☐ No ☐
Plan #:_________Design Engineer____________Superintendent________________Geotechnical Engineer ________________
Plan provided at site Yes ☐ No ☐ Plan Date_________Detail Sheet Date _________
Weather at site____________________ Concrete Contractor__________________ Geotechnical Report #______________

(THIS FORM NOT APPLICABLE FOR SLURRY PLACED PIERS)

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

SITE
Subdivision Lot _____Other _____________ Explain______________________
Fill on site Yes ☐ No ☐
Compaction verified by Geotechnical Engineer Yes ☐ No ☐ Date____________
Trees removed Yes ☐ No ☐ Location:____________________________
Are trees within 20' of foundation Yes ☐ No ☐

ADDITIONAL REVIEWS
Date_____ Time_______

PIERS
Name of drilling company: ______________________________
Can drill equipment access all pier locations Yes ☐ No ☐
Type of drilling apparatus: Truck Mounted__________________ Bobcat:_________________ Other:__________________
Total number of piers: ________

PIER SIZES

Shaft	Bell Dia.	Pier Depth	No. Rebar	Rebar Size	Stirrups Piers	Spacing	Total
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______

Sketch Typical Pier Showing Depth

Describe the manner of measuring the bell sizes:____
(Bell checking tool required)
Boring logs from Geotechnical report on site Yes ☐ No ☐
Describe bearing strata: ____
____
Pocket Penetrometer reading taken from auger cutting Yes ☐ No ☐ _____ TSF Note locations below________________
Was water apparent in pier hole Yes ☐ No ☐ Depth________" Action Taken____________________________

REINFORCING

Rebar placed per plan Yes ☐ No ☐
Rebar grade______________________
Does rebar extend above pier top Yes ☐ No ☐ How much above__________(in) Sleeved Yes ☐ No ☐ Describe___________

CONCRETE
Will concrete truck be able to access site Yes ☐ No ☐
Concrete company:_______________ Truck numbers:_____________
Was pump truck used Yes ☐ No ☐
Specified strength of concrete: __________ psi
Was concrete placed on the same day as the pier drilling Yes ☐ No ☐
Estimated time of completion___________
If not, explain: _________________________________
Draw a sketch of the structure indicating the pier placement

SKETCH

ARE THE PIER HOLES READY FOR CONCRETE PLACEMENT Yes ☐ No ☐
CHANGES NEEDED: ____
____
____

Quality Controller's Signature **Superintendent's Signature**

CLIENT________________ **QUALITY CONTROL COMPANY**______________________________

QC Checklist #6 – REPAIR PIERS

Owner__________________________Subdivision_________________________Date__________Time__________
Site Address_______________________Lot______Blk______Sec______ Plan site specific Yes ☐ No ☐
Plan #:_________Design Engineer_____________Superintendent________________Geotechnical Engineer ________________
Plan provided at site Yes ☐ No ☐ Plan Date_________Detail Sheet Date _________
Weather at site_______________________Permit #_________________________Geotechnical Report #___________________

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

SITE
Subdivision Lot _____ Other _____________ Explain______________________
Soils Report on site Yes ☐ No ☐ Bearing Soils at what depth _______________(ft)
Test hole drilled to what depth _______________(ft) Bearing soils at __________(ft)
Underground plumbing test Yes ☐ No ☐ Water lines under slab Yes ☐ No ☐
Site obstructions to drilling, Describe:____________________________________
Trees removed Yes ☐ No ☐ Location___________________________________

ADDITIONAL REVIEWS
Date_____ Time_______

UNDERPINNING
Name of repair contractor: ______________________________
Method of repair:_____________________________________
Total number of piers: ________ Interior _____ Exterior _______

PIER SIZES

Shaft	Bell Dia.	Pier Depth	No. Rebar	Rebar Size	Stirrups Piers	Spacing	Total
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______
______(in)	______(in)	______ (ft)	______	______	______	______ (in)	______

Sketch Typical Pier Showing Depth

Describe the manner of measuring the bell sizes:____
(Bell checking tool required)
Describe bearing strata: ____

Pocket Penetrometer reading Yes ☐ No ☐ ________ TSF Note locations below______________________
Was water apparent in pier hole Yes ☐ No ☐ Depth________" Action Taken___________________________

REINFORCING
Rebar per plans Yes ☐ No ☐
Rebar grade_____________________

HELICAL PIERS
Test hole depth __________________(ft) Bearing Data __________________________ Pier Log Onsite Yes ☐ No ☐
Helix Size ______________________ Bracket Style __________________________ Shaft Diameter _____________

CONCRETE
Will concrete truck be able to access site Yes ☐ No ☐ Was pump truck used Yes ☐ No ☐
Concrete company:________________ Truck numbers:___________________ Batch Time ________ Onsite Time______
Specified strength of concrete: _______ psi Slump as delivered _______________ Water added Yes ☐ No ☐ Amount ________
Was concrete placed on the same day as the pier was belled Yes ☐ No ☐
Projected time of completion of concrete placement __________
If not, explain: ____
ESTIMATED MAXIMUM LIFT __________INCHES: TO BE GROUTED Yes ☐ No ☐
Draw a sketch of the structure indicating the pier placement

SKETCH

ARE THE PIER HOLES READY FOR CONCRETE PLACEMENT Yes ☐ No ☐
CHANGES NEEDED: ____
____
____

Quality Controller's Signature **Superintendent's Signature**

CLIENT____________________ **QUALITY CONTROL COMPANY**________________________________

QC Checklist #7 – SEGMENTED REPAIR PILES

Builder____________________________Subdivision______________________________Date__________Time__________
Site Address________________________Lot______Blk______Sec______ Plan site specific Yes ☐ No ☐
Plan #:_________Design Engineer_____________Superintendent________________Geotechnical Engineer ________________
Plan provided at site Yes ☐ No ☐ Plan Date_________Detail Sheet Date _________
Weather at site______________________Permit #___________________________Geotechnical Report #________________

Check (✓) If Items Comply With The Plans
(X) If Items Do Not Comply With The Plans

SITE

Subdivision Lot _____ Other _____________ Explain_____________________
Geotechnical Report on site Yes ☐ No ☐ Bearing Soils at what depth _________(ft)
Test hole drilled to what depth ______________(ft) Bearing soils at _________(ft)
Underground plumbing test Yes ☐ No ☐ Water lines under slab Yes ☐ No ☐
Site obstructions to drilling, Describe:___________________________________
Trees removed Yes ☐ No ☐ Location___________________________________
Were builder's piers present Yes ☐ No ☐

ADDITIONAL REVIEWS

Date_____ Time_______

UNDERPINNING

Name of repair contractor: _______________________________
Piling system:____
Total number of piles: _______ Interior _____ Exterior _______

FIELD OBSERVATIONS

Pile Size Round	Pile Size Square	(A) Segment Length	(B) Number of Segments	(C) Pile Cap Size	(D) Pile Cap Quantity	(E) Distance From Top of Slab To Top of Pile Cap	Total Depth From Top of Slab	Observed Measurement of Lift at Refusal
______(d)	______(in)	______ (in)	______	______	______	______	______ (ft)	______ (in)
______(d)	______(in)	______ (in)	______	______	______	______	______ (ft)	______ (in)
______(d)	______(in)	______ (in)	______	______	______	______	______ (ft)	______ (in)
______(d)	______(in)	______ (in)	______	______	______	______	______ (ft)	______ (in)
______(d)	______(in)	______ (in)	______	______	______	______	______ (ft)	______ (in)

(A x B) + (C x D) + E = TOTAL DEPTH

Total number of pilings observed driven to completion _______ *(Minimum five is recommended)*
Was pile log available at the site Yes ☐ No ☐ Explain ____
Were the piles shimmed immediately upon completion of being driven Yes ☐ No ☐
If no, explain ____
Is the piling cap horizontal Yes ☐ No ☐ If no, explain ____
Were the piles driven without interruption Yes ☐ No ☐ If no, explain ______________________________
Were builders piers detached prior to jacking Yes ☐ No ☐
Were final shims determined to be tight Yes ☐ No ☐
What is the method of interlock ____
Were interior piles installed Yes ☐ No ☐ If so, were tunnels used Describe ___________________________
Was dewatering system used and maintained in excavating and tunnels Yes ☐ No ☐
Describe materials used in backfilling tunnels ____
Describe method of protecting tunnel entrance from water intrusion ______________________________
Was jetting required to install piles Yes ☐ No ☐ Explain____
ESTIMATED MAXIMUM LIFT __________INCHES: TO BE MUD PUMPED Yes ☐ No ☐

Draw a sketch of the structure indicating the pile placement

CHANGES NEEDED: ____
____
____

Quality Controller's Signature **Superintendent's Signature**

Recommended Practice for the Design of Residential Foundations

Version 1

By the Texas Section
American Society of Civil Engineers

Forward to Version 1

The Texas Section of the American Society of Civil Engineers (ASCE) adopted Guidelines for residential foundation engineering on October 3, 2002, with an effective date of January 01, 2003. The Section began this work in 1999.

This effort grew out of the response of many Section members to the Policy Advisory issued by the Texas Board of Professional Engineers (TBPE) in 1998, which addressed residential foundation engineering. Many ASCE practitioners expressed the opinion that technical guidelines should more rightly be created by a technical society such as ASCE rather than by the TBPE. One goal of the guidelines has been to provide the TBPE with guidance in their evaluation of complaints brought against engineers practicing residential foundation engineering.

The committees were composed entirely of ASCE members who were licensed engineers. The dollar value of the professional services donated to the effort is conservatively estimated to exceed $1,000,000.

The Guidelines are not intended to be Standards, but are guidelines only, reflecting the engineering opinions and practices of the committee members. They in no way replace the basic need for good engineering judgment based on appropriate education, experience, wisdom, and ethics in any particular engineering application. Thus, they are primarily suited as an aid for and use by engineers.

Members of the Residential Foundation Investigation and Design Subcommittee:

Philip G. King, PE, Chair

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Table of Contents

Recommended Practice for the Design of Residential Foundations – Version 1

By the Texas Section of the American Society of Civil Engineers

Section 1. INTRODUCTION

1.1 Objective

The function of a residential foundation is to support the structure. The majority of foundations constructed in Texas consist of shallow, stiffened and reinforced slab-on-ground foundations. Many are placed on expansive clays and/or fills. Foundations placed on expansive clays and/or fills have an increased potential for movement and resulting distress.

National building codes have general guidelines, which may not be sufficient for the soil conditions and construction methods in the State of Texas. The purpose of this document is to present recommended practice for the design of residential foundations to augment current building codes to help reduce foundation related problems. Where the recommendations in this document vary from published methods or codes, the differences represent the experience and judgment of the majority of the committee members.

On sites having expansive clay, fill, and/or other adverse conditions, residential foundations shall be designed by licensed engineers utilizing the provisions of this document. Expansive clay is defined as soil having a weighted plasticity index greater than 15 as defined by Building Research Advisory Board (BRAB) or a maximum potential volume change greater than 1 percent. This provision should also apply where local geology or experience indicates that active clay soils may be present. We propose that local and state governing bodies adopt this recommended practice.

1.2 Limitation

This recommended practice has been developed by experienced professional engineers and presents practices they commonly employ to help deal effectively with soil conditions that historically have created problems for residential foundations in Texas. This recommended practice presumes the existence of certain standard conditions when, in fact, the combination of variables associated with any given project always is unique. Experienced engineering judgment is required to develop and implement a scope of service best suited to the variables involved. For that reason, the developers of this document have made an effort to make the document flexible. Thus, successful application of this document requires experienced engineering judgment; merely following the guidelines may not achieve a satisfactory result. Unless adherence to this document is made mandatory through force of law or by contractual reference, adherence to it shall be deemed voluntary. This document does not, of itself, comprise the standard of care which engineers are required to uphold.

1.3 Adopted Changes

The Texas Section of the American Society of Civil Engineers (ASCE) has adopted procedures for changing the guidelines. In general, those interested in submitting changes for consideration by the Section should access the website at www.texasce.org, and follow the instructions for submitting changes. Changes may also be submitted in writing to the Texas Section - ASCE, 3501 Manor Road, Austin, 78723, phone 512.472.8905, fax 512.472.2934. Anonymous changes will not be considered. Those submitting changes should include contact information, state why a change is proposed, include applicable calculations if appropriate, and provide alternative language to incorporate the change. The appropriate committee will consider the changes, and from time to time the Texas Section may adopt the changes and issue revised Guidelines.

Readers should check with the Texas Section ASCE to make sure they are using the most recent version.

Section 2. DEFINITION OF "ENGINEERED FOUNDATION"

An engineered foundation is defined as one for which design is based on three phases:

a. geotechnical engineering information
b. the design of the foundation is performed by a licensed engineer
c. construction is observed with written documentation

These phases are described herein.

Section 3. DESIGN PROFESSIONALS' ROLES AND RESPONSIBILITIES

3.1 Geotechnical Services

Prior to foundation design, a geotechnical investigation and report shall be completed by a geotechnical engineer.

3.2 Design Services

The foundation design engineer shall prepare the plans and specifications for the foundation, and shall be the engineer of record. The foundation shall be built in accordance with the design. The engineer of record shall approve any design modifications. The geotechnical and foundation design engineering may be performed by the same individual.

3.3 Construction Phase Services

The engineer of record shall specify on the plans that construction phase observations shall be incorporated into the foundation construction. These activities shall be performed by: the engineer of record or a qualified delegate. The qualified delegate may be a staff member under his/her direct supervision, or outside agent approved by the engineer of record. The observation reports shall be provided to the engineer of record. The engineer of record shall issue a compliance letter as described in Section 6.3.

Section 4. GEOTECHNICAL INVESTIGATION

4.1 Minimum Field Investigation Program

The geotechnical engineer, in consultation with the engineer of record, if available, shall lay out the proposed exploration program. A minimum exploration program for subdivisions shall cover the geographic and topographic limits of the subdivision, and shall examine believed differences in geology in sufficient detail to provide information and guidance for secondary investigations, if any. The geotechnical exploration program should take into account site conditions, such as vegetation, depth of fill, drainage, seepage areas, slopes, fence lines, old roads or trails, man-made constructions, the time of year regarding seasonal weather cycles and other conditions that may affect foundation performance.

As a minimum for unknown but believed to be uniform subsurface conditions, borings shall be placed at maximum 300-foot centers across a subdivision. Non-uniform subsurface conditions may require additional borings. One soil boring may be sufficient for a single lot investigated in isolation for a simple residence under 2500 square feet. However, more borings may be required on sites having fill, having large footprints, or noticeably varying geological conditions such as steep slopes or locations near known fault zones or geological transitions.

Borings shall be a minimum of 20 feet in depth unless confirmed rock strata is encountered at a lesser depth. However, if the upper 10-ft of soils are found to be predominately cohesionless, then the boring depth may be reduced to 15 ft.. Borings shall extend through any known fill or potentially compressible materials even if greater depths are required.

All borings shall be sampled at a minimum interval of one per two feet of boring in the upper 10 feet and at 5-foot intervals below that. In clayey soil conditions, relatively undisturbed tube samples should be obtained. In granular soils, samples using Standard Penetration Tests should be obtained. Borings shall be sampled and logged in the field by a geotechnically-trained individual and all borings shall be sampled such that a geotechnical engineer may examine and confirm the driller’s logs in the laboratory.

Exploration may either be by drill rig or by test pit provided the depth requirements are satisfied. Sites, which are obviously rock with outcrops showing or easily discoverable by shallow test pits, may be investigated and reported without resorting to drilled borings.

Field logs shall note inclusions, such as roots, organics, fill, calcareous nodules, gravel and man-made materials. If encountered, the depth to water shall be logged. If the geology or site conditions indicate, overnight water levels shall be recorded prior to backfilling boreholes. Additional measurements shall be taken at the direction of the geotechnical engineer.

4.2 Minimum Laboratory Testing Program

The geotechnical engineer, in consultation with the engineer of record, if available, shall develop the laboratory-testing program. Sufficient laboratory testing shall be performed to identify significant strata and soil properties found in the borings across the site. Such tests may include:

a. Dry Density
b. Moisture Content
c. Atterberg Limits
d. Pocket Penetrometer Estimates of Cohesive Strength
e. Torvane
f. Strength Tests
g. Swell and/or Shrinkage Tests
h. Hydrometer Testing
i. Sieve Size Percentage
j. Soil Suction
k. Consolidation

All laboratory testing shall be performed in general accordance with the American Society for Testing and Materials (ASTM) or other recognized standards.

4.3 Geotechnical Report

4.3.1 Report Contents

Geotechnical reports shall contain, as a minimum:

a. purpose and scope, authorization and limitations of services
b. project description, including design assumptions
c. investigative procedures
d. laboratory testing procedures
e. laboratory testing results
f. logs of borings and plan(s) showing boring locations
g. site characterization
h. foundation design information and recommendations
i. Professional Engineer’s seal

4.3.2 Site Characterization

The geotechnical engineer shall characterize the site for design purposes. The report shall comment on site conditions which may affect the foundation design, such as:

a. topography including drainage features and slopes
b. trees and other vegetation
c. seeps
d. stock tanks
e. fence lines or other linear features

f. geologic conditions
g. surface faults, if applicable
h. subsurface water conditions
i. areas of fill detected at the time of the investigation
j. other man made features

4.3.3 Foundation Design Information and Recommendations

Reports shall contain the applicable design information and recommendations requested by the engineer of record for each lot in the project. If the engineer of record is not known at the time of the geotechnical report, the following design information should be presented, if applicable.

4.3.3.1 Soil movement potential as determined by the estimated depth of the active zone in combination with at least two of the following methods (identify each method used):

a. Potential Vertical Rise as determined by the Texas Department of Transportation Method 124-E, dry conditions
b. Swell tests
c. Suction and hydrometer tests
d. Linear Shrinkage tests
e. Any other method which can be documented and defended as good engineering practice in accordance with the principles of unsaturated soil mechanics

4.3.3.2 BRAB design information including:

a. Climatic Rating (C_w) of the site
b. Weighted Plasticity Index
c. Bearing capacity of the soil

4.3.3.3 Post-Tensioning Institute (PTI) parameters (using their most current design manual and technical notes) including:

a. e_m and y_m for edge lift and center lift modes (The e_m and y_m in the PTI design manual are based on average climate controlled soil movements and the design recommendations should take into account the added effect of trees and other environmental effects, as noted in the PTI design manual).
b. Bearing capacity of the soil.
c. If suction values are used to determine the depth and value of suction equilibrium or evaluate special conditions such as trees, the values shall be determined using laboratory suction tests. y_m determination shall be based on suction profile change and laboratory determined values of suction-compression index.
d. e_m and y_m shall be reported for design conditions for suction profile varying from equilibrium, and for probable extreme suction conditions.

4.3.3.4 Wire Reinforcing Institute (WRI) parameters including:

a. Climatic Rating (C_w) of the site
b. Weighted Plasticity Index
c. Slope Correction Coefficient (C_s)
d. Consolidation Correction Coefficient (C_o)

4.3.3.5 Deep Foundation (pier/pile) design information including:

a. Bearing capacity and skin friction along the pier length
b. Pier types and depths, and bearing strata
c. Uplift pressures on the pier and estimated depth of active zone (pier depth must be below the active zone and provide proper anchorage to resist the uplift pressures)
d. Down drag effects on the piers

4.3.3.6 Shallow foundations (including post and beam footings) design parameters.

a. Bearing capacity and footing depth
b. Minimum bearing dimension

4.3.3.7 Soil treatment method(s) to reduce the soil movement potential and the corresponding reduction in predicted movement.

4.3.3.8 Lateral pressures on any retaining structures or on piers undergoing lateral forces.

4.3.3.9 Trees and other site environment concerns that may affect the foundation design. Information useful for design and construction of residential foundations is presented in Appendix A.

4.3.3.10 Moisture control procedures to help reduce soil movement.

4.3.3.11 Surface drainage recommendations to help reduce soil movement.

4.3.3.12 Potential for load induced settlement.

4.3.3.13 On sloping sites, recommend whether a slope stability analysis is required due to possible downhill creep or other instability that may be present.

4.3.3.14 The presence and methods of dealing with existing and proposed fill. Fill criteria useful for design and construction of residential foundations is presented in Appendix B.

4.3.3.15 Geotechnical considerations related to construction.

Section 5. DESIGN OF FOUNDATIONS

5.1 Design Information

The foundation design engineer shall obtain sufficient information for the design of the foundation. This may include:

a. information gathered by a site visit
b. the subdivision plan, site plan or plat
c. the topography of the area including original and proposed final grades
d. the geotechnical report
e. special requirements of the project
f. the project budget
g. the architectural elevations and floor plans and sufficient additional architectural information to determine the magnitude, construction materials and location of structural loads on the foundation
h. exposed or architectural concrete schedule, if applicable

5.2 Design Procedures for Slab on Ground

5.2.1 The foundation engineer shall utilize one of the following methods, with the modifications presented in this section, as a minimum:

a. BRAB
b. Finite Element
c. PTI
d. WRI
e. other methods which can be documented and defended as good engineering practice

5.2.2 Input variables for residential slab-on-ground foundations shall be as follows:

5.2.2.1 BRAB:

a. Use the current design manual and technical notes, and the following design provisions:
   a.1 Regardless of the actual beam length, the analysis length should be limited to a maximum of 50 ft; and
   a.2 Use a maximum long-term creep factor as provided in ACI 318, Section 9.5.2.5.

5.2.2.2 Finite Element:

a. Use soil support parameters that can be documented and defended as good engineering practice in accordance with the principles of unsaturated soil mechanics;
b. Use a cracked moment of inertia for beams that exceed the cracking moment; and

c. Use a maximum design deflection ratio of 1 / 360 (deflection ratio is defined as the maximum deviation from a straight line between two points divided by the distance between the two points).

5.2.2.3 PTI:

a. Use the current design manual and technical notes, and the following design provisions.
b. Provide minimum residual average prestress of 100 psi.
c. Maintain the calculated prestress eccentricity within 5.0 inches. Bottom beam reinforcing should always be used.
d. If the computed concrete tensile stress at service loads, after accounting for prestress losses, exceeds $4\sqrt{f'_c}$, provide bonded additional reinforcement at the top or bottom of the beam as required by tensile forces equal to 0.0033 times the gross beam section. The transformed area of steel may be used to determine a new stiffness value for the beam.
e. The e_m and y_m in the PTI design manual are based on average climate controlled soil movements and the design analysis should take into account the added effect of trees and other environmental effects, as noted in the PTI design manual.

5.2.2.4 WRI:

a. Use the current design manual and technical notes, and the following design provisions.
b. Regardless of the actual beam length, the analysis length should be limited to a maximum of 50 ft; and
c. The minimum design length (L_c) shall be increased by a factor of 1.5 with a minimum increased length of 6 ft.

5.2.3 Design Considerations

The foundation design engineer should consider the following (deviation shall be based on generally accepted engineering practice)**:**

5.2.3.1 The latest ACI publications.

5.2.3.2 Exterior corners may require special stiffening. This can be accomplished with diagonal beams or parallel interior beams near the perimeter beams.

5.2.3.3 Provide continuous beams at reentrant corners. For post tensioned foundations, all exterior and interior beams should be continuous. For conventionally reinforced beams, interior beams may be discontinuous as long as the beam is continued a distance equal to at least twice the L_c distance.

5.2.3.4 Provide stiffening beams perpendicular to offsets (such as fireplaces or bay windows) in perimeter beams when the offset exceeds 18-inches.

5.2.3.5 Provide interior beams at concentrated loads such as fireplaces, columns and heavy interior line loads.

5.2.3.6 Sites with soil movement potential (see Section 4.3.3.1) exceeding 1.0 inch should have special design considerations such as strengthened sections, revised footprint, site soil treatment, or structurally suspended foundation if any of the following conditions is present:

a. a shape factor (SF) exceeding 20, (SF = perimeter squared divided by area)
b. extensions over 12 ft.

5.2.3.7 Slab-on-ground foundations with piers shall be designed as stiffened soil supported slabs for heave conditions and as structurally suspended foundations with the beams and slabs spanning between piers for shrinkage and settlement conditions. Piers shall not be attached to the slabs or grade beams unless the connections and foundation systems are designed to account for the uplift forces.

5.3 Design Procedures for Structurally Suspended Foundations

5.3.1 Structurally suspended floors supported by deep foundations shall be designed in accordance with applicable building codes.

5.4 Design Procedures for Footing Supported Foundations

5.4.1 Design in accordance with applicable building codes.

5.4.2 Shallow individual or continuous footing foundations should not be used on expansive soils, unless the superstructure is designed to account for the potential foundation movement.

5.5 Minimum Foundation Plan and Specification Information

5.5.1 Plans shall be signed and sealed by the engineer of record, and be specific for each site or lot location. Plans shall identify the client's name and engineer's name, address and telephone number; and the source and description of the geotechnical data.

5.5.2 The engineer's drawings shall contain as a minimum:

a. a plan view of the foundation locating all major structural components and reinforcement
b. sufficient information to show details of beams, piers, retaining walls, drainage details, etc., if such features are integral to the foundation
c. sufficient information for the proper construction and observation by field personnel
d. information or notes addressing minimum perimeter and lot drainage requirements

5.5.3 The engineer's specifications shall include as a minimum:

a. descriptions of the reinforcing or pre-stressing cables and hardware;
b. concrete specifications including compressive strengths;
c. site preparation requirements;
d. notes concerning nearby existing or future vegetation and the required design features to accommodate these conditions; and
e. the schedule of required construction observations and testing.

5.5.4 The engineer's plan shall address site fill:

a. The plans shall address fill existing at the time of the design or to be placed during construction of the foundation and shall require any fills which are to support the bearing elements of the foundation to be tested and approved by a geotechnical engineer assisted by a qualified laboratory (Bearing elements of a suitably designed slab-on-ground foundation are defined as the bottoms of exterior or interior stiffener beams.)
b. The plan shall require that a geotechnical engineer issue a summary report describing the methods, and results of investigation and testing that were used, and a statement that the existing or placed fills are suitable for support of a shallow soil-supported slab-on-ground, or that the foundation elements should penetrate the fill to undisturbed material. See Appendix B for more detailed information on fills.

Section 6. CONSTRUCTION PHASE OBSERVATIONS

6.1 Responsibility for Observations

Construction phase observations and testing shall be performed in accordance with this document.

6.2 Minimum Program of Observation and Testing

At a minimum, foundations should be observed and tested as applicable to determine whether:

a. exposed subgrade soils are prepared in accordance with the plans and specifications;
b. fill material and placement are in accordance with the plans and specifications;
c. pier placement, size and depth meet plans and specifications;
d. foundation elements, including reinforcement, meet plans and specifications immediately before concrete placement;
e. concrete properties and placement meet plans and specifications;
f. for post tension slabs, stressing meets the specified elongation and stressing load of each tendon; and.
g. specified site grading and drainage has been constructed.

6.3 Compliance Letter

6.3.1 At the satisfactory accomplishment of all the requirements of the plans and specifications, the engineer of record shall provide a letter to the client indicating, to the best of his knowledge (which may be based on observation reports by a qualified delegate as defined in Section 3.3), the construction of the foundation was in substantial conformance with:

a. the minimum standards of practice presented in this document; and
b. the engineer's plans and specifications including any modifications or alterations authorized.

6.3.2 A non-compliance letter shall be issued if the construction of the foundation did not meet the requirements of Section 6.3.1.

APPENDIX A

IMPACT OF MOISTURE CHANGES ON EXPANSIVE SOILS

Most problems resulting from expansive soils involve swelling or shrinking as evidenced by upward or downward movement of the foundation producing distress to the structure. The difference between the water content at the time of construction and the equilibrium water content is an important consideration. Potential swell increases with lower initial moisture content, while potential shrinkage increases with higher initial moisture content. Moisture contents and shrink/swell movements may vary seasonally even after equilibrium is reached.

Precipitation and evapotranspiration control soil moisture and groundwater levels. A slab will greatly reduce the evapotranspiration rate beneath the slab and partially reduces the inflow due to precipitation or irrigation because of groundwater's ability to migrate laterally. Therefore, soils beneath a slab are frequently wetter than soils at the same depth away from the slab. However, a wet season may result in wetter conditions away from the slab than under the slab. With time and normal precipitation patterns, the soil moisture profile will return to its normal condition. Seasonal variations in soil moisture away from the slab will generally occur fairly quickly. Seasonal variations in soil moisture beneath the slab will be slower. In addition roots from trees and large vegetation will seasonally remove moisture from nearby soils.

Wetting of expansive soils beneath slabs can occur as a result of lateral migration or seepage of water from the outside. It can be aggravated by ponded water resulting from poor drainage around the slab or landscape watering. Leaking utility lines and excessive watering of soil adjacent to the structure can also result in foundation heave.

Foundations can experience downward movement as the result of the drying influence of nearby trees. As trees and large bushes grow, they withdraw greater amounts of water from the soil causing downward foundation movement. The area near trees removed shortly before construction may be drier and subject to localized heave.

Some construction and maintenance issues include the following:

a. In general, set top of concrete at least eight inches above final adjacent soil grade for damp proofing.
b. For adjacent ground exposed or vegetative areas, provide adequate drainage away from the foundation (minimum five percent slope in the first ten feet and minimum two percent slope elsewhere). The bottom of any drainage swale should not be located within four feet of the foundation. Pervious planting beds should slope away from the foundation at least two inches per foot. Planting bed edging shall allow water to drain out of the beds.
c. Gutters or extended roof eaves are recommended, especially under all roof valleys. For adjacent ground exposed or vegetative areas, all extended eaves or gutter down

spouts should extend at least two feet away from the foundation and past any adjacent planting beds.

d. Avoid placement of trees and large vegetation near foundations (taking into account the water demands of specific trees and vegetation).

APPENDIX B

IMPACT OF FILL ON FOUNDATIONS

B.1 Fill

Fill is frequently a factor in residential foundation construction. Fill may be placed on a site at various times. If the fill has been placed prior to the geotechnical investigation, the geotechnical engineer should note fill in the report. Fill may exist between borings or be undetected during the geotechnical investigation for a variety of reasons. The investigation becomes more accurate if the borings are more closely spaced. Occasionally, fill is placed after the geotechnical investigation is completed, and it may not be detected until foundation excavation is started.

If uncontrolled fill (see discussion below) is discovered later in the construction process, for instance, by the Inspector after the slab is completely set up and awaiting concrete, great expense may be incurred by having to remove reinforcing and forms to provide penetration through the fill. Therefore, it is important to identify such materials and develop a strategy for dealing with them early on in the construction process. Fill can generally be divided into three types: engineered fill, forming fill, and uncontrolled fill. These three types of fill are discussed below.

B.1.1 Engineered Fill

Engineered fill is that which has been designed by an engineer to act as a structural element of a constructed work and has been placed under engineering inspection, usually with density testing. Engineered fill may be of at least two types. One type is “embankment fill,” which is composed of the material randomly found on the site, or imported to no particular specification, other than that it be free of debris and trash. Embankment fill can be used for a number of situations if properly placed and compacted. “Select fill” is the second type of engineered fill. The term “select” simply means that the material meets some specification as to gradation and P.I., and possibly some other material specifications. Normally, it is placed under controlled compaction with engineer inspection. Examples of select fill could be crushed limestone, specified sand, or crusher fines, which meet the gradation requirements. Select underslab fill is frequently used under shallow foundations for purposes of providing additional support and stiffness to the foundation, and replacing a thickness of expansive soil. Engineered fill should meet specifications prepared by a qualified engineer for a specific project, and includes requirements for placement, geometry, material, compaction and quality control.

B.1.2 Forming Fill

Forming fill is that which is typically used under residential foundation slabs and is variously known as sandy loam, river loam or fill dirt. Forming fill is normally not expected to be heavily compacted, and a designer should not rely on this material for support. The only requirements are that this material be non-expansive, clean, and that it works easily and stands when cut. If forming fill happened to be properly compacted and inspected in accordance with an engineering specification it could be engineered fill.

B.1.3 Uncontrolled Fill

Uncontrolled fill is fill that has been determined to be unsuitable (or has not been proven suitable) to support a slab-on-ground foundation. Any fill that has not been approved by a qualified geotechnical engineer in writing shall be considered uncontrolled fill. Uncontrolled fill may contain undesirable materials and/or has not been placed under compaction control. Some problems resulting from uncontrolled fill include gradual settlement, sudden collapse, attraction of wood ants and termites, corrosion of metallic plumbing pipes, and in some rare cases, site contamination with toxic or hazardous wastes.

B.2 Building on Non-Engineered (Forming Or Uncontrolled) Fill

Foundations shall not be supported by non-engineered fill. To establish soil-supported foundations on non-engineered fill, the typical grid beam stiffened slab foundation is required to penetrate the non-engineered fill with the perimeter and interior beam bottoms forming footings. Penetration will take the load supporting elements of the foundation below the unreliable fill. Penetration could be accomplished by deepened beams, spread footings or piers depending on the depth and the economics of the situation. Generally, piers are most cost effective once the fill to be penetrated exceeds about three feet, but this depends on the foundation engineer's judgment and local practice. Floor systems shall be designed to span between structurally supported foundation elements.

Pre-existing fill may be classified as engineered fill after investigation by the geotechnical engineer. The approval may depend on the fill thickness, existence of trash and debris, the age of the fill, and the results of testing and proof rolling. The geotechnical engineer must be able to expressly state after investigation that the fill is capable of supporting a residential slab-on-ground foundation.

Guidelines for the Evaluation and Repair of Residential Foundations

Version 1

By the Texas Section
American Society of Civil Engineers

Forward to Version 1

The Texas Section of the American Society of Civil Engineers (ASCE) adopted Guidelines for residential foundation engineering on October 3, 2002, with an effective date of January 01, 2003. The Section began this work in 1999.

This effort grew out of the response of many Section members to the Policy Advisory issued by the Texas Board of Professional Engineers (TBPE) in 1998, which addressed residential foundation engineering. Many ASCE practitioners expressed the opinion that technical guidelines should more rightly be created by a technical society such as ASCE rather than by the TBPE. One goal of the guidelines has been to provide the TBPE with guidance in their evaluation of complaints brought against engineers practicing residential foundation engineering.

The committees were composed entirely of ASCE members who were licensed engineers. The dollar value of the professional services donated to the effort is conservatively estimated to exceed $1,000,000.

The Guidelines are not intended to be Standards, but are guidelines only, reflecting the engineering opinions and practices of the committee members. They in no way replace the basic need for good engineering judgment based on appropriate education, experience, wisdom, and ethics in any particular engineering application. Thus, they are primarily suited as an aid for and use by engineers.

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Guidelines for the
Evaluation and Repair of Residential Foundations – Version 1

By the Texas Section of the
American Society of Civil Engineers

Section 1. PURPOSE AND SCOPE

1.1 Introduction

The purpose of this document is to provide guidance for engineers practicing in the field of residential foundation evaluation and repair within the State of Texas with the goal of protecting the public when obtaining these services. The principal items discussed in this document are as follows:

1. An introduction presenting the background leading to the need for this document
2. Qualifications of engineers performing evaluations or repair designs
3. Scope of services
4. Methodology
5. Information typically presented in the evaluation report
6. Performance criteria for residential foundations
7. Foundation repair and remedial alternatives
8. Anticipated structure performance after remedial measures

1.2 Background

Texas has large areas with clayey soils that shrink and swell with changes in soil moisture content. This shrinking and swelling may cause movement of residential foundations that adversely affects the residence. Other factors may influence foundation performance. Some of these factors are inadequate design or construction, unanticipated loads, deterioration of materials, compressibility of the supporting soils, landscaping practices, leaking plumbing, and slope instability. The American Society of Civil Engineers, Texas Section (ASCE, TX) developed this document as a guideline for evaluation and repair of residential foundations. A separate document, *Recommended Practice for the Design of Residential Foundations,* also developed by ASCE, TX, addresses residential foundation design.

1.3 Objectives

The most common purpose of an engineering evaluation of a residential foundation is to assess its performance. This involves observation and evaluation of cosmetic (non-structural) distress and structural damage. The evaluation may also provide opinions of probable causes of distress or damage, assessment of risk of further damage, recommendations for remedial measures, and cost estimates. If the evaluation determines that remedial measures are appropriate, the engineer may be asked to provide the design and construction documents.

1.4 Limitation

These guidelines have been developed by experienced professional engineers and presents practices they commonly employ to help deal effectively with soil conditions that historically have created problems for residential foundations in Texas. These guidelines presume the existence of certain standard conditions when, in fact, the combination of variables associated with any given project always is unique. Experienced engineering judgment is required to develop and implement a scope of service best suited to the variables involved. For that reason, the developers of this document have made an effort to make the document flexible. Thus, successful application of this document requires experienced engineering judgment; merely following the guidelines may not achieve a satisfactory result. Unless adherence to this document is made mandatory through force of law or by contractual reference, adherence to it shall be deemed voluntary. This document does not, of itself, comprise the standard of care which engineers are required to uphold.

1.5 Adopted Changes

The Texas Section of the American Society of Civil Engineers (ASCE) has adopted procedures for changing the guidelines. In general, those interested in submitting changes for consideration by the Section should access the website at www.texasce.org, and follow the instructions for submitting changes. Changes may also be submitted in writing to the Texas Section - ASCE, 3501 Manor Road, Austin, 78723, phone 512.472.8905, fax 512.472.2934. Anonymous changes will not be considered. Those submitting changes should include contact information, state why a change is proposed, include applicable calculations if appropriate, and provide alternative language to incorporate the change. The appropriate committee will consider the changes, and from time to time the Texas Section may adopt the changes and issue revised Guidelines.

Readers should check with the Texas Section ASCE to make sure they are using the most recent version.

Section 2. QUALIFICATIONS OF THE ENGINEER

2.1 Professional Qualifications

The evaluation and repair design shall be performed by a professional engineer licensed in the State of Texas. Engineers in responsible charge of this type of work must be competent to apply scientific and engineering education, training, knowledge, skill and experience to the investigation and analysis of constructed facilities. This determines the cause and extent of diminished performance and the means of remediation. Engineers should be competent in the related disciplines or should retain outside consultants as needed.

2.2 Professional Ethics

It is essential to avoid conflicts of interest to maintain the credibility of the evaluation investigation. The evaluating engineer must demonstrate qualities of character that will ensure impartiality. These qualities include objectivity, confidentiality, honesty and integrity.

ASCE members subscribe to the ASCE Code of Ethics, which includes the Fundamental Principles, Fundamental Canons, and Guidelines to Practice Under the Fundamental Canons of Ethics. Professional Conduct and Ethics comprise a sub chapter of the Texas Engineering Practice Act.

Section 3. LEVELS OF INVESTIGATION

3.1 General

The engineer should recommend an appropriate level of investigation to fulfill the objective of the evaluation. However, the scope of services shall be jointly established and agreed to by both the client and engineer. The engineer should personally visit the site and be in responsible charge of the investigative activities. If requested by the client, the engineer may only provide evaluation of reports by others, but this should be described as consultation, not investigation. For the purpose of aiding the client in determining the type of evaluation desired or actually performed, the following three levels of investigation are offered as guidelines.

3.1.1 Level A

This level of investigation shall be clearly identified as a report of first impressions and shall not imply that any higher level of investigation has been performed. This level of investigation will typically include, but is not restricted to:

1. Interview the occupant, owner and client if possible, regarding a history of the property and performance of the structure
2. Request from the client and review the provided documents regarding the foundation, such as construction drawings, geotechnical reports, previous testing and inspection reports, and previous repair information
3. Make visual observations during a physical walk-through
4. Observe factors influencing the performance of the foundation
5. If requested by the client, provide a written report, containing at least the following:
    a. scope of services
    b. observations, site characteristics, and data deemed pertinent by the engineer
    c. discussion of major factors influencing foundation performance and rationale in reaching conclusions concerning the subject residence
    d. conclusions and any recommendations for further investigation and remedial or preventative measures

3.1.2 Level B

This level of investigation should include a written report including the items listed above for a Level A inspection and also the following items:

1. A determination of relative foundation elevations in sufficient detail to represent the shape of the foundation or floor adequately.
2. A drawing showing relative elevations

3.1.3 Level C

This level of investigation shall include the items listed above for Level A and Level B inspections and additional services, testing and related reports deemed appropriate by the Engineer. These may include, but are not limited to, the following:

1. Site specific soil sampling and testing
2. Plumbing testing
3. Material testing
4. Steel reinforcing survey
5. Post tensioning cable testing

This level of investigation should also include a more detailed level of reporting, which may include the following:

1. Scaled drawings
2. Description of factors that affect soil moisture
3. Observations of cut and fill
4. Tree survey
5. Photographs
6. Detailed distress survey

Section 4. EVALUATION METHODOLOGY

4.1 General

A rational method should be used to establish causes of distress or diminished performance, if any. A suggested method is summarized as follows:

1. Observe the structure, site conditions, other relevant phenomena, and collect pertinent data
2. Analyze the data
3. Formulate hypotheses
4. Test the hypotheses using analyses acceptable to the engineering profession along with engineering experience
5. Reach conclusions or reformulate the hypotheses

4.2 Analysis

Diminished performance of a structure may have several causes. The engineer should approach the analysis with an open mind. The analysis should follow a logical path to its conclusion. The evaluation should be quantitative to the extent practical, but should not assume greater accuracy or precision than warranted by the data.

Section 5. EVALUATION CRITERIA

5.1 General

Residential foundations are expected to remain reasonably flat and level to provide acceptable performance. The criteria herein are intended to lend rationality and reasonable uniformity, supported by a consensus of practitioners, to the evaluation of performance and the need for repair of residential foundations.

The bases of these evaluation criteria are structural integrity (strength) and performance (serviceability). Both may be affected by foundation deformation and tilt. Evaluations may be interpreted from the body of evidence or demonstrated by calculations.

5.2 Structural Integrity

In evaluating a foundation, structural integrity considers the capability of the foundation to support its design loads as well as results and effects on other load bearing members of the superstructure. Elements of concern are stability, component strength and condition, and material soundness. In evaluating structural integrity, it should be understood that in many instances portions of the foundation and other structural components may not be available for observation.

Lack of structural integrity may be indicated by excessive deflection, cracking, partial collapse, loss of section, material deterioration, or demonstrated by calculations. If loss of structural integrity is demonstrated by calculations, the conclusion must be consistent with the physical evidence. Examples of lack of structural integrity include loss of shear capacity in concrete through excessive cracking, excessive tilt of structural elements such as posts or piers, unstable conditions in non load-bearing masonry, and rotting of wood structural members. The engineer should evaluate the following, if they are observed:

1. Observed cracks. Cracks may make concrete structural members weaker, although the majority of cracks do not compromise structural integrity.
2. Tilting of posts or piers above grade. Tilting can affect structural integrity or stability, although posts or piers above grade designed for eccentricity of load can tolerate some tilting without overstress. However, ordinary construction tolerances may result in vertical members being built out of plumb.
3. Tilt, if any, of masonry veneer panels. Excessive tilt can lead to catastrophic panel collapse. Masonry veneer or infill is normally non load-bearing, and in some cases the veneer or infill may not be held in place except by its own weight. Wall tilt large enough to cause the weight vector (or center of gravity) to fall outside the middle third of bearing area is sufficient to cause tension in masonry veneer.

4. Observed material deterioration. The strength of deteriorated material may raise a structural integrity issue. Evaluation of material deterioration may be based on observation, material sampling and testing, or non-destructive methods.

5.3 Performance

Performance considers the capability of the building to serve its intended purpose. Elements of concern are safety, function, durability, and habitability. Inadequate performance may result from inadequate strength or insufficient stiffness, and is shown in many ways. Visible indications may include:

1. Cracking or separating of exterior walls
2. Rotating, buckling, or deflecting masonry veneer panels
3. Cracking of concrete foundation elements
4. Cracking of gypsum board walls and ceilings
5. Separating of walls from ceilings or floors
6. Separating of rafters from a ridge board
7. Racking of door and window frames
8. Separating or racking of other structural framing
9. Cracking, buckling, or separating of floor coverings
10. Separating of initially tight joints
11. Deflecting, deforming, or tilting of structural elements
12. Deteriorating materials

Observation of some of the listed conditions does not necessarily imply inadequate structural performance or insufficient stiffness.

5.4 Deflection and Tilt

Foundation deflection (bending or angular distortion) and tilt (planar rotation) may affect structural integrity and performance. Determining the deflection and tilt of a slab-on-ground foundation is an approximation without an as built or previous floor elevation survey, because the original surface configuration is unknown. Therefore, a floor elevation survey should not be the only basis for evaluating foundation deflection and tilt.

Deflection may be more difficult to evaluate quantitatively than any other element of performance. Deflection is characterized by the deflection ratio, which is defined as the maximum deviation from a straight line between two points divided by the distance (L) between the two points. Overall deflection, as defined below, may be more easily interpreted and evaluated than localized deflection. Localized deflection may be a more common occurrence.

5.5 Overall Deflection

Overall deflection necessarily involves the overall foundation dimension in a given direction. When additions have been made to a foundation, the overall foundation dimension should be considered for each separate foundation element and for the entire foundation. The amount of overall deflection is measured by the deflection ratio.

Building codes specify that structural members shall be designed to have adequate stiffness to limit deflections. The *International Code Council International Residential Code™* for One- and Two-Family Dwellings (IRC) specifies a maximum allowable live load deflection of L/360. This deflection criterion may be appropriate for the analogous in-service deflection of a residential foundation due to loading from varying soil conditions. The maximum live load deflection of a floor is the in-service deflection that typically will not result in excessive damage to cosmetic finishes.

A single floor level survey yields the shape of the foundation at one instant, and may or may not furnish sufficient information to support a conclusion. An evaluation may include repeated floor level surveys performed over months or years. In such cases, the change in shape is measured between surveys. In addition, previous foundation repairs may change elevation shapes.

The engineer evaluating deflection must consider the floor level survey (Levels of Investigation B or C), and other indications of movement, such as:

1. Brick coursing not level.
2. Poor door alignment.
3. Levelness of built in horizontal surfaces, such as cabinets, countertops, sills and trim.
4. Cracking of exterior and interior wall finishes may indicate deflection, as do most items listed in 5.3 above.

If a foundation profile indicates the deflection is less than the analogous deflection limit of L/360, it is unlikely the foundation is deflected materially unless visible indications show otherwise.

If a foundation profile indicates the deflection is more than the analogous deflection limit of L/360 and minimal symptoms of deflection are present, then additional information is needed by the engineer to develop a conclusion. The additional information may allow the engineer to determine whether or not the foundation has deflected excessively.

If a foundation profile indicates the deflection is more than the analogous deflection limit of L/360 and sufficient symptoms of deflection are present, then the engineer generally will be justified in determining that the foundation has deflected excessively.

5.6 Localized Deflection

Localized deflection means a change from original profile or shape in an area smaller than the overall foundation. Localized deflection manifests itself in similar ways as overall deflection. It sometimes results in localized structural integrity or performance problems. The engineer should evaluate the significance of localized deflections and their consequences as in Section 5.5, but caution is advised when evaluating floor deviations over only a few feet because built-in unevenness can dominate.

5.7 Tilt

Foundation tilt can affect structural integrity and performance. Tilt of entire foundations may be evaluated for structural integrity using the criterion stated for veneer panels, as discussed in Section 5.2 of this document. This criterion may be found in the 1997 Uniform Code for Abatement of Dangerous Buildings.

Floors may tilt enough to affect comfortable or convenient use of the building. A floor slope greater than 1 percent is usually noticeable. The Americans with Disabilities Act considers a 2 percent slope too large.

5.8 Remediation Criteria

If the residence is found to be unsafe due to structural inadequacies, the client and/or civil authorities should be informed immediately. The engineer should recommend repair, restoration, remediation, adjustment, or use alternatives if the structural integrity is inadequate. The engineer should provide alternatives for the client's consideration if performance is inadequate. Recommendations and alternatives should be commensurate with the nature and cause of the inadequacy, and the seriousness of its consequences.

The engineer should consider the cost effectiveness and practicality of the recommendations, the projected performance, and the needs of the client. For example, an owner may choose to perform periodic cosmetic repairs and door adjustments, rather than comprehensive foundation underpinning.

Risks of continued diminished performance are involved in all remedial measures. The engineer can, however, provide recommendations for remedial measures that reduce risks. Not implementing the entire remedial plan may increase such risks.

Section 6. REPORTING

The report provides a record of the investigation, analysis and conclusions. Report formats may vary, but should contain pertinent information that was obtained or generated during the investigation. The following list includes items that may be included in a report:

1. Authorization and Scope
2. Property Location and Description
3. Sources of Information
4. Data
5. Assumptions
6. Analysis of Information and Data
7. Conclusions
8. Recommendations
9. Limiting Conditions

Section 7. REMEDIAL MEASURES

7.1 Objectives and Limitations of the Remedial Measures

The objective of the engineer should be to design and recommend cost effective remedial measures. Remedial measures should address diminished structural integrity and performance identified during the evaluation process. Recommendations for remedial measures should include a clear description of what the remedial measures are intended to accomplish.

Perfection is not attainable by remedial measures. Recommendations for remedial measures should identify important or significant limitations of the measures, and should comment on reasonable expectations of the remedial measures.

7.2 Responsibility of the Engineer

The engineer who provides sealed remediation documents or plans and specifications shall be the engineer of record and shall have approval authority over any changes. The Texas Engineering Practice Act and Rules adopted by the Texas Board of Professional Engineers prohibits the practice known as “plan stamping” by requiring that engineers seal only work done by them or under their direct supervision.

7.3 Non-structural Remedial Measures

Non-structural remedial measures may improve foundation performance and reduce future movement. Applying non-structural remedial measures and monitoring foundation performance prior to or in lieu of structural repairs may be a prudent approach. Typical recommendations for non-structural remedial measures may include, but are not limited to, the measures listed below.

7.3.1 Conscientious Watering Program

The client should be informed that maintaining near uniform soil moisture conditions near all sides of the foundation may be beneficial. Caution should be advised against excessive watering.

7.3.2 Vegetation Alteration

Trees or large shrubs near a foundation may cause soil shrinkage under the foundation. Removal of these trees or shrubs may stop shrinkage or lead to partial restoration of settled areas of the foundation. Removal may result in upheaval caused by soil moisture increase, especially if the tree predates construction. If trees are removed, a suitable waiting period may be recommended to allow for soil heave.

7.3.3 Root Barriers

Root barriers or periodic root pruning may mitigate the effects of vegetation. Root barriers are generally not as effective as tree removal.

7.3.4 Gutters and Downspouts

Uncontrolled roof runoff can cause erosion and ponding of water near the structure, which can be mitigated by addition of gutters and downspouts. Downspouts should be extended well past the edge of the foundation, past the edge of abutting planting beds, and into well-drained areas.

7.3.5 Drainage Improvements

Drainage improvements may be appropriate to address foundation movement. If drainage improvements are considered, the following guidelines may be appropriate.

7.3.5.1 Surface Grading

Where practicable, for adjacent ground exposed or vegetative areas, a minimum slope of 5 percent (i.e. 6 inches in 10 feet) away from the foundation should be provided for the first 5 feet all around. Swales should have longitudinal slopes of at least 2 percent (i.e. 6 inches in 25 feet), if practicable, and 1 percent (i.e. 3 inches in 25 feet) at a minimum.

7.3.5.2 Erosion Control

The remedial documents should indicate locations where fill, ground cover or retaining structures are to be added.

7.3.5.3 Surface Water Drainage

When surface drainage cannot be improved adequately by grading, or when otherwise appropriate, solid pipe drainage systems should be specified. The ground surface should be graded to slope to one or more drainage inlets. Cleanouts should be provided for maintenance. Downspouts may be connected to solid pipe drainage systems, if the pipe is large enough for the hydraulic load of roof drainage.

7.3.5.4 Subsurface Water Drainage

Subsurface water drains are appropriate to control subsurface water, and usually consist of perforated pipe, with or without filter fabric, in an aggregate-filled trench. Provide a continuous minimum slope of 0.5 percent to a surface outfall. Cleanouts should be provided for maintenance. Downspouts should not be connected to perforated pipe subsurface drainage systems.

7.3.6 Moisture Barriers

Vertical or horizontal moisture barriers may be effective to mitigate moisture migration under the foundation. Moisture barriers may consist of durable impermeable plastic sheeting or other appropriate material attached to the foundation.

7.4 Structural Remedial Measures

Structural remedial measures may be necessary to improve foundation performance.

7.4.1 Structural Remedial Documents

The engineer should provide documents or plans and specifications that show specific details of the remedial measures. Plans should be specific for the project, and be based upon generally accepted engineering practice, including appropriate engineering calculations.

Remediation documents should include the following:

1. The site address
2. The engineer's name and the firm's name, address, and telephone number
3. The client's name and address
4. The purpose and limitations of the remedial measures
5. Available geotechnical information and source
6. A plan view of the foundation locating known relevant structural components
7. Details to show how to construct repair components
8. Specifications to identify appropriate materials and methods
9. Requirements for construction observation or testing by the engineer or others
10. Existing floor elevations or contours and elevation adjustment requirements, if appropriate
11. The requirement for performing a floor elevation survey after completion of the remedial measures
12. Site restoration requirements

7.4.2 Geotechnical Information

The engineer designing structural remedial measures will need geotechnical information. In some cases, geotechnical information may be derived from successful local practice, or other experience, verified during construction. For major or comprehensive remedial measures, geotechnical information should be derived from a site specific boring and testing program tailored to the project's needs.

7.4.3 Repair of Slab Foundations

Concrete slab-on-ground foundation repair methods include, but are not limited to: underpinning, grouting, mudjacking, crack injecting, tendon stressing, and partial demolition and reconstruction.

7.4.3.1 Underpinning

The plans should show or specify specific locations of underpinning elements and their sizes, depths, material types, and minimum required material strengths if appropriate. Underpinning design shall be based upon generally accepted engineering practice and appropriate engineering calculations. Performance of underpinning can be compromised by integrity of existing slab components, changes in soil moisture, skin friction, point load, and other factors.

Underpinning part of a structure may be specified if calculations, tests, or experience show that the unsupported structure can support its design loads. The construction documents should state that underpinning will not improve the performance of the foundation in non-underpinned areas.

Elevation adjustments by jacking or lifting atop underpinning elements may be applicable when floor slopes are excessive, or when the design requires that the foundation be lifted clear of expansive soil. Elevation adjustments should be governed by field judgment to limit damage to the foundation and finishes. It is unlikely that elevation adjustments will result in a level foundation.

7.4.3.2 Grouting and Mudjacking

In general, grouting provides continuous slab support without lifting appreciably. Mudjacking is done to adjust elevations of a foundation hydraulically with continuous uniform support. Grouting or mudjacking may be accomplished with temporary support atop shallow footings or long-term support atop deep piles or piers. Grouting or mudjacking should not be performed beneath underpinned foundations if expected swelling of the soil in the injected area is sufficient to damage the structure.

7.4.3.3 Crack Injecting

Injecting slab cracks of about 1/32 inch and larger with epoxy repair cement is intended to restore stiffness across the injected crack. If the objective of the repair is solely to limit moisture intrusion or insect ingress, then alternative materials, such as sealants, may be appropriate.

7.4.3.4 Tendon Stressing

Stressing relaxed or inadequately stressed post-tensioned tendons may be applicable when tests show tendon forces below those specified in the original design or by applicable authority. Stressing may restore the residual prestress in the concrete, and should be performed after elevation adjustments and epoxy crack injecting, if any.

7.5 Repair of Pier and Beam Foundations

Pier and beam foundations consist of structurally supported floor systems atop piers, posts or footings. Repairs may include shimming the floor framing atop the existing supports, repairing or strengthening the floor framing, replacing or adding supports, and re-establishing void space.

7.5.1 Floor Shimming

Floor framing may be adjusted by addition of shims atop pier caps. Hardwood or steel shims may be used to fill gaps.

7.5.2 Framing Repairs

Structural members that are damaged or distressed should be replaced or reinforced. Treated lumber is recommended for general use in framing repairs.

7.5.3 Additional Supports

Additional supports can be installed when beam or floor framing spans are too great for the design loads, or when existing supports have deteriorated or are otherwise ineffective.

7.5.4 Void Space

Void spaces designed under foundation elements should be reestablished as necessary.

7.5.5 Under-Floor Crawl Space Moisture Control

Under-floor moisture control measures include crawl space cross ventilation, under-floor drainage, floor beam and floor joist ground clearance, and treated lumber.

7.6 Post Lift Plumbing Testing

Water supply and sanitary drain lines should be tested for leaks if jacking or lifting is included in the remedial measures. Gas service lines may require adjustment. Leaks found by such testing should be repaired.

7.7 Floor Elevations

Floor elevation measurements should be made after implementation of remedial measures. The engineer should keep a record of these elevation measurements and furnish a copy to the client.

7.8 Compliance Letter

Upon satisfactory completion of the remedial measures, the engineer, if retained to do so, should provide a letter of substantial completion to the client stating that to the best of the engineer's knowledge, the remedial measures generally conform to the remediation documents, including approved changes. Deviations from the remediation documents should be noted in the letter.