Post-Tensioning Institute
Foundation Performance Association
and
Geotech Engineering and Testing

Present

Seminar on Design of Foundations on Expansive Soils

November 5, 1999

7:00 a.m. - 6:00 p.m.

at

Sheraton North Houston Hotel
Houston, Texas

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DESIGN OF FOUNDATIONS ON EXPANSIVE SOILS

PROGRAM AGENDA

Time	Topics
7:15 a.m.	Seminar Opening by David Eastwood Texas Board of Professional Engineers Policy Advisory by Mr. David Eastwood, P.E.
7:45 a.m.	Report of the Design Subcommittee of the Residential Foundation Committee, Texas Board of Professional Engineers by David A. Eastwood, P.E., Kirby T. Meyer, P.E.
8:30 a.m.	Post-Tensioning Institute - Mr. Gerald McGuire
8:45 a.m.	Introduction to Unsaturated Soils - Dr. Lytton
	Field Exploration and Site Conditions
	Laboratory Testing (Dr. Bryant) Swell Tests Soil Suction Tests Volume Change Coefficient
10:00a.m.	Break
	Computations of Swell and Shrinkage in Expansive Soils (Dr. Lytton) Potential Vertical Rise Soil Suction Estimates of Vertical Movement Horizontal Moisture Movement
	Updated VOLFLO Program by Mr. Kirby Meyer, P.E.
12:00-1:15 p.m.	Lunch
1:15 p.m.	Examples using VOLFLO, Dr. Bryant and Dr. Lytton
3:30 p.m.	Break
3:45 p.m	Design Concepts of Various Foundation Systems Drilled Footings Floating Slabs Moisture Barrier Root Barrier
5:00 p.m.	Questions and Answers Period
6:00 p.m	Adjourn

Biography

of

David A. Eastwood, P.E.

David Eastwood is the President of Geotech Engineering and Testing. Mr. Eastwood has practiced consulting engineering for about 21 years serving in key technical, project management, and administrative roles on both domestic and international assignments. His experience in these functions include a wide range of project types and large capital investments ranging from residential and industrial to commercial buildings. Geotech Engineering and Testing has been a leader in providing soils and foundation engineering services to the builders, developers, architects, and designers. Mr. Eastwood has conducted soils and foundation explorations and foundation studies for a wide variety of projects including a large number of residences, subdivisions, apartment buildings, shopping centers, and office buildings.

Mr. Eastwood received his Bachelor and Masters of Science in Civil Engineering from the University of Houston with specialization in soils engineering. Mr. Eastwood has attended Continuing Education Seminars at Rice, Princeton, University of Maryland, and the University of Houston.

He has several publications on the design and construction of foundations on expansive soils. Mr. Eastwood is a member of PTI, GHBA, AIA, ASTM, TSPE, TIBD, ACME, and ASCE. Mr. Eastwood is the Chairman of the Geotechnical Committee of Post-Tensioning Institute Slab-On-Grade Committee. Furthermore, he is the past President of the Foundation Performance Association. The mission of this organization is to serve the public by advancing the skill and the art of engineering analysis, investigation on light foundations.

Mr. Eastwood is also a member of the Residential Foundation Committee with the Texas State Board of Professional Engineers. The purpose of this committee is to investigate the engineering, economics, and ethical situations facing consumers and engineers with respect to the service and failure of residential foundations in Texas.

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Board Initiatives

All information provided via the links on the *Board Initiatives* site is for the purpose of informing the public and interested parties of the positions on issues and possible actions to be taken by the Texas Board of Professional Engineers. If you have *questions or comments*, you may send them in writing with the written signature of the commentator by e-mail, fax or regular mail. Since the Board meets quarterly throughout the year, these specific questions may take additional time for review by the Board.

Click on the headings below for:

Software Engineering Statement - This article is in the process of being rewritten. It will be available at a later date.

Board Establishes Software Engineering Discipline

Continuing Professional Competency (CPC)

Policy Advisory 09-98-A, Regarding Design, Evaluation and Repair of Residential Foundations. - This policy has been temporarily removed. The Residential Foundation Advisory Committee will meeting to discuss the current policy advisory. Once the Committee has finalized the new policy advisory it will be posted on this website.

For more information contact:

Texas Board of Professional Engineers

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www.main.org/peboard/
Last Updated: 10/99

POLICY ADVISORY

09-98-A
Regarding Design, Evaluation and Repair of Residential Foundations

Texas Board of Professional Engineers

I. Background & Purpose

Under the exemptions of Section 20(d) of the Texas Engineering Practice Act, any person who designs, constructs or repairs engineering features for a Texas residence does not need to be licensed as a professional engineer to legally perform that task. However, licensed professional engineers are actually performing a large number of the residential foundation designs, evaluations and repairs performed in Texas each year. According to data collected by the Real Estate Center at Texas A&M University, approximately 76,000 single-family residential building permits were issued on an annual basis since 1995, representing a significant impact on Texas business.

The Board receives a disproportionately high number of complaints against license holders performing the design or evaluation of residential foundations. Since these complaints frequently appear to be a result of poor communications or procedures, the Board established the Residential Foundation Committee (RFC) to pinpoint some of the most common problems and offer a summary of concerns and/or recommendations for the Board's consideration. The RFC and a volunteer support team met in the fall of 1997 and spring of 1998, resulting in the issuance of two reports to the Board's General Issues Committee for staff use in drafting this policy statement. Although the RFC's reports are not a part of this policy, they provide an interesting and quite valuable commentary on various aspects of engineering related to residential foundations. Single copies of the RFC's reports are available by request or may be copied from the Board's home page at http://www.main.org/peboard

The purpose of this policy statement is twofold:

A. Provide recommendations to various non-engineering entities on how to minimize the probability that residential foundation problems, currently encountered by homeowners, will occur.
B. Provide practicing licensed professional engineers with guidance in the preparation of designs and evaluations of residential foundations to minimize the probability that problems, currently encountered by homeowners, will occur.

While the Board may use this policy statement as a tool to evaluate specific complaints, this statement is not intended to replace professional engineering judgment. This statement is intended to emphasize the professional judgment requirements of Board Rules 22 TAC 131.151-155, not to replace or modify them in any way. Under no circumstances should a professional engineer use this statement as a "checklist" of activities needed to adequately

perform an engineering assignment related to residential foundations. In its evaluations of complaints, the Board has consistently been most concerned that the intent of the Board rules of conduct and ethics are followed and that the public and client interests are well served. This statement is designed to underscore that concern.

II. Recommendations

While proper professional engineering practice on individual projects is integral to the success of the project, public policy alterations should be evaluated by the local government entities for probable positive impacts on the property interests of tax-paying homeowners.

The Board makes the following recommendations for consideration by the appropriate entities:

A. Where not already required by existing code, building code enforcement entities such as cities or special districts should require that a licensed professional engineer prepare the designs and directly supervise the construction of residential foundations in situations where soil conditions warrant the involvement of a professional engineer. The public entity should be concerned that warranting conditions may exist:

1. Where the weighted BRAB* equivalent plasticity index of the soil exceeds 20; or
2. Where the site settlement potential exceeds approximately one inch under expected loads; or
3. Where the structure will be supported by fill material; or
4. Where known geological hazards exist.

*Building Research Advisory Board Report #33

B. Warranting conditions should be established in one of two ways. First, licensed professional engineers can establish warranting conditions on a site-specific basis. Second, in areas where general soil conditions are sufficiently well known, licensed professional engineers familiar with local conditions can be requested to aid public entities in the establishment of geographic boundaries where warranting conditions exist.

C. Purchasers of forensic foundation evaluations from licensed professional engineers should base their purchase request on one of three levels of evaluation described in section IV of this statement and understand the scope and limitations associated with that level. The requested level of evaluation to be purchased for the foundation should match the level of analysis of any other evaluations to which it may be compared if a direct comparison is desired. If a particular purpose is intended for the evaluation (such as the development of a repair plan or a forensic report), the engineer must establish the minimum level of evaluation required to adequately accomplish that purpose.

III. Practice Guidance for Licensed Engineers: Design and Repair

Professional engineers designing residential foundations or designing repairs for residential foundations will meet the requirements of all of the applicable Board rules of professional conduct and ethics in their practice. Special emphasis is placed upon:

A. Board Rule 22 TAC 131.151(a) - Engineers have an obligation to protect the property interests of the future homeowner, the builder, the lender and all other parties involved. Inherent in this rule is the notion that an engineer is to provide an optimized, cost-effective design.

B. Board Rule 22 TAC 131.151(b) - Engineers must perform their design in a manner which can be favorably measured against generally accepted standards or procedures. A design or repair plan should include all information needed to delineate its scope, intended use, limitations, client contract requirements or other factors that can impact its proper implementation. If called upon to evaluate a complaint under this rule, the Board will assess engineers' work against design procedures such as the Post-Tensioning Institute's design guideline, the Building Research Advisory Board Report #33, or other similar procedures. Engineers' work will be expected to address significant design issues that may include (but may not be limited to):

1. collection of sufficient geotechnical data;
2. selection of reasonable sample locations and testing activities for geotechnical data;
3. completion of a site characterization activity, denoting key feature such as the presence of water or fill material;
4. inclusion of all needed specification documentation for adequate construction of the foundation;
5. inclusion of a plan for supervising or inspecting the foundation construction; and
6. documentation of all engineering functions in a suitable manner for clients, code officials, etc.

C. Board Rule 22 TAC 131.166 - Engineers must only seal work that they have personally performed or has been performed under their direct supervision. Direct supervision as defined under 22 TAC 131.18 requires the engineer to provide some acceptable combination of exertion of control over the work, regular personal presence, reasonable geographic proximity to the work being performed, and an acceptable employment relationship with the person(s) being supervised. If called upon to evaluate a complaint under this rule, the Board will evaluate all work attributed to an engineer (including post-tension designs, pier layouts, repair details, etc.) for conformity to these direct supervision requirements.

D. Engineers in responsible charge of this type of work must be competent to perform it adequately. Competence is established through education, training or experience in appropriate areas of endeavor; these areas might include residential foundation design, structural engineering, soils and geotechnical engineering, materials engineering and general civil engineering.

IV. Practice Guidance for Licensed Engineers: Evaluations of Existing Foundations

A. When evaluating an existing residential foundation, engineers will be expected to report their findings in a manner that clearly identifies:

1. the purpose of the evaluation;
2. the level of evaluation at which the work was performed; and
3. limitations regarding the conclusions that are drawn given the level of evaluation used.

All evaluations, regardless of the level at which they are performed must be of professional quality as evidenced by sufficient and appropriate data, careful analyses, and disciplined and unbiased judgment when drawing conclusions and stating opinions. In accordance with Board Rule 22 TAC 131.152(b) engineers must communicate using clear and concise language that can be readily understood by their client or other expected audiences.

B. In certain cases, the level of evaluation is established by the client. However, in most cases involving the potential for repair of a condition, the engineer will recommend an appropriate level of evaluation for the situation. Under Board Rule 22 TAC 131.155(a), the engineer is expected to recommend and perform the lowest level of evaluation needed for adequate analysis of the situation. For the purpose of aiding the client in determining the type of evaluation performed (or desired), the Board recommends the use of the following three levels of evaluation designations:

1. Level A - This level of evaluation will be clearly identified as a report of first impression conclusions and/or recommendations and will not imply any higher level of evaluation has been performed. Level A evaluations will typically:

a. define the scope, expectations, exclusions, and other available options;
b. interview the home owner and/or client if possible;
c. document visual observations personally made by the engineer during a physical walk-through;
d. describe the analysis process used to arrive at any performance conclusion; and
e. provide a report containing one or more of the following: observations, opinions, performance conclusions, and recommendations based on the engineer's first impressions of the condition of the foundation.

2. Level B - This level builds upon the elements found in a Level A evaluation. In addition to the items included in Level A, a Level B evaluation will typically:

a. request and review available documents such as geotechnical reports, construction drawings, field reports, prior additions to the foundation and frame structure, etc.;
b. determine relative foundation elevations to assess levelness at the time of evaluation and to establish a datum;
c. if appropriate, perform non-invasive plumbing tests, recognizing that additional invasive testing is also available;
d. document the analysis process, data and observations;
e. provide conclusions and/or recommendations; and
f. document the process with references to pertinent data, research, literature and the engineer's relevant experience.

3. Level C - This level builds upon the elements found in the Level B evaluation. In addition to the items included in Levels A and B, a Level C evaluation will typically:

a. conduct non-invasive and invasive plumbing tests as required by the engineer;
b. conduct site specific geotechnical investigations as required by the engineer;
c. conduct materials tests as required by the engineer to reach a conclusion;
d. obtain other data and perform analyses as required by the engineer;
e. document the analysis processes, data and observations; and
f. provide conclusions and/or recommendations.

C. Engineers performing evaluations of residential foundations should be especially aware of their obligations under Board Rules 22 TAC 131.153(c), 22 TAC 131.151(b), and 22 TAC

131.152(b) as they report their findings. They should substantiate all assumptions, conclusions, and recommendations using appropriate references. Terms such as "failure", "distress", "damage", etc. must be clearly defined. When an evaluation is to be used in comparison with another report, the engineers should make every effort to provide a correlation to the definition used in the previous report in addition to any other definitions used in their own report. Engineers must draw any needed distinctions between "failures" discussed from a structural aspect and "failures" discussed from a performance aspect.

D. As previously noted in section III (D), engineers in responsible charge of this type of work must be competent to perform it adequately. Competence is established through education, training or experience in appropriate areas of endeavor; these areas might include specific residential foundation design, structural engineering, soils and geotechnical engineering, materials engineering and general civil engineering.

V. Related Advisories & Updates

There are no related advisories at this time. Updates may be made periodically by the board. Date of this advisory: 09/11/98.

Questions regarding this advisory may be sent to:

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RECOMMENDED HOMEOWNER FOUNDATION MAINTENANCE PROGRAM FOR RESIDENTIAL PROJECTS IN TEXAS BY DAVID A. EASTWOOD, P.E.

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 Texas. 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 Texas 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 State of Texas area. Highly expansive soils exist in parts of Houston, Dallas, San Antonio, Austin and other places in Texas. In general, the concentrations of expansive soils are along the east side of the state.

Sandy soils with potential for severe perched water table problems as a result of poor drainage are present in 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.

Weathered rock formations generally exist on the west side of the state in areas such as San Antonio, Austin, Midland, Odessa, parts of Dallas, etc. These formations are generally non-expansive; however, expansive shale and/or weathered limestone are present in some areas. Variable geology exists in the areas such as Austin where several types of soils/rocks can be found within a site.

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 from 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, nonswelling 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 create 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 that 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.

Landscaping

General. 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 the form of brick cracking, window/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 the form of foundation settlement, brick cracking, etc.

Sprinkler Systems. 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 36-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 the 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 which is where 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 arboriculturist.

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 a normal weather pattern may prevent further damage from occurring. Permission from the local authority is required before pruning the roots of a tree with a 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 around or under the trench. However, the barrier should at least increase the time it takes for the roots to grow back. Root barriers serve as bio barrier root control system and appear to perform satisfactorily. The design of the root barrier system should be developed in construction with the geotechnical engineer to assume long-term performance of the structure.

Erosion Protection

In the event that a residence is constructed on top of a slope, near a ravine, bayou, etc. The homeowner must make sure that (a) the foundation is properly designed for the specific site conditions, and (b) proper erosion control systems are in place to assure the stability of the slopes and corresponding foundation stability. A proper erosion control system may consist of the use of grass cover, rip-rap, concrete lining, etc. Other types of erosion protection system can also be used.

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.☺

GEOTECHNICAL GUIDELINES

FOR

DESIGN, CONSTRUCTION, MATERIALS AND

MAINTENANCE OF

RESIDENTIAL PROJECTS IN THE HOUSTON AREA

By

David A. Eastwood, P.E.
Geotech Engineering and Testing, Inc.

Presented at the

Soils-Structure Interaction Seminar

November 1998

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ILLUSTRATIONS

INTRODUCTION

The variable subsoil conditions in the Gulf Coast area has resulted in very special design requirements for residential and light commercial foundations. The subsurface conditions should be carefully considered when a subdivision or a residence is to be built. Proper planning from the stand point of environmental conditions, subsidence, faulting, soil conditions, design, construction, materials, quality control and maintenance program should be considered prior to any development.

The purpose of this document is to recommend the scope of geotechnical work to develop soils and foundation data for a proper and most economical design and construction of foundations in the Houston area. It is our opinion that portions of these studies should be performed prior to developing the subdivision or buying the lots in order to minimize potential future soils and foundation problems. These problems may arise from the presence of hazardous waste, faulting, poorly compacted fill, soft soil conditions, expansive soils, perched water table, presence of sand and silts, tree roots, etc. This guideline is divided into six segments, including Pre-Development Studies, design, construction, materials, quality control, maintenance program and foundation stabilization. Our recommendations are presented from a geotechnical stand point only and should be complemented by a structural engineer.

PRE-DEVELOPMENT STUDIES

<u>Environmental Site Reconnaissance Study</u>

Environmental site assessment studies are recommended on the tracts of land for subdivision or commercial developments. A study like this is generally not required for a single lot in an established subdivision or an in-fill lot in the city. This type of study is used to evaluate the potential risk of environmental contamination that is on or used to be on a project site prior to development. The study is divided into phases, Phases I through III.

The scope of Phase I includes a preliminary site reconnaissance, including: (a) document search, (b) site walk through, (c) review of aerial photographs, (d) historical ownership report, (e) regulatory data review and (f) a report of observations and recommendations.

In the event that the results of the Phase I study indicates the potential for the presence of contaminants, a Phase II study is performed. The scope of Phase II study may include: (a) soil and groundwater sampling, (b) chemical testing and analysis, (c) site reconnaissance, (d) contact with state and federal regulatory personnel, and (e) reporting.

A Phase III study involves implementing the recommendations given in the Phase II study; including remediation and monitoring.

Subsidence

Potential subsidence problems should be considered when developing subdivisions in the coastal areas, such as Clear Lake, Seabrook, Baytown, etc. Also, other parts of Houston, subject to groundwater removal are also subject to subsidence. This type of study is generally not needed for a single lot in an established subdivision or an in fill lot in the city.

Subsidence is the sinking of the land surface caused by the withdrawal of groundwater. The land elevation lost to subsidence is generally permanent and irreversible. In the Harris-Galveston region of Texas, subsidence poses the greatest threat in the coastal areas susceptible to flooding due to high tides, heavy rainfall and hurricane storm surge. Because of low elevation, any additional subsidence in the coastal areas results in a significant increase in potential tidal flooding or permanent inundation.

The rate of land subsidence in Harris County has been reduced significantly due to changes in water development from the surface water instead of groundwater.

A review of recent subsidence data available from Harris County Subsidence District indicates that the subsidence in areas such as Pasadena, Southwest Houston, etc. have slowed down significantly. However, the subsidence rate in the Addick Area (West, Northwest Houston) is about one-inch per year.

Geologic Faulting

Many faults have been observed within the Gulf Coast Region of Texas. In general, faults are caused by groundwater and oil removal from the underlying surface. Faults originate several thousand feet below the ground surface and can often cause displacement of the ground surface, causing broken pavement and damage to residential and commercial structures.

Faults are studied in several phases. A Phase I fault study will include the first step in identification of faulting. The scope of a Phase I investigation includes the following elements:

1. Literature Review. This includes a search for, and study of, published data on surface faults in the area of the site.

2. Remote Sensing Study. Aerial photographs, infra-red imagery, where available, should be studied.

3. Field Reconnaissance. This includes a visit to the study area and vicinity by a qualified engineer to examine the area for physical evidence of a possible fault or faults. Physical evidence includes, but is not limited to, (a) natural topographic scarps, (b) soil layer displacements that may be recognized in ditches, creek banks and trenches, (c) breaks in pavements, (d) distress in existing buildings, and (e) vertical offsets in fences.

Once a residence is built on an active fault, the foundation for the residence will be subject to a continual movement and subsequent distress. Foundation stabilization of structures built on active faults can be difficult, but possible. A study of geologic faulting is recommended prior to development of any subdivision in the Gulf-coast area.

GENERAL SOILS AND GROUNDWATER

Geology

The Houston area is located on the Gulf of Mexico Coastal Plain, which is underlain largely by overconsolidated clays, clay shales and poorly cemented sands to a depth of several miles. Nearly all soil of the area consists of clay, associated with moderate amounts of sand. Some of the formations in the Houston area consist of Beaumont, Lissie, and Bentley.

The Beaumont formation has significant amounts of expansive clays, resulting in shrink/swell potential. Desiccation of this formation also produces a network of fissures and slickensides in the clay that is potential plains of weakness. The Beaumont formation generally occurs in South, Southwest, East, and Central Houston. The Lissie and Bentley formations generally occur in North and part of West Houston. These formations consist of generally sands and sandy clays. These soils are generally low to moderate in plasticity with low to moderate shrink/swell potential.

General Soils Conditions

Variable soil conditions occur in the Houston area. These soils are different in texture, plasticity, compressibility, and strength. It is very important that foundations for residential and light commercial structures be designed for subsoil conditions that exists at the specific lot in order to minimize potential foundation and structural distress. Details of general subsoil conditions at various parts of the Houston area are described below. These descriptions are very general. Significant variations from these descriptions can occur. The General soil conditions are as follows:

Location	Soil Conditions
Northwest and Northeast Houston, including Kingwood, The Woodlands, Cypresswood, Copperfield, Atascocita area, Fairfield, Worthom, and Oaks of Devonshire	Generally sandy surficial soils occur in these areas. The sands are generally loose and are underlain by relatively impermeable clays and sandy clays. This condition promotes perched water table formation which results in the loss of bearing capacity of the shallow foundations such as a conventionally-reinforced slab or post-tensioned slabs. This condition also may cause subsequent foundation settlement and distress.

South, Southwest, Southeast, and part of West Houston including, Kirbywoods, parts of the South Shore Harbour, Kelliwood Gardens, Clear Lake area, New territory, Greatwood, First Colony, Brightwater, Vicksburg, Pecan Grove, Woods Edge, Cinco Ranch, and Lake Olympia.	Generally highly plastic clays, and sandy clays are present in these areas. These clays can experience significant shrink and swell movements. The foundations must be designed for this condition. Parts of Cinco Ranch has a surficial layer of sands, underlain by expansive clays. The foundations these soils should be designed, assuming a perched water table condition.
Central Houston, including Bellaire, Tanglewood, West University, River Oaks.	Highly expansive clays, drilled footings are the preferred foundations system. Soft soils are observed in some lots. The soils in the River Oaks area are generally moderately expansive.
Memorial area, Heights, spring Branch, Hunter's Creek, Bunker Hill, Piney Point, Hedwig Village.	Moderately expansive sandy clays, clays, and sands. Special foundations must be used for structures near ravines. Look for faults.
Other Locations:	
(a) Weston Lakes, Oyster Creek.	Very sandy soils in some areas, variable soil conditions. Slab-at-Grade is a typical foundation; sometimes piers. Shallow water table at Oyster Creek. Highly expansive soils in parts of Weston Lakes.
(b) Sugar Mill, Sugar Creek, Plantation Colony, Quail Valley, Sweetwater.	Highly expansive clays on top of loose silts and sands. Variable soil conditions. A floating slab is a typical foundation. Piers can also be used at some locations. Soft in some lots. Shallow water table.

Water Level Measurements

The groundwater levels in the Gulf Coast area vary significantly. The groundwater depth in the Houston area generally ranges from 8- to 30-feet. Fluctuations in groundwater level generally occurs as a function of seasonal rainfall variation, temperature, groundwater withdrawal, and construction activities that may alter the surface and drainage characteristics of the site.

The groundwater measurements are usually evaluated by the use of a tape measure and weight at the end of the tape at the completion of drilling and sampling.

An accurate evaluation of the hydrostatic water table in the relatively impermeable clays and low permeability silt/sands requires long term observation of monitoring wells and/or piezometers. It should be noted that it is not possible to accurately predict the pressure and/or level of groundwater that might occur based upon short-term site exploration. The installation of piezometers/monitor wells is beyond the scope of a typical residential geotechnical reports. We recommend that the groundwater level be verified just before construction if any excavations such as construction of drilled footings/underground utilities, etc. are planned.

The geotechnical engineer must be immediately notified if a noticeable change in groundwater occurs from the one mentioned in the same report. The geotechnical engineer should then evaluate the affect of any groundwater changes on the design and construction of the facilities.

Some of the groundwater problem areas in Houston include Southside Place, parts of Sugar land, etc. One should not confuse the perched water table with the groundwater table. A perched water table occurs when bad drainage exists in areas with a sand or silt layer, about two- to four-foot thick, underlain by impermeable clays and sandy clays. During the wet season, water can pond on the clays and create a perched water table. The surficial sands/silts become extremely soft, wet and may lose their load carrying capacity.

DESIGN

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 is 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 following are the foundation types typically used in the area with increasing levels of risk and decreasing levels of cost:

FOUNDATION TYPE	REMARKS
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 areas 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 with 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 Conventionally-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 dowelled into the grade beams. The structural loads can also be supported on spread footings if expansive soils are not present.
Floating Slab Foundation (Conventionally-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 areas where trees have been removed prior to construction and where expansive clays exists, these foundations must be significantly stiffened to minimize the potential differential movements as a result of subsoil heave due to tree removal.

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, house geometry, and soil types. For example, in some cases, a floating slab foundation may perform better than a drilled footing type foundation.

Foundation Types

Residential structures in the Houston area are supported on drilled footings, post-tensioned slabs, or conventionally reinforced slabs. In general, properly designed post-tensioned slabs or conventionally-reinforced slabs perform satisfactory on most subsoils. Drilled footings may provide a superior foundation system when large slabs, significant offsets or differential loading occurs on the foundations.

The selection of foundation is a function of economics and the level of the risk that the client wants to take. For example, a structural slab foundation is not used for a track home that costs about $100,000. This type of foundation is used for houses that cost usually much more expensive. In general, floating slab type foundations are used with houses with price ranges of less than $200,000 or when subsoil conditions dictates to use this type of foundation.

Geotechnical Foundation Design Criteria

Foundations for a residential structure should satisfy two independent design criteria. First, the maximum design pressure exerted at the foundation base should not exceed the allowable net bearing pressure based on an adequate factor of safety with respect to soil shear strength. Secondly, the magnitude of total and differential settlements (and shrink and swell) under sustained loads must be such that the structure is not damaged or its intended use impaired.

It should be noted that properly designed and constructed foundation may still experience distress from improperly prepared bearing soils and/or expansive soils which will undergo volume change when correct drainage is not established or an incorrectly controlled water source becomes available.

The design of foundations should be performed by an experienced structural engineer using a soils report from an experienced soils engineer. The structural engineer must use a lot/site specific soils report for the foundation design. The structural engineer should not use general subdivision soils reports written for underground utilities and paving for the slab design. Furthermore, he should not design slabs with disclaimers, requiring future soils reports to verify his design. The designers or architects should not provide clients with foundation design drawings with generic foundations details. All of the foundation drawings should be site and structure specific and sealed by a professional structural engineer.

Recommended Scope of Geotechnical Studies

Soil testing must be performed on residential lots before a foundation design can be developed. The recommended number of borings should be determined by a geotechnical engineer. The number of borings and the depths are a function of the size of the structure, foundation loading, site features, and soil conditions. As a general rule, a minimum of one boring for every five lots should be performed for subdivision lots. This boring program assumes that a conventionally-reinforced slab or a post-tensioned slab type foundation is going to be used. Furthermore, many lots will be tested at the same time so that a general soils stratigraphy can be developed for the entire subdivision. In the event that a drilled footing foundation is to be used, a minimum of one boring per lot is recommended. In the case of variable subsoil conditions, two or more borings per lot should be performed. A minimum of two borings is recommended for custom homes or a single in-fill lot. A minimum boring depth of 15-feet is recommended for the design of post-tensioned or conventially-reinforced slabs. The boring depths for the design of drilled footing foundations should be at least 15-feet deep. In the event that the lot is wooded and expansive soils are suspected, the boring depth (if drilled footings are to be used) should be increased to 20-ft. On the wooded lots, when the presence of expansive soils are suspected the borings should be drilled near the trees, if possible. Root fibers should be obtained to estimate the active zone depth. The active zone depth is defined as the depth within which seasonal changes in moisture content/soil suction can occur. In general, the depth of active zone is about two-feet below the lowest root fiber depth.

The borings for the residential lots should be performed after the streets are cut and fill soils have been placed and compacted on the lots. This will enable the geotechnical engineer to identify the fill soils that have been placed on the lots. All fill soils should have been tested for compaction during the placement on the lots. A minimum of one density test for every 2500 square feet per lift must be performed once a subdivision is being developed. Fill soils may consist of clays, silty clays, and sandy clays. Sands and silts should not be used as fill materials. Typical structural fill in the Houston area consists of silty clays and sandy clays (not sands) with liquid limits less than 40 and plasticity index between 10 and 20. The fill soils should be placed in lifts not exceeding eight-inches and compacted to 95 percent of the maximum dry density (ASTM D698-91). On-site soils with the exception of sands can also be used as structural fill under floating slab foundations. A floating slab foundation is defined as a conventionally-reinforced slab and a post-tensioned slab.

In the case of a subdivision development, the developer should perform only the borings for the streets and underground utilities. The borings for the lots should wait until all fill soils from street and underground utility excavations are placed and compacted on the lots. In general, the geotechnical testing of the soils for the lots should be the builders responsibility. We recommend that all of the foundations in the subdivision be engineered by a registered professional engineer specializing in residential foundation design.

In the areas where no fill will be placed on the lots prior to site development, the borings on the lots can be performed at the same time as the time as the borings for streets. The soils data from the street and underground borings should never be used for the slab design. This is due to potential in variability in the soil conditions, including soils stratigraphy, compressibility, strength, and swell potential.

Soil borings must be performed prior to foundations underpinning for distressed structures. This is to evaluate the subsoil properties below the bottom of the drilled footings. The depth of drilled footings for foundation underpinning should be determined by a geotechnical engineer. Unfortunately, this is not always followed, and many "so called" foundation repair jobs are performed incorrectly, causing significant financial loss for the client.

In the event of building additions, a minimum of one boring is recommended on residential additions of less than 1,000 square feet. A minimum of two borings is recommended for additions greater than 1,000 square feet.

In general, a scope of typical geotechnical exploration does not include the evaluation of fill compaction. These studies should have been performed at the time of fill placement.

Foundation Design Considerations

In the areas where highly expansive soils are present, the drilled footings should be founded in a strong soil stratum below the zero movement line. This depth is defined as the depth below which no upward movements occur. It is possible to found a drilled footing below the zero movement line and within the active zone depth. The active zone is defined as the zone within which seasonal changes in subsoil moisture can occur. This is shown on Plate 1. Drilled footings in the area with deep active zones, where trees are present, and subsoils are expansive can be as much as 18-feet deep. The depth of drilled footings should also be determined such that the uplift along the pier shafts be resisted by the presence of bells or shaft skin friction below the zero movement line. The depth of the active zone should be verified by a geotechnical exploration. The evaluation of active zone depths and zero movement line should be performed using the techniques provided in the 1996 Post-Tensioning Institute Slab-on-Grade Design Manual. Drilled footings founded at shallower depths may experience uplift due to expansive soils. In the areas where non-expansive soils are present, the footing depth can be as low as eight-feet.

The grade beams for a floating slab foundation should penetrate the clay soils a minimum of 12-inches. The grade beam penetrations for a floating slab foundation into the surficial sands should be at least 18-inches to develop the required bearing capacity. A minimum grade beam width of 12-inches is recommended in sands and silts.

In the event that a floating slab (post-tensioned slab or a conventionally-reinforced slab) is constructed in sands or silts, the geotechnical engineer must specify bearing capacity, assuming saturated subsoil conditions. This results in bearing capacities in the range of 600- to 900 psf in a typical sand or silt soils in the Houston area. Higher bearing capacity values can be used if the sands/silts do not get saturated during the life of the residence. This assumption is generally unrealistic due to the presence of sprinkler systems, negative drainage, and cyclic rainfall in the Houston area.

Design parameters for a post-tensioned slab on expansive clays must carefully evaluated by a geotechnical engineer. It should be noted that the 1996 post-tensioned slab design manual does not directly model the poor drainage, the effect of the trees, and the depth of the active zone. The geotechnical engineer must modify the design parameter presented in the manual to come up with the proper design parameter. It should be noted that it is currently very difficult (to impossible) to design economical floating slab foundations on expansive soils on wooded lots where trees are to be removed prior to slab construction.

Floor Slabs

The floor slabs for foundations supported on drilled footings may consist of (a) structural slabs with crawl space, (b) slab-on-fill or (c) slab-on-grade.

A structural slab should be used when a minimum risk foundation is to be used. This type of floor slabs are generally expensive. A slab-on-fill will be less expensive than a structural slab with crawl space. The fill thickness in areas where expansive soils are present should be about 18-to 48-inches. The higher fill thickness should be used in areas such as Bellaire, Tanglewood, New Territory, etc, where highly expansive clays exists (plasticity indices above 50).

In the event that a structural slab foundation is used, the crawl space area should be properly drained so that any water would drain towards the exterior grade beams. Furthermore, the area should be properly vented.

The floor slabs can be supported at grade on drilled footings if the subsoils are non-expansive. All of the subgrade soils should be prepared in accordance to the soils report site preparation section prior to fill placement.

Void Boxes

Void boxes are historically used under the grade beams to separate the expansive soils from the grade beams. The void boxes collapse once the underlying expansive soils swell up; thereby minimizing uplift loads as a result of expansive soils on the grade beams. This can be an effective feature for reducing potential pressures on grade beams.

In areas of poor drainage, void boxes may act as a pathway for water to travel under a foundation system. This condition may result in an increase in subsoil moisture contents and subsequent swelling of the soils. This may result in uplift loads on the floor slabs, and subsequent distress to the foundation and structural system.

We recommend that the decision on whether or not to use void boxes be made by the owner/builder after both the positive and negative aspects of this issue are evaluated. Based on our and other experts personal experience with void boxes, it is our opinion that they will not provide an effective feature for reducing swell pressure on the grade beams. In general, the validity of void box usage is presently being questioned because of the frequency of observed negative effects which may outweigh its benefits.

Site Drainage

It is recommended that site drainage be well developed. Surface water should be directed away from the foundation soils (use a slope of about 5% within 10-feet of foundation). No ponding of surface water should be allowed near the structure.

Residential Structures Constructed near the Bayous

Many large residential structures are being build near the bayous. Portions of the slopes on the bayous are very steep with slopes steeper than 3(h):1(v). The foundations for residences near the bayous must be provided by the use of deep drilled footings/piling. The geotechnical boring depths should be at least twice the depth of the bayou.

Any foundation which falls within the hazard zone which extends from the toe of the slope, extending backward on a 4(h):1(v) slope to the existing grade should be supported on deep foundations. Foundations outside the hazard zone may be supported on shallow piers. The floor slabs in the hazard zone should consist of a structural slab. The floor slabs outside the hazard zone may consist of slab-on -fill or slab-on-grade. No skin friction should be used for piers within the hazard zone from the surface to the toe of the slope elevation.

We recommend the stability of bayou slopes be evaluated using a slope-stability analyses, using computer solutions. The house should be placed on top of the slope and the stability of the slope for global stability should be evaluated. The slopes should then be flattened and covered with erosion protection to minimize potential sloughing and erosion problems.

CONSTRUCTION

Site Preparation

Our recommendations on site preparation are summarized below:

1. In general, remove all vegetation, tree roots, organic topsoil, existing foundations, paved areas and any undesirable materials from the construction area. Tree trunks under the floor slabs should be removed to a root size of less than 0.5-inches. We recommend that the stripping depth be evaluated at the time of construction by a soil technician.

2. Any on-site fill soils, encountered in the structure and pavement areas during construction, must have records of successful compaction tests signed by a registered professional engineer that confirms the use of the fill and record of construction and earthwork testing. These tests must have been performed on all the lifts for the entire thickness of the fill. In the event that no compaction test results are available, the fill soils must be removed, processed and recompacted in accordance with our site preparation recommendations. Excavation should extend at least two-feet beyond the structure and pavement area. Alternatively, the existing fill soils should be tested comprehensively to evaluate the degree of compaction in the fill soils.

3. The subgrade areas should then be proofrolled with a loaded dump truck, scraper, or similar pneumatic-tired equipment. The proofrolling serves to compact surficial soils and to detect any soft or loose zones. Any soils deflecting excessively under moving loads should be undercut to firm soils and recompacted. The proofrolling operations should be observed by an experienced geotechnician.

4. Scarify the subgrade, add moisture, or dry if necessary, and recompact to 95% of the maximum dry density as determined by ASTM D 698-91 (Standard Proctor). The moisture content at the time of compaction of subgrade soils should be within -1 to +3% of the proctor optimum value. We recommend that the degree of compaction and moisture in the subgrade soils be verified by field density tests at the time of construction. We recommend a minimum of four field density tests per lift or one every 2500 square feet of floor slab areas, whichever is greater.

5. Structural fill beneath the building area may consist of off-site inorganic silty clays or sandy clays with a liquid limit of less than 40 and a plasticity index between 10 and 20. In the event that a floating slab foundation system is used, on-site soils (with the exception of sands or silts), free of organics, can be used as structural fill. Other types of structural fill available locally, and acceptable to the geotechnical engineer, can also be used.

   These soils should be placed in loose lifts not exceeding eight-inches in thickness and compacted to 95 percent of the maximum dry density determined by ASTM D 698-91 (Standard Proctor). The moisture content of the fill at the time of compaction should be within $\pm 2\%$ of the optimum value. We recommend that the degree of compaction and moisture in the fill soils be verified by field density tests at the time of construction. We recommend that the frequency of density testing be as stated in Item 4.

6. The backfill soils in the trench/underground utility areas should consist of select structural fill, compacted as described in Item 4. In the event of compaction difficulties, the trenches should be backfilled with cement-stabilized sand or other materials approved by the Geotechnical Engineer. Due to high permeability of sands and potential surface water intrusion, bank sands should not be used as backfill material in the trench/underground utility areas.

7. In cut areas, the soils should be excavated to grade and the surface soils proofrolled and scarified to a minimum depth of six-inches and recompacted to the previously mentioned density and moisture content.

8. The subgrade and fill moisture content and density must be maintained until paving or floor slabs are completed. We recommend that these parameters be verified by field moisture and density tests at the time of construction.

9. In the areas where expansive soils are present, rough grade the site with structural fill soils to insure positive drainage. Due to their high permeability of sands, sands should not be used for site grading where expansive soils are present.

10. We recommend that the site and soil conditions used in the structural design of the foundation be verified by the engineer's site visit after all of the earthwork and site preparation has been completed and prior to the concrete placement.

Other Construction Considerations

1. Grade beam excavations should be free of all loose materials. The bottom of the excavations should be dry and hard.

2. Surfiçial subgrade soils in the floor slab areas should be compacted to a minimum of 95% of Standard Proctor Density (ASTM D 698-91). This should be confirmed by conducting a minimum of four field density tests per slab, per lift.

3. Minimum concrete strength should be 3,000 psi with a maximum slump of 5-inch. Concrete workability can be improved by adding air to the concrete mix and the use of a concrete vibrator. The concrete slump and strength should be verified by slump tests and concrete cylinders.

4. The Visqueen, placed under the floor slabs, should be properly stretched to maximize soil-slab interaction.

5. In the areas where expansive soils are present, the backfill soils for the underground utilities under the floor slabs should consist of select fill and not sands or silts. The cohesionless backfill can act as a pathway for surface water to get under the foundation and resulting in subsoil swelling. In the event that a floating slab is used, on-site soils (not sands or silts), free of organics, can be used as structural fill.

6. Tree stumps should not be left under the slabs. This may result in future settlement and termite infestation.

MATERIALS

The use of proper materials is crucial to the performance of a foundation system. Some of the relevant material issues is as follows:

- o Inadequate concrete strength.

- o Reinforcement, steel grade.

- o Improperly manufactured post-tensioned materials.
- o The geotechnical technician must check the earthwork testing, concrete pier, installation, and concrete placement.

QUALITY CONTROL

General

Construction monitoring and quality control tests should be planned to verify materials and placement in accordance with the project design documents and specifications. Earthwork observations on the house pad, pad thickness measurements, drilled footing installation monitoring, and concrete placement monitoring should be performed. Details of each of these items is described in the following paragraphs.

Earthwork Observations

The subgrade and fill soils under the floor slabs should be compacted to about 95 percent of maximum dry density (ASTM D 698-91). Furthermore, the fill soils should be non-expansive. Atterberg limit tests should be performed on the fill soils, obtained from the borrow pit, to evaluate the suitability of these soils for use as structural fill and their shrink/swell potential. Expansive soils, of course, should not be used as structural fill. In the event that a floating slab foundation is used, on-site soils with the exception of sands/silts can be used as structural fill.

Field density tests should be conducted on the subgrade soils and any borrow fill materials in the floor slab and pavement areas. In the areas where expansive soils are present, about 18- to 36-inches of structural fill is placed under the floor slab areas. Laboratory proctor tests will also be performed on the on-site soils as well as off-site borrow fill materials to evaluate the moisture-density relationship of these soils.

Fill Thickness Verification

Fill soils may have to be placed on the lots to raise the lot or to provide a buffer zone in between the on-site expansive soils and the floor slabs. We recommend that the required thickness of the fill be verified after the completion of the building pad. This task can be accomplished by drilling two borings to a depth of two-feet in the building pad area, examining and testing the soils to verify the fill thickness.

Drilled Footing Observations

In the event that the structure is supported by drilled footings, we recommend that the installation of the footings be observed by a geotechnical technician.

The technician will conduct hand penetrometer tests on the soil cuttings to estimate the bearing capacity of the soil at each footing location. He will make changes to the foundation depth and dimensions if obstacles or soft soils are encountered. Therefore, minimizing costly construction delays. In addition, the technician must verify the bell size by a bell measurement tool. One set of concrete cylinders (four cylinders) will be made for each 50 yards of pour. Two cylinders will be broken at seven days, and two cylinders at 28 days.

Concrete Placement Monitoring

The concrete sampling and testing in the floor slab and placement areas will be conducted in accordance with ASTM standards. A technician will monitor batching and placing of the concrete. At least four concrete cylinders should be made for each 50 yards floor slab pour. Two concrete cylinders are tested at seven days and two cylinders at 28 days.

HOMEOWNER MAINTENANCE PROGRAM

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.

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 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 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 on-site soils (not sand or silts), free of organics, can be used as structural fill.

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, nonswelling 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 nine 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. Furthermore, The crawl space area should be properly vented.

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.

Landscaping

General. 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.

Sprinkler Systems. 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 7-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 36-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, subsoil swelling can occur for several years. Studies have shown that for certain types of trees this process can last as much as 20 years in the areas where highly expansive clays are present. In this case the foundation for the house 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. 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 equal/stabilize to the surrounding subsoil moisture 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 general, in the areas where non-expansive soils are present, no foundation heave 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 arboriculturist.

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.

FOUNDATION STABILIZATION

General

Several methods of foundation stabilization are presented here. These recommendations include foundation underpinning, using drilled footings or pressed piling, moisture barriers, moisture stabilization, and chemical stabilization. Some of these methods are being used in the Houston area. A description of each method is summarized in the following sections of this document.

Foundation Underpinning

Foundation Underpinning, using drilled footings or pressed piling has been used in the Houston area for a number of years. The construction of a drilled footing consists of drilling a shaft, about 12-inches in diameter (or larger) constructed underneath the grade beam. The shaft is generally extended to depths ranging from 8 to 12-feet below existing grade. The bottom of the shaft is then reamed with an underreaming tool. The hole is then backfilled with steel, concrete, and the grade beams are jacked to a level position and shimmied to level the foundation system.

In a case of pressed piling, precast concrete piers are driven into the soils. These pier attain there bearing capacity based on the end bearing and the skin friction. In general, the precast concrete units are about 12-inches in height, six-inches in diameter and jacked into the soil. It is important the precast pier foundations are driven below the zero movement line to resist the uplift loads as a result of underlying expansive soils. Some of these jacked piles may consist of perma-piles, ultra piles, cable lock piles, etc.

The use of drilled footings/pressed piles should be determined by a geotechnical/structural engineer. Each one of these foundation systems have their pluses and minuses. Neither of these foundations can resist upward movement of the slabs. In general, they only limit the downward movement of the slabs. The pressed piles may not resist uplift loads as a result of skin friction of expansive soil if they are not connected with a cable or reinforcement. Therefore, if the units are not properly connected, they will not provide any tensile load transfer. The construction of each method should be monitored by an experienced geotechnical/structural engineer.

Helical piles which consist of Helical auger drilled into the soils provide a good method for underpinning, especially in the areas where sand, silts, shallow water table or caving clays are present. The helical piles are drilled into the soils until the desired resistance to resist the compressive loads are achieved. The augers are then fitted with a bracket and jacked against the grade beams to lift and to level the foundations.

Interior foundations may be required to level the interior of the residence. This can be accomplished by installing interior piers, tunneling under foundations and using pressed piling, or the use of polyurethane materials injected at strategic locations under the slab. The use of tunneling to install interior piers may introduce additional problems, such as inadequate compaction of backfill soils under the slab. However, the author has never encountered such a problem with pressed piling.

Partial underpinning is used in the areas where maximum distress is occurring under a slab. The use of full underpinning which includes placement of piers/pressed piling underneath all foundations is not necessarily a better method of stabilizing foundations. Many foundations are performing satisfactorily with partial underpinning. In the event that foundation underpinning is used, the home owners should put into place a foundation maintenance program to prevent additional foundation distress as a result of changes in subsoil moisture content.

Moisture Stabilization

Moisture Stabilization can be an effective method of stabilizing subsoil shrink swell movements in the ares where expansive soils are present. This method of stabilization is not effective in the areas where sands are present such as north of Harris County in areas such as Kingwood, Fairfield and The Woodlands. This method could be effective in the areas of highly expansive soils such as Tanglewood, Bellaire, West University, River Oaks, South Houston, and Southwest Houston. The method uses a porous pipe that is placed around the perimeter of the foundation and is connected to a water pressure system. A timer turns the water on and off depending on the subsoil moisture conditions, the moisture conditions around the perimeter of the house are monitored by moisture sensors. In general, the purpose of the system is to stabilize the moisture content around the slab to a uniform condition; therefore, minimizing the extremes of shrink and swelling problems. As it was mentioned earlier, the use of this method can result in major problems in the areas where sandy soils are present.

Moisture Barriers

Moisture barriers can be used to isolate subsoil moisture variations in the areas where expansive soils are present. This can be as a result of surface water, groundwater, and tree root systems. In general, a moisture barrier may consist of an impermeable filter fabric, placed just outside the grade beams to depths ranging from five- to seven-feet. The moisture barriers can be horizontal or vertical. A horizontal moisture may consist of a sidewalk attached the exterior grade beams. The waterproofing between the moisture barrier and the exterior grade beams are very important. The connection should be completely sealed so that surface water can not penetrate under the horizontal moisture barrier. In general, it may take several years for the moisture barriers to effectively stabilize the moisture content underneath the floor slabs. A minimum vertical moisture barrier depth of five-feet is recommended.

Chemical Stabilization

This method of foundation stabilization has not been used in the Houston area routinely; however, it has been used for many projects in Dallas and San Antonio areas. The purpose of chemical stabilization is to chemically alter the properties of expansive soils; thus, making it non-expansive. In a chemical stabilization technique, the chemicals which may consist of lime or other chemicals are injected into the soil to a depth of about 7-feet around the perimeter of the structure. The chemical stabilization may (a) chemically alter the soil properties, and (b) provide a moisture barrier around the foundation. In general, this type of stabilization is effective when the chemicals are in intimately mixed with the soil. This can occur in soils that exhibit fissured cracks and secondary structures. This method of stabilization is not effective in the areas where soils do not experience significant cracking.

Regardless of what method of foundation stabilization is used, the homeowner maintenance with respect to proper drainage and landscaping is extremely important for success of any method.

RECOMMENDED QUALIFICATIONS FOR THE GEOTECHNICAL ENGINEER

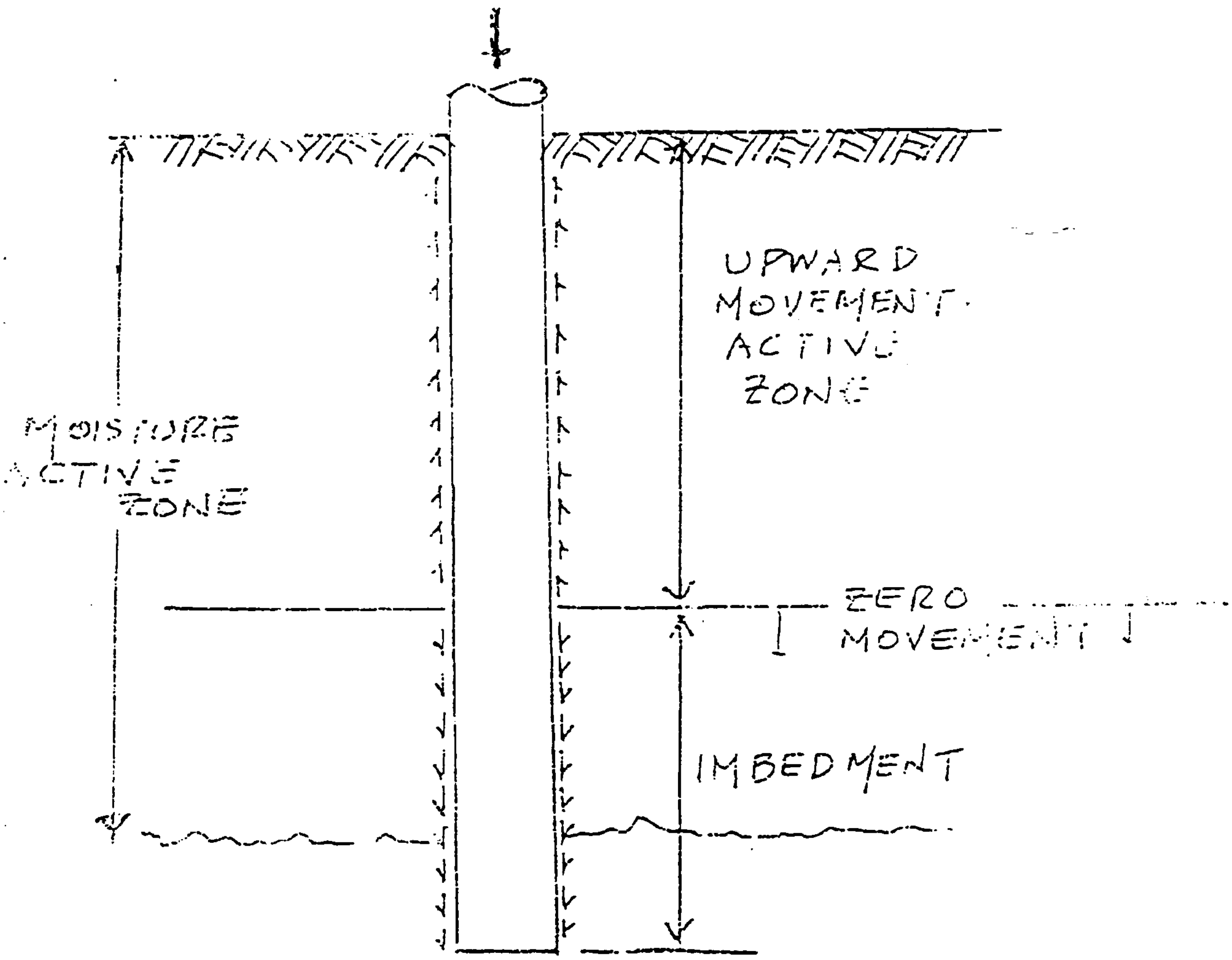
We recommend that the geotechnical engineer should have the following qualifications:

- Engineer must have several years experience in the same geographical area where the work will take place (i.e. proven designs in a given area).
- A Professional Engineer (P.E.) designation with a geotechnical engineering background should be required. A civil engineer with a master's degree or higher is preferred. The civil engineer must have a geotechnical engineering specialty.
- The geotechnical engineering firm must have a A2LA Laboratory certification in geotechnical engineering.
- The firm must have professional liability insurance with errors and omissions.

SLIDE NO.

10

DESIGN CONDITIONS: UPLIFT



From Dr. Lytton

POST-TENSIONING INSTITUTE

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July 28, 1999

Mr. John Speed, PE
Executive Director
Texas Board of Professional Engineers
1917 IH 35 South
Austin, TX 78741

Re: Texas Board of Professional Engineers
Policy Advisory 09-98-A

Dear John:

I am writing to you on behalf of the Post-Tensioning Institute and at the direction of our Executive Committee with respect to the above noted policy.

First, let me assure you that PTI and its members strongly support the goals and purpose of Policy Advisory 09-98-A as outlined on page 1 of the policy. However, we have become increasingly concerned with the potential use and application of this Policy Advisory. Although the Policy Advisory clearly states that the Residential Foundation Committee (RFC) reports are not a part of the document and have no official standing, the reality is that some persons may be using the RFC reports in a manner not intended by the Board. They are, given the legal environment in which we operate, making the RFC reports a part of the Policy Advisory, in an effort to create new unintended areas of liability for designers, P/T suppliers and installers.

The Institute's concern is with Appendix A - Interim Procedures for Determining PTI parameters from the Design Sub-Committee report. In the body of the report (on page 5) it refers to Appendix A as *"an acceptable technique"* for developing the PTI parameters. It goes on to state it *"is a modification of the PTI manual procedures and has been recommended by the senior author of the publication."* The Institute's publication was developed, reviewed, revised and voted on by a committee of 30 members. No one member is responsible for the entire work. There was and is no "senior author". PTI supports and endorses the concept of consensus-based documents and has attempted to follow such guidelines in production of all of its publications including the *"Design and Construction of Post-Tensioned Slabs-on-Ground."* However, since the full PTI Slab-on-Ground Committee has never had an opportunity to review and evaluate the technique (as described in Appendix A) and vote on it through the consensus-based process, the Institute by this letter must go on record and state that it **cannot** at this time endorse it and wishes to make it clear that it is **not sanctioned** by the Post-Tensioning Institute in anyway.

Several weeks ago I requested that the PTI Slab-on-Ground Committee undertake a review and evaluation of this modified technique and report back to me with a schedule for completion of this task following the consensus-based process throughout. Because of the urgent nature of this issue, Ken Bondy, the chairman of this committee, has indicated to me, he feels his committee could complete this review and deliver a recommendation to the Institute by the end of October.

The Institute welcomes the guidelines as they relate to evaluation and repair of existing foundations and have so far received favorable feed back on these provisions. Our concerns are related to creating a level playing field for the design of **all** engineered foundations. Given the present direction of the design report and the practical application of Appendix A to only P/T slab-on-ground design, this is clearly not the case. As the policy guideline itself states on page two under Board Rule 22 TAC 131.151(a) *"Inherent in this rule is the notion that an engineer is to provide an **optimized, cost-effective design**"* (emphasis added). That notion and your own interpretation of Board Rule 22 TAC 131.151(b) that *"Engineers must perform their design in a manner which can be favorably measured against generally accepted standards or procedures"* are being put at risk by what appears to be occurring in the marketplace.

Although the Policy Advisory is not a part of any building Code, at least one major city has already advised those designing and/or inspecting engineered residential foundations that they may have legal responsibility under this Policy Advisory. The Policy Advisory on page one makes reference to the reports upon which the policy was written and while stating that *"although the RFC's reports are **not** a part of this policy"* (emphasis added) it further goes on to state *"they provide an interesting and quite valuable commentary on various aspects of engineering related to residential foundation"* and then directs persons to where copies of these reports can be obtained. This has the effect of forcing designers to obtain these reports and to base their designs on the recommendations contained therein. Otherwise, in the hands of a skillful lawyer, the designer would be potentially exposed to numerous cases of nuisance lawsuits and increased liability exposure. How would a jury regard a designer who did not follow **to the letter** the recommendations of a professional engineering board committee upon which a formal Policy Advisory was written? PTI believes that this may be an unintended consequence and an oversight, and respectfully suggests that clarification of the Policy Advisory would be helpful.

While not trying to diminish the valuable contribution of numerous volunteers and the great service this Policy Advisory will ultimately create, the Institute is nevertheless gravely concerned with the policy's current effect in the marketplace and especially on the design of post-tensioned slabs-on-ground. On this point, I would like to recommend that a meeting be arranged between members of PTI's Slab-on-Ground Committee and members of the Texas Board of Professional Engineer and its Design Sub-Committee to discuss our concerns and agree on an action plan to mitigate these issues while ensuring the ultimate goals and objectives of both the Texas Board of Professional Engineers and PTI are maintained and achieved.

Looking forward to hearing from you on this matter.

Yours very truly,

POST-TENSIONING INSTITUTE

Gerard McGuire

Gerard McGuire
Executive Director

CC: PTI Slab-on-Ground Committee members
PTI Executive Committee
Doug Rohrman - PTI legal counsel

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PROFESSIONAL REGISTRATION

Registered Professional Engineer No. 23228 - State of Texas

ACADEMIC BACKGROUND

Bachelor of Science in Civil Engineering in 1957, Texas A & M University.

Master of Science in Civil Engineering in 1959, Texas A & M University.

Post-graduate work at Harvard University in 1969 -- Dr. Arthur Casagrande's specialized course for teachers and practicing engineers in geotechnical engineering.

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. Specialized training includes week long seminars in HEC-1 Hydrology and HEC-2 Hydraulics, upgrading water treatment plants, biological waste treatment, and chemical/physical treatment of wastewater at the University of Texas.

Attendance at numerous seminars and technical continuing education courses in the areas of management, geotechnical engineering, foundation engineering and drainage.

Teaching undergraduate laboratory strength of materials and co-instructor of foundation case studies, both at the University of Texas at Austin.

Presenter of numerous seminars to builders, contractors, engineering societies at state and local level, and building inspectors, all of which pertain to geotechnical engineering and foundation 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 geotechnical and construction materials testing laboratory in Austin.

1964 to present - Owner, Director, Officer and Principal Engineer in MLAW Consultants and Engineers and predecessor organizations, active in structural and geostructural design, pavement engineering, and forensic engineering. Structrual Engineer of Record on hundreds of commercial buildings, apartments, shopping Centers, warehouses, custom homes, retaining walls and over 20,000 single family residences. Civil Engineer of Record for several hundred land development and municipal projects, including subdivisions, municipal utility districts, roads, water and wastewater treatment plants, lines, pumping stations, stand pipes and drainage works.

1985 to present - Owner, Director, President and Principal Engineer in MLA Labs, Inc., a geotechnical and construction materials testing laboratory. 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, Director, President 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

"Foundation Design in Swelling Clays", with R.L. Lytton, presented to Texas Section ASCE, Fall meeting 1966. Recipient of John B. Hawley award.

"Comparison of Performance of Slab-on-Ground Foundations on Expansive and Non-Expansive Soils", with W.L. Snowden, presented to Texas Section ASCE, Fall meeting, 1968.

"Stiffened Mats on Expansive Clay", with R.L. Lytton, July 1971, Journal of Soil Mechanics and Foundation Division, ASCE.

"Utilities for Underground Structures", presentation at the conference Alternatives in Energy Conservation: The Use of Earth Covered Buildings, July, 1975.

"Pavement Design for a Major Truck Terminal in Houston", presented to Texas Section ASCE, Spring meeting, 1979.

"A New Breakthrough in Technology - Roller Compacted Concrete Pavement", article in Texas Civil Engineer, June 1987.

"Experiences with Roller Compacted Concrete Pavement", presented to Texas Section ASCE, Fall meeting, 1987.

"Experiences with Roller Compacted Concrete Pavement in Austin", presented to Twenty-Fifth Paving and Transportation Conference, University of New Mexico, Albuquerque, January, 1988.

"Roller Compacted Concrete Pavement: New Application of Old Technology", presented to American Concrete Institute Convention, Houston, Texas, November 1988.

"Defining Foundation Failure", presented to Texas Section ASCE, Fall meeting, 1991.

"Pavement Design for Cargo Terminal at the Port of Corpus Christi Authority, Corpus Christi, Texas", presented to the American Association of Port Authorities Facilities Engineering Workshop, Corpus Christi, 1999, with William Goldston, P.E.

In addition, Mr. Meyer is the author of a long running series of newsletters "Foundation Topics" distributed in Central Texas.

AFFILIATIONS & COMMUNITY ACTIVIES

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 of Forensic Engineering

Texas Section - American Society of Civil Engineers - (office held - V.P. Professional Affairs)

Austin Branch - American Society of Civil Engineers - (offices held - Secretary, V.P. Programs, President)

Texas Board of Professional Engineers - Chairman of Sub-Committee - Initial Design of Residential Foundations

National Society of Professional Engineers

Texas Society of Professional Engineers

American Society for Testing and Materials

American Concrete Institute

American Association of Cost Engineers - Member

National Forensic Center

Post Tensioning Institute - Member - Slab-on-Ground Committee

Houston Foundation Performance Committee - Member

Tau Beta Pi

Phi Kappa Phi

South Austin Civic Club - (offices held - Secretary, President)

Austin Chamber of Commerce

Capitol City A & M Club - (office held - V.P. Communications)

Association of Former Students - Texas A & M University

Bannockburn Baptist Church - (offices held - Building Committee-Chair, Finance Committee-Chair)

Report of Design Sub-Committee of the Residential Foundation Committee Texas Board of Professional Engineers

1.0 INTRODUCTION

Approximately ½ to 1 billion dollars is spent per year in the state for construction of residential foundations. Their function is to properly support a residential structure without permitting unusual distress or unsafe conditions to occur. The vast majority of foundations constructed in Texas consist of shallow, stiffened and reinforced slab-on-ground foundations. Many of these foundations have to be placed on expansive clays or fills, which must be stable to adequately support the foundation. Other types of foundations include suspended pier and beam, pier and beam constructed on grade, and strip or spread footing type foundations.

The situation of residential foundations which are slab-on-ground in contact with expansive clay soils creates one of the most complex design problems to be encountered in structural engineering. The problem is a soil-structure interaction problem with the added complexity of the soil support conditions changing in response to environmental factors. In this context of technical complexity it is easy for inadequate design procedures or no design procedures to be utilized. The results of poor design and construction of a foundation are sometimes not noticed for several years after the construction is completed.

Because engineered foundations are not required by typical building codes for residential structures, many houses are constructed each year without benefit of engineering input. When such un-regulated structures are placed on problem sites considerable financial loss can occur including over-design, and distress of structures. This report addresses the three problems of lack of engineering requirements, inadequate engineering and poor construction phase practices with regard to residential foundations.

It is not the intent of this report to provide a prescriptive process for design of residential foundations, but to present a methodology which will produce a professional service. Other methods which produce comparable results could be used.

2.0 RECOMMENDED ACTIONS OF CITY AND COUNTY BUILDING OFFICIALS

It is recommended that the Board of Registration encourage local and state legislation promoting the inclusion in local building codes of a requirement for engineered foundations for residences on sites where any of the following conditions exist: the weighted PI using a weight of 3 for upper 5 feet, or the 2 for middle 5 feet and 1 for lower 5 feet. PI of the upper three feet, which ever is greater, has a value greater than 20, the site settlement potential exceeds one inch under the expected structure loads, the structure will be structurally supported over man made fill, when PVR is greater than 1 inch, or sites with geologic hazards. If any of the above conditions exist, the local officials should require an "engineered foundation" as defined below.

3.0 MODEL STANDARD OF PRACTICE

3.1 Definition of "Engineered Foundation"

An engineered foundation is defined as one for which design is based on adequate site specific geotechnical information embodied in a report and prepared by a geotechnical engineer; the design of the foundation is performed by a foundation or structural engineer; and construction phase activities are observed with proper documentation.

3.2 Design Professional's Roles and Responsibilities

The geotechnical investigation and report shall be conducted under the supervision of and properly sealed by a geotechnical engineer. The design of the foundation shall be performed by a foundation or structural engineer and sealed by that person. The geotechnical and structural engineering may be performed by the same individual, provided that individual is sufficiently qualified in both disciplines.

The foundation design engineer will be the foundation engineer of record and this individual shall be responsible for performing the construction phase observations personally, by staff members under his direct supervision, or by outside agencies who are under his direct control. Quality control testing of construction materials as required by the foundation plans, may be provided by an independent testing laboratory, employed by others, provided reports are provided to the foundation engineer of record. The foundation engineer of record shall issue a compliance letter at the conclusion of construction activities stating that, to the best of his knowledge, the foundation was constructed in substantial accordance with the plans and specifications and any authorized modifications. Any modifications, additions, or alterations to the construction of the foundation shall be done by proper change order or modification authorized by the foundation engineer of record.

3.3 Geotechnical Investigation

3.31 Information to be Assembled by Geotechnical Engineer

Prior to laying out the investigative program, the geotechnical engineer will obtain the following, when available: a plat or map of the subdivision, local topography maps, aerial photographs, the existing grades and proposed final grading plan, geological outcrop maps, fault maps (if appropriate), soil conservation service report, and general information concerning the characteristics of the structures to be built.

3.32 Minimum Field Investigation Program

The geotechnical engineer will lay out the proposed investigative program including the location and depths of borings, sampling procedures and laboratory testing required. A minimum investigative 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. As a minimum standard for believed uniform subsurface conditions borings shall be placed at a maximum 300 foot centers across a subdivision. Non-uniform subsurface conditions may require additional borings. A single lot investigated in isolation shall have a minimum of one boring or more placed as determined jointly by the geotechnical engineer in consultation with the foundation engineer. Borings should be a minimum of 15 feet in depth unless confirmed bedrock is encountered at less depth. In certain circumstances, some borings should be placed near trees to obtain depths of probable root penetration. Borings shall extend through any fill or potentially compressible materials even if greater depths are required. All borings shall be sampled by either augured samples or semi-disturbed samples at a minimum interval of one per two feet of boring in the upper 10 feet and at 5 foot intervals below that. Borings shall either be sampled and logged in the field by a geotechnically trained professional or all borings shall be sampled as described such that a geotechnical engineer may examine and confirm the driller's logs in the laboratory.

Investigative borings 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 resort to drilled borings.

Presence of roots shall be logged by depth in all borings.

Intermittent or aquifer water levels shall be noted in borings that indicate groundwater and if material to needs of the project. Observations shall be based on a minimum of one hour observation of the bore holes or pits.

During the field investigations, sketches shall be made of the apparent extent of fill, seepage areas, major vegetation and approximate slopes. The presence of fence lines, old roads or trails or other manmade constructions should also be noted in addition to the borings.

If appropriate, fault studies shall be performed.

3.33 Minimum Laboratory Testing Program

Sufficient laboratory testing shall be performed to identify all significant strata found in the borings across the site. Testing need not be done in every boring provided sufficient correlation can be obtained between borings by a qualified geotechnical professional. Characterization of each significant stratum shall include the moisture content profiles, Atterberg limits, laboratory or field penetrometer estimates of cohesion and the Unified System classification. Additional testing required for each significant stratum will include hydrometer testing to determine the percent fine clay (-2 micron size) and the -#200 sieve size percentage.

At least one volume change/pressure relationship test is recommended to be performed for each significant clay stratum if deemed necessary by the geotechnical engineer. On sites with more than seven feet of expansive clay soils, defined as having plasticity indices greater than 20, it is recommended that at least 1 out of every 10 borings have sufficient samples obtained to provide an in-situ moisture content test and soil suction test at two foot intervals to the entire depth of the boring.

All laboratory testing shall be generally in accordance with ASTM Standards.

3.34 Site Characterization

The geotechnical engineer shall characterize the site for design purposes. If shallow foundations are recommended, report for each lot in the project the Soil Support parameters (including e_m and y_m for edge and center lift modes) as found in Appendix "A", bearing capacity and the presence and condition of fill. In addition, the geotechnical report shall indicate the approximate slopes of each lot and discuss whether downhill creep or other instability may be present. Vegetation shall be noted as well as other visible or known features such as seeps, manmade improvements, fence lines or other linear features and their impact on design. If the site is subject to settlement, an estimate shall be made of the probable settlement based on the proposed structure loading and soil properties. In lieu of the above recommendations, alternate recommendations may be provided by the geotechnical engineer if so requested by the foundation engineer.

The Soil Support parameters shall be developed by calculation using formulations compatible with the principles of unsaturated soil mechanics. Refer to Appendix A for an acceptable technique. These procedures are applicable to all types of shallow foundations on expansive soils, whether reinforced with re-bars or post-tensioning.

If pier and beam foundations are to be recommended, the bearing capacity and establishment depth of piers shall be calculated and values of skin friction or alternate reinforcing steel shall be recommended to be used in uplift and down drag calculations as well as bearing analysis of the piers shall be calculated. The piers shall be anchored below the depth where vertical movement is calculated to be zero or this depth may be based on extensive local experience based on measurements. This depth will be greater within the root zones of major vegetation.

Lateral pressures to be applied in design of retaining type structures shall be determined by the geotechnical engineer.

3.35 Geotechnical Report

Geotechnical reports shall contain, as a minimum, an introduction of the project, the investigative procedures, the laboratory testing procedures utilized, the results of laboratory testing, the geologic conditions, slopes, logs of borings and plans showing boring location, a plan showing areas of probable fill in existence at the time of the investigation and, as appropriate, other features such as seeps, vegetation or lineations and man-made structures.

Reports shall contain specific recommendations for the foundation engineer to use for the design of foundations for each lot on the project, including suitable foundation types and, as appropriate, the Soil Support parameters for each lot, shallow bearing values, deep foundation bearing and skin friction as well as depth of establishment, lateral pressures for use in designing retaining structures or deep grade beams, and other specific recommendations concerning site construction. Sufficient geotechnical data shall be included to permit the foundation engineer of record to adjust design inputs for specific needs of the design. In lieu of the above recommendations, alternate recommendations may be reported if requested by the foundation engineer. The report shall be prepared, signed and sealed by a geotechnical engineer.

3.36 Fill

The presence and methods of dealing with existing and proposed fill to be placed during construction shall be discussed by the geotechnical engineer in his report. Fill criteria useful for design and construction of residential foundations may be seen in Appendix B.

3.4 Design of Foundations

3.41 Information to be Assembled by Foundation Engineer

why not qualified to review

The foundation engineer shall assemble or be provided by his client the subdivision plan, the topography of the area including original and proposed final grades, the geotechnical report, the architectural floor plan of the structure and sufficient additional architectural information to determine the magnitude, construction materials and location of structural loads on the foundation. If exposed or architectural concrete is to be used in finished concrete surfaces, this information should be provided to the foundation engineer.

3.42 Design Procedures

The foundation engineer shall utilize a procedure that will provide designs that will meet minimum criteria of either the

Standard Building Code 1997 Section 1804.3.3.3 or Uniform Building Code 1997 Section 1815, 1816, for the design of slab-on-ground foundations for potentially expansive clay sites. Geotechnical procedures shall be those of Section 3.

Foundations utilizing piers with suspended slab and beams shall be designed by building code procedures.

Soil supported slab-on-ground foundations with piers shall be designed as a stiffened slab on ground by above procedures and piers shall not be attached to the slab or grade beams.

Permissible design deflection ratios of residential foundations shall meet the following criteria:

Construction	Permissible Foundation Deflection Ratios*
Masonry Walls	1/800
Sheetrock Interior Walls - Edge or Center Lift Mode	1/480
Full Span Roof Trusses - Edge Lift Mode	1/960

* Deflection ratio is defined as (Δ)/L where (Δ) is the vertical movement at the center of a symmetrical bowl shaped depression or mound and L is the distance from side to side of the considered depression or mound.

Foundations which will span between piers, or slab panels which span between stiffener beams or perimeter beams or other points of load transfer, shall be designed to meet the above deflection criteria assuming all dead loads and live loads.

3.43 Minimum Plan and Specification Information

The engineer's drawings shall show a plan view of the foundation locating all major structural components and reinforcement. Details shall be shown to indicate construction and dimensions of stiffener beams, piers, retaining walls, drainage details, etc.,if such features are integral to the foundation. Drawings shall contain sufficient information for the proper construction and observation by field personnel. Specifications shall include the reinforcing or pre-stressing cables and hardware; concrete specifications including compressive strengths; notes concerning existing or proposed fill, nearby existing and known future vegetation and the required design features to accommodate these conditions; and the schedule of required inspections.

Minimum perimeter and lot drainage requirements shall be shown or noted on the plan.

Plans shall be specific for each site or lot location and shall include the client's name and engineer's name, address and telephone number and the geotechnical data and source used.

3.44 Foundation Engineer's Specification for Fill

The foundation engineer's 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. Such approval shall include a summary report by the geotechnical engineer of 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 also Appendix B for more detailed information on fills.

3.5 Construction Phase Observation

3.51 Responsibility for Observations

The foundation engineer of record will be responsible for performing observations of construction personally or having them performed by a staff member under his direct supervision or by an independent third party agency qualified to do such observation, which reports to the foundation engineer of record. In any event, the responsibility for issuing the final compliance report shall rest with the foundation engineer of record. Fills which are to support bearing elements of a foundation shall be tested and approved by a geotechnical engineer, assisted by a qualified laboratory.

3.52 Minimum Program of Observation

*A

At a minimum, foundations should be inspected as applicable to see that fill conditions are satisfied in accordance with the plans and specifications; piers are observed for proper placement and depth, concrete and reinforcing installation; observation of all foundation elements immediately ~~before~~ concrete placement; observation of concrete placement; and a stressing observation noting and comparing the elongation of each cable to the calculated elongation and the stressing load applied to each cable.

3.53 Compliance Letter

At the satisfactory accomplishment of all the requirements of the plans and specifications, the foundation engineer of record shall provide a letter to the client indicating the geotechnical data and source used in the design; the design procedures used; the construction observation performed and results; and, as appropriate, a statement concerning the methodology of dealing with fill, either by satisfactory compaction compliance or by penetration; and the methodology of dealing with slopes, vegetation and drainage. The letter shall conclude with a statement that, in his opinion, the construction of the foundation was in substantial conformance with the engineer's plans and specifications including any modifications or alterations authorized.

4.0 REFERENCES

"Design and Construction of Post-Tensioned Slabs-on-Ground", 2nd Edition, (PTI Manual) Post Tensioning Institute, Phoenix, 1996.

American Society for Testing and Materials, West Conshohocken, Pennsylvania.

Standard for Residential Foundations on Expansive Clay (Draft), American Society of Civil Engineers, Reston, Virginia, 1999.

"Prediction of Movement in Expansive Clay", *Vertical and Horizontal Deformations of Foundations and Embankments*, Geotechnical Special Publication No. 40, Lytton, R.L., Yeung, A.T., and Félio, G.Y., ed., ASCE, New York, New York, Vol. 2, 1827-1845, 1994.

APPENDIX A

Procedures for Determining Soil Support Parameters For Shallow Foundations on Expansive Soil Sites

(APPENDIX A IS BEING RE-WRITTEN)

APPENDIX B

Fill Guidelines

FILL GUIDELINES

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

Uncontrolled 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.

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 no wise designer will 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. When designing a foundation for this type of fill, the beam bottoms must penetrate it completely and slab panels should be designed to span between beams if more than 48" will exist below the slab panels.

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 will 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.

BUILDING ON FILL

To establish soil supported foundations on fill, the typical grid beam, stiffened slab foundation is required to penetrate through **forming fill** or **uncontrolled 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 done 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 3 feet, but this depends on the foundation engineer's judgment and local practice.

Pre-existing, uncontrolled fill can be approved through proper investigation by the foundation engineer or a geotechnical engineer. The approval may depend on whether or not the fill is fairly shallow, free of trash, the age of the fill, and the results of testing and proof rolling. These procedures should be performed under the observation and approval of the engineer, who must be able to expressly state after his investigation that the fill is capable of supporting a residential slab-on-ground foundation.

BIOGRAPHY

JOHN THOMAS BRYANT, Ph.D., P.G., P.E.

PRESIDENT
BRYANT CONSULTANTS, INC.
EARTHSYSTEMS TECHNOLOGIES, INC.

Dr. John T. Bryant is a native of New Mexico who was born on February 17, 1962 in Hobbs, New Mexico. He grew up in Hobbs, and he finished his high school education in his hometown. He attended New Mexico Junior College, where he studied science. He further continued his undergraduate education at Texas A&M, where he earned two degrees: one in Civil Engineering and another one in Engineering Geology. He further attended Graduate School at Texas A&M University, where he earned his Master's and his Ph.D. He is currently residing with his wife and his daughter in Plano, Texas.

EDUCATION:

Doctor of Philosophy, (Ph.D.) Civil Engineering (Geotechnical/Constructed Facilities Group)
Texas A&M University, 1991

Master of Science, Geography (Geomorphology)
Texas A&M University, 1987

Bachelor of Science, Engineering Geology
Texas A&M University, 1985

Bachelor of Science, (BSCE) Civil Engineering
Texas A&M University, 1991

Associate in Science, Engineering
New Mexico Junior College, 1983

PROFESSIONAL REGISTRATIONS AND MEMBERSHIPS:

Registered Professional Engineer (Texas), No. 80163

Registered Professional Geologist (Tennessee), No. TN2555

American Institute of Professional Geologists
Certified Professional Geologist, CPG-9926

Committee Member, National Research Council, Transportation Research Committee on Environmental Factors Except Frost, Section on Geology and Properties of Earth Materials, Group 2, A2L06.

PROFESSIONAL BACKGROUND:

As president and founder of Bryant Consultants, Inc., Dr. Bryant is the principal in charge of all engineering operations including forensic, geotechnical, geo-structural and geophysical testing and consulting. He coordinates the operations of the company with other affiliated technical experts when required. His experience also includes large buildings to individual houses, forensic investigations of foundation failure, field and design work for pipeline, tunneling and light rail projects for the City of Farmers Branch and Dallas Area Rapid Transit (DART), use of laterally loaded shaft and slope stability programs to determine safe construction and final slope designs, and use of electrical resistivity Profiles to identify leachate migration from municipal landfills, to locate gravel and sand deposits and for the grounding of transmission towers.

Dr. Bryant has performed work at the world's largest earthwork project located at the Lakes of Arlington in Arlington, Texas and has worked on individual house failures such as the dramatic slope failure shown in the news in Trophy Club, Texas. He has performed work throughout Texas including Dallas, Houston, Austin and San Angelo. Across the country, he has extended his services to Oklahoma, New Mexico, Colorado, Arkansas and Louisiana.

Dr. Bryant has contributed in designing several post-tensioned slab-on-grade foundations for three story hotel structures and for individual residences using the VOLFLO and PTISLAB design programs. During the past few years, several thousand total metric soil suction tests have been performed for structural inspections at numerous residences across the DFW Metroplex to evaluate the distress patterns at these structures and to develop possible causes for the distress conditions. He has also provided expert witness testimony in 30 to 35 various mediations, arbitrations and court cases over the past several years.

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Variation of Soil Suction with Depth in Dallas and Fort Worth, Texas

JOHN T. BRYANT

The variation of soil suction and the estimate of constant soil suction with depth is used to design many slab-on-grade foundations and pavement moisture barriers. The Post-Tensioning Institute's design procedures for slab-on-grade foundations and design of vertical pavement moisture barriers use the constant suction at depth to predict differential soil movements that influence shear, deflection, and moment magnitudes and the effective barrier depth. Constant soil suction estimates can be correlated to the climate or long-term weather conditions at any given site by using the Thornthwaite moisture index (TMI), which estimates the amount of net moisture surplus or deficit from precipitation and evapotranspiration of moisture from the ground surface. On the basis of the empirical curves, the constant value of total soil suction for the Dallas–Fort Worth, Texas (DFW) area is about 246 kPa based on an average TMI of 0. Analysis of more than 1,200 total soil suction laboratory tests performed on developed and undeveloped lots indicates that the measured average total soil suction value in the upper 6 m is closer to 979 kPa for the DFW area ranging between 55 kPa and 11,246 kPa during the 1995–1997 period. Some hypothesized reasons for the difference between the empirical and measured equilibrium (constant) soil suction values are amounts of clay, clay origin, variable plasticity indexes, soluble salt content, and equilibration curve differences.

The variation of soil suction and the estimate of the constant soil suction with depth is used in the design of many slab-on-grade foundations. By using the Post-Tensioning Institute's (PTI's) design method, the design of posttensioned slab-on-grade foundations uses the value of constant suction at depth to aid in the prediction of differential soil movements, which influence the shear, deflection, and moment magnitudes affecting these foundation systems (*1*). Further, the design of vertical moisture barriers for pavement structures and foundations uses the constant soil suction values and the depth to constant soil suction to estimate the effective depth of the barrier.

The estimate of the constant soil suction value can be made on the basis of the climate or long-term weather conditions at any given site. This climatic variable is called the Thornthwaite moisture index (TMI) (*2*) and is used to determine the amount of net moisture surplus or deficit as a result of precipitation and evapotranspiration of moisture from the ground surface. The TMI is a characteristic of a site's climatic influences over a distinct period. A better estimate of the average value of the characteristic constant soil suction is obtained by using longer periods of climatic data for a given site.

The predicted values of soil suction based on the TMI are correlated to published curves (*1,3*). These curves predict slightly different constant soil suction values for respective TMI values, as shown in Figure 1. For the Dallas–Fort Worth, Texas (DFW) area the constant value of total soil suction is on the order of 246 kPa based on an average TMI of 0.

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TOTAL SOIL SUCTION

Soil suction, also known as potential, is a measure of the free energy content of soil water. The theory of soil suction and the related equipment and methods used in its measurement are well-documented. The measurement of suction can be obtained by direct or indirect method. The filter paper method is an indirect measurement in which the filter paper serves as a passive sensor. The basic principle of this method is that a filter paper, after an equilibrium period, exchanges moisture with the soil at a specific soil suction. This occurs because the relative humidity inside the soil specimen container is controlled by the soil water content and suction. Following the equilibrium period, the filter paper will have absorbed moisture equivalent to the relative humidity in the container, and the corresponding suction in the filter paper will be the same as that in the soil specimen.

LABORATORY TESTING

Measurements of the total soil suction used in this research were performed on undisturbed soil samples taken in the field at depths ranging from 0.3 m to more than 12 m below the ground surface using nominal 76-mm-diameter seamless tube samplers. The samples were packaged in the field and were wrapped in foil and placed in a plastic bag to prevent desiccation. Transportation of the soil samples to the laboratory typically occurred within several hours of the sampling.

Laboratory testing of the soil samples for total soil suction were performed in accordance with ASTM D5298-93. The total soil suction test involved placing the soil samples into sealed containers with calibrated filter papers and allowing approximately 7 days for the relative humidity within the container to come into equilibrium with the pore water vapor pressure inside the soil interstices.

Deviations from the ASTM D5298-93 apparatus requirements were as follows:

1. Whatman No. 42 ashless 55-mm filter paper was used. No special pretreatment of the filter paper was applied.
2. A 348-mL polyethylene specimen container was used instead of a metal or glass container. The container had a clamp seal.
3. Two wraps of electrical tape, approximately 6 mm wide, were used instead of the flexible plastic electrical tape to further seal the outside lid–container connection.
4. Rubber O-rings were used instead of a screen wire or brass discs to separate the filter papers during equilibration.

All weighing and transfer of the filter papers from the specimen container into the metal weighing container was performed by a trained laboratory geotechnician or by the author. The filter paper moisture contents were converted to suction values by using the Whatman No. 42 calibration curve given in ASTM D5298-93.

GEOLOGIC CONDITIONS OF AREA

The DFW area lies within the Upper Cretaceous and Lower Cretaceous sedimentary rock and Quaternary aged alluvial deposits. The sedimentary rock strata dip gradually toward the south and southeast and increase in age from east to west. The sedimentary strata present from approximately the east boundary to the west boundary of the area consist of the following, in order: Ozan–Lower Taylor marl; Austin chalk limestone; Eagle Ford shale; Woodbine formation including sands, clays shales, and sandstones; Main Street/Paw Paw limestone; and Paluxy formation consisting predominantly of sands and sandstone strata. The interbedded sedimentary rock formations typically are dissected by the Trinity River and its tributaries, which have deposited Quaternary aged sands, silts, sandy clays, and gravel along the present and ancient channels and flood plains of these rivers and creeks. Samples from all of these formations are combined in the analysis of the total soil suction variation across the DFW area.

ANALYSIS AND DISCUSSION OF RESULTS

Figure 1 provides a graphical comparison of the Russam-Coleman and PTI soil suction curves as functions of the TMI. The soil suction values are reported in pF units (logarithm to the base 10 of the negative pore pressure in centimeters of water). The curves are of similar shape, although the Russam-Coleman curve overpredicts the PTI curve for dry climates with TMI values less than −10. Conversely, the Russam-Coleman curve underpredicts the PTI curve for wet climates with TMI values greater than 0.

The DFW area is approximately bounded by the TMI values of −10 to 10 with an average value on the order of 0. The Russam-Coleman and the PTI curves indicate that the equilibrium suction for soils in the DFW area are on the order of 246 kPa. However, the curves shown in Figure 1 appear to underpredict the actual measured values of the total soil suction values for the DFW area measured between 1995 and 1997.

Figures 2, 3, and 4 show the variation of the total soil suction with depth across the DFW area during 1995 through 1997. Soil samples were taken in both developed and undeveloped areas so that a range of the total soil suction values preconstruction and postconstruction could be estimated. Figures 2, 3, and 4 represent 1,225 separate independent laboratory measurements of total soil suction using the filter paper method. Figure 2 presents the suction profile measured in 1995, and Figures 3 and 4 present the suction profiles measured in 1996 and 1997, respectively. Table 1 presents a statistical summary of the soil suction data collected for this study. The results of these soil suction tests fall between 10 kPa (2 pF) and 97 948 kPa (6 pF), which are considered to be extreme values for soil suction at the field capacity and an extreme controlling dry suction.

Review of Figures 2, 3, and 4 reveals that the total soil suction is most variable at the surface of the ground, becoming slightly less variable with depth. Table 1 indicates that the average total soil suction values are on the order of 979 kPa (4 pF) for the DFW area over the last 2.5 years. This number is substantially higher than the value of 246 kPa (3.3 to 3.4 pF) predicted by Russam-Coleman or PTI from Figure 1, and it underpredicts the actual measured total soil suction value for the DFW area between 1995 and 1997. The skew shown in Figure 4 in the 1997 results most probably is due to the

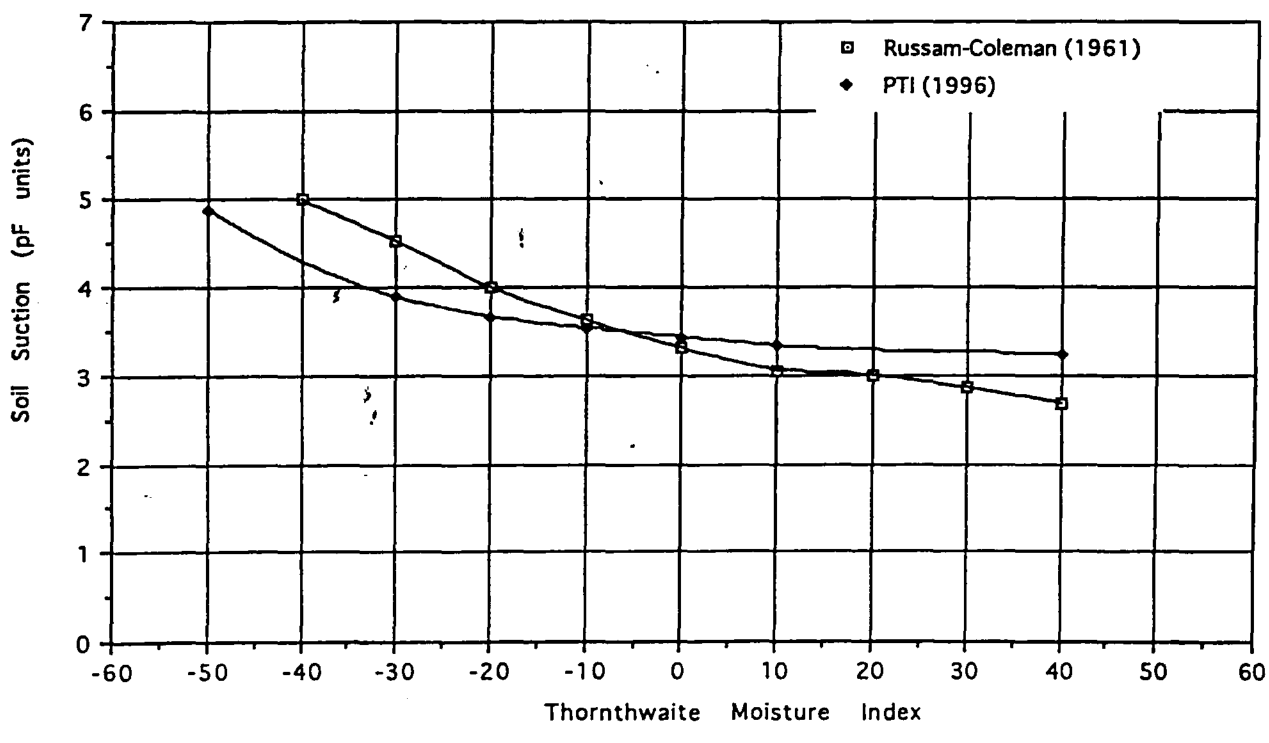

FIGURE 1 Comparison of Post-Tensioning Institute and Russam-Coleman soil suction variation with Thornthwaite moisture index.

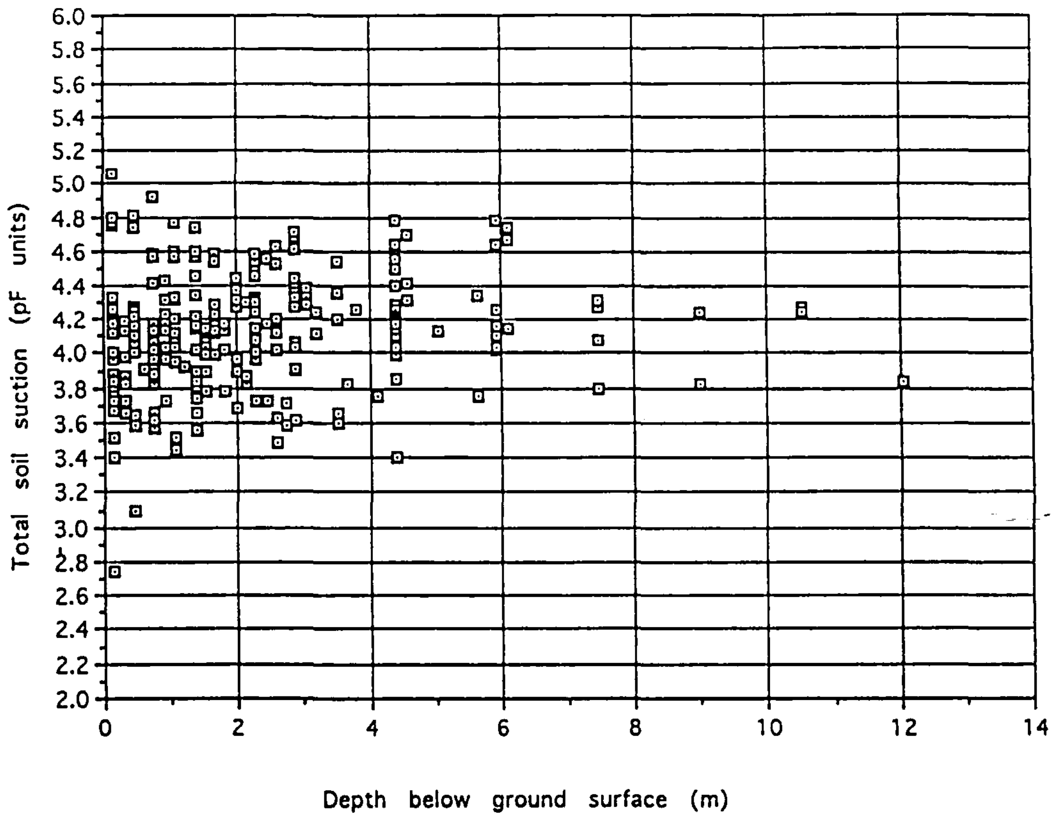

FIGURE 2 **Total soil suction profile, Dallas–Fort Worth, 1995.**

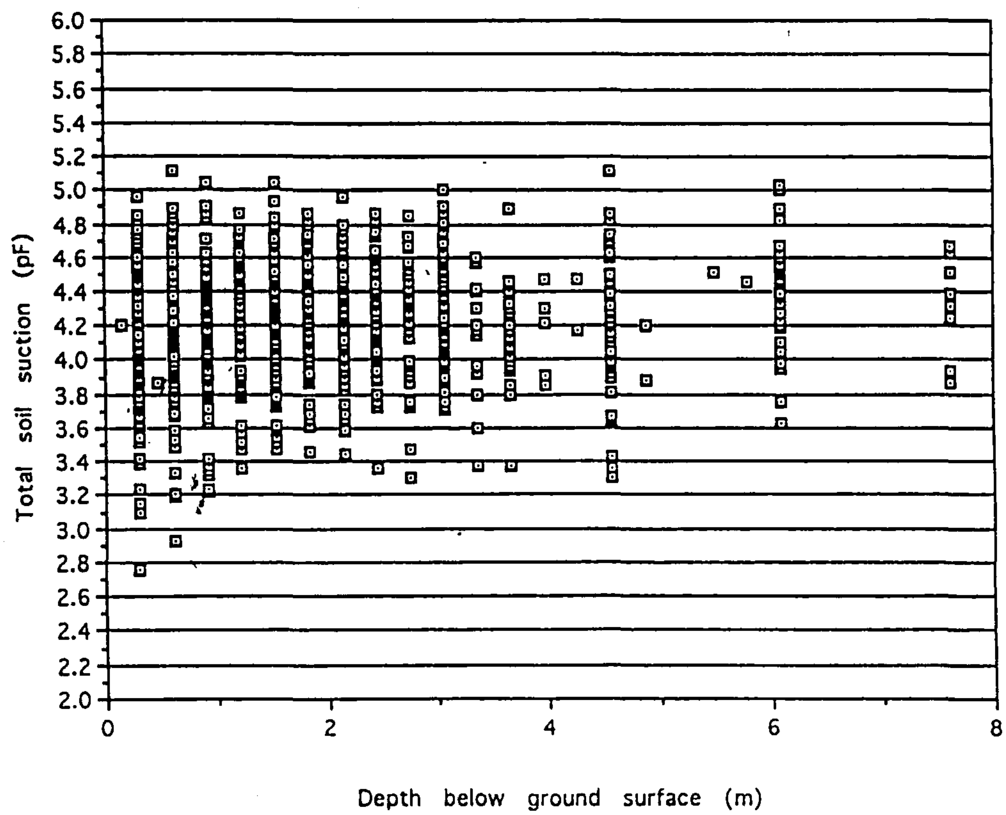

FIGURE 3 **Total soil suction profile, Dallas–Fort Worth, 1996.**

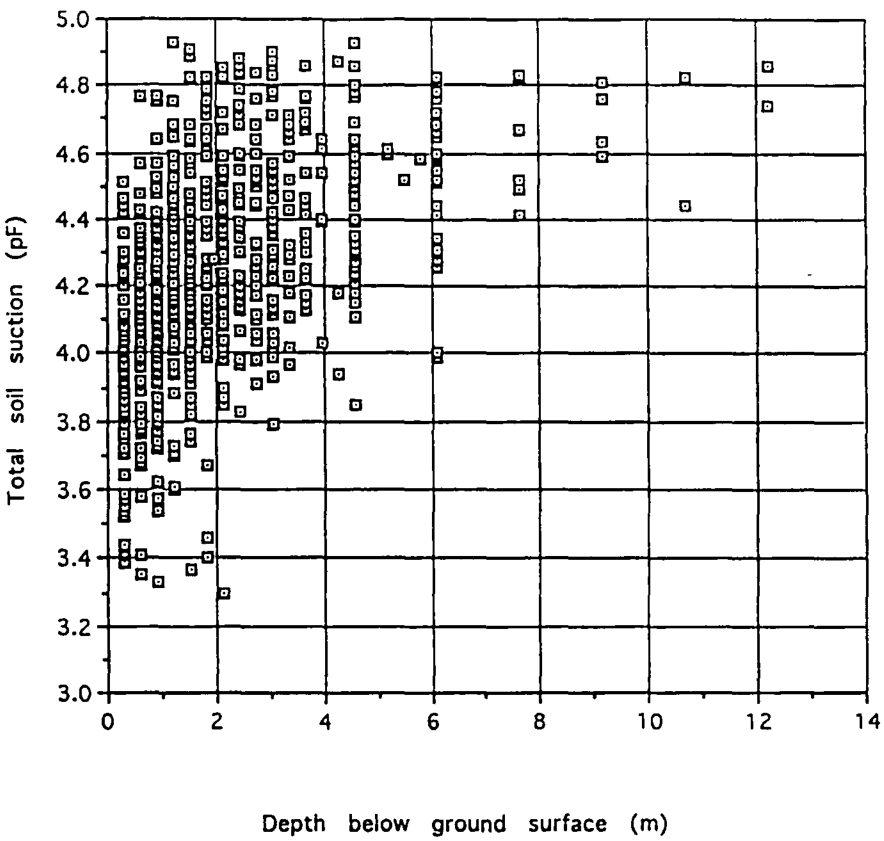

FIGURE 4 Total soil suction profile, Dallas–Fort Worth, 1997.

influence of more rock measurements at depth, which increase the total soil suction values because of their internal fabric, lower moisture content, and composition.

Table 1 also indicates that the range of total soil suction values tends to decrease from about 2.313 in 1995 to about 1.63 in 1997. This corresponds, as shown in Figure 5, to a progression from a near-normal precipitation period experienced in 1995 and early 1996 to an above-normal or wet period from the last part of 1996 to 1997 in the DFW area.

A cursory visual analysis of the soil suction data indicates that the residual clay and sandy clay soils weathered in place from the parent rock materials typically have higher total soil suction values than the alluvial soils. The alluvial soils tend to have lower total soil suction values corresponding more closely to those predicted in the Russam-Coleman diagram. Quantitative analysis of this hypothesis should be the subject of future research.

TABLE 1 Statistical Summary of Total Soil Suction Measurements in pF Units, Dallas–Fort Worth, 1995–1997

YEAR	1995	1996	1997
AVERAGE	4.1384	4.1675	4.2482
MEDIAN	4.15	4.18	4.26
COUNT	252	308	665
MINIMUM	2.75	2.76	3.30
MAXIMUM	5.06	4.82	4.93
RANGE	2.313	2.06	1.63
STANDARD DEVIATION	0.3303	0.3606	0.3233

Another fundamental question to answer for future research is "What is the mechanism responsible for the differences between the empirical or theoretical predictions of soils suction based on the TMI and the actual observed field measurements?" Some hypotheses for this mechanism to explain the differences between empirical predictions of total soil suction and the actual measured values are as follows:

- Residual clay and sandy clay soils that weathered from the parent material in place may have more complete rock fabric and cementation than the alluvial clay and sandy clay soils deposited by recent river or creek processes;
- Different amounts of clay with alluvial soils may contain more silty and sand fractions;
- Variable plasticity indexes may be present;
- Amounts of soluble salts may be higher in residual clays weathered in a semiarid climate; and
- There may be differences in equilibration curves for a highly structured residual rock fabric versus the alluvial deposited clay soils.

Lytton (*4*) found discrepancies between the empirical Russam-Coleman relation and observed value of suction measured in the field. Lytton reasoned that these discrepancies do not call the value of the empirical Russam-Coleman relation into question; rather, the discrepancies emphasize the need to determine the equilibrium suction on a more fundamental basis, which includes use of the soil's desorption suction-versus-volumetric water-content characteristic curve on any given site.

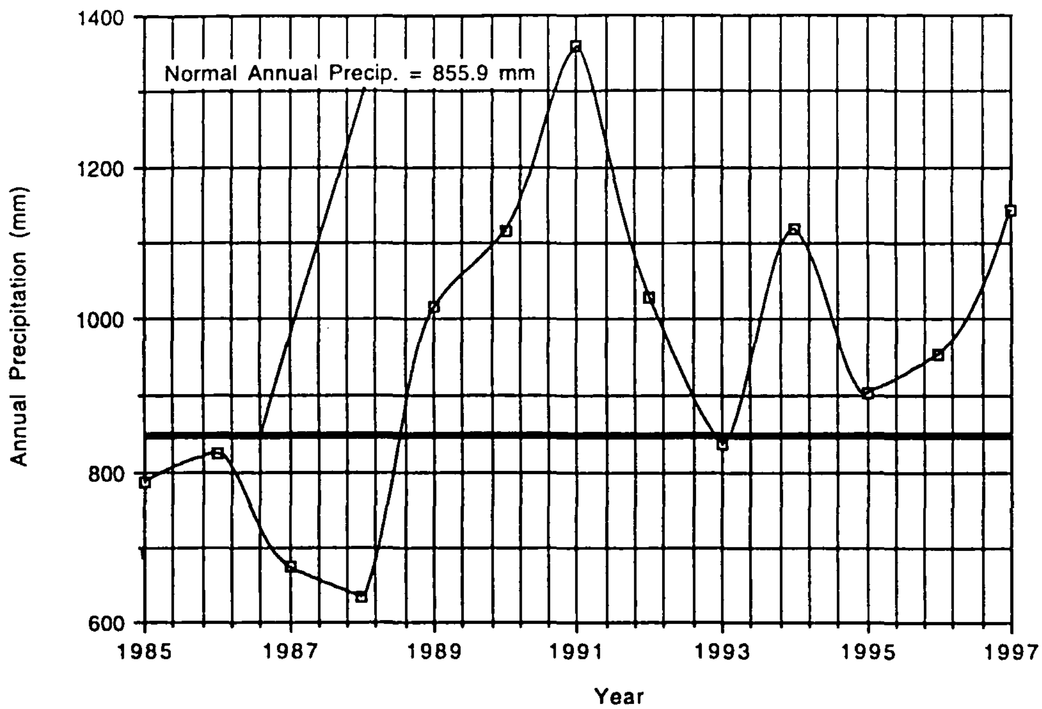

FIGURE 5 Annual precipitation for Dallas–Fort Worth, 1985–1997.

Gay (5) developed a mathematical model to calculate the soil's desorption suction-versus-volumetric water-content characteristic curve. Lytton (4) concludes that higher equilibrium soil suction values can be produced by using the analysis of the desorption suction-versus-volumetric water-content characteristic curve on any given site, and that using the relationships can provide an equilibrium soil suction value routinely.

CONCLUSIONS

On the basis of the results of this research, the following conclusions can be drawn:

1. The measured average total soil suction value or constant soil suction value for the DFW area measured over the last 2.5 years is estimated to be on the order of 979 kPa.
2. The range of total soil suction values was the greatest at the surface and decreased with depth with a minimum measured value of 55 kPa (2.75 pF) and the maximum measured value of 11 246 kPa (5.06 pF).
3. The range of measured total soil suction measurements decreased from 1995 to 1997, which generally corresponds to higher precipitation during the later part of 1996 and 1997.
4. Additional research into the distribution of total soil suction values with depth in the DFW area and across the United States is necessary to understand the variability and range of the total soil suction values used in pavement and slab structure design.
5. Additional research is needed to quantify the differences between the total soil suction values of residual clay and shaley clay soils that weathered from the parent material in place and sandy clay soils deposited from recent river and creek alluvial and fluvial depositional processes.

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1. *Design and Construction of Post-Tensioned Slabs-on-Ground.* Post-Tensioning Institute, Phoenix, Ariz., 1996.
2. Thornthwaite, C. W. An Approach Toward a Rational Classification of Climate. *Geographical Review*, Vol. 38, No. 1, 1948, pp. 55–94.
3. Russam, K., and J. D. Coleman. The Effect of Climatic Factors on Subgrade Moisture Conditions. *Geotechnique*, Vol. 11, No. 1, 1961, pp. 22–28.
4. Lytton, R. L. Engineering Structures in Expansive Clay Soils. *1997 Seminar on Current Practice & Advancement of Post-Tensioned Residential Slabs on Expansive Soil.* Post-Tensioning Institute, 1997.
5. Gay, D. E. *Development of a Predictive Model for Pavement Roughness on Expansive Clay.* Ph.D. dissertation. Texas A & M University, College Station, Tex. 1993.

Publication of this paper sponsored by Committee on Environmental Factors Except Frost.

DESIGN OF FOUNDATIONS ON EXPANSIVE SOILS

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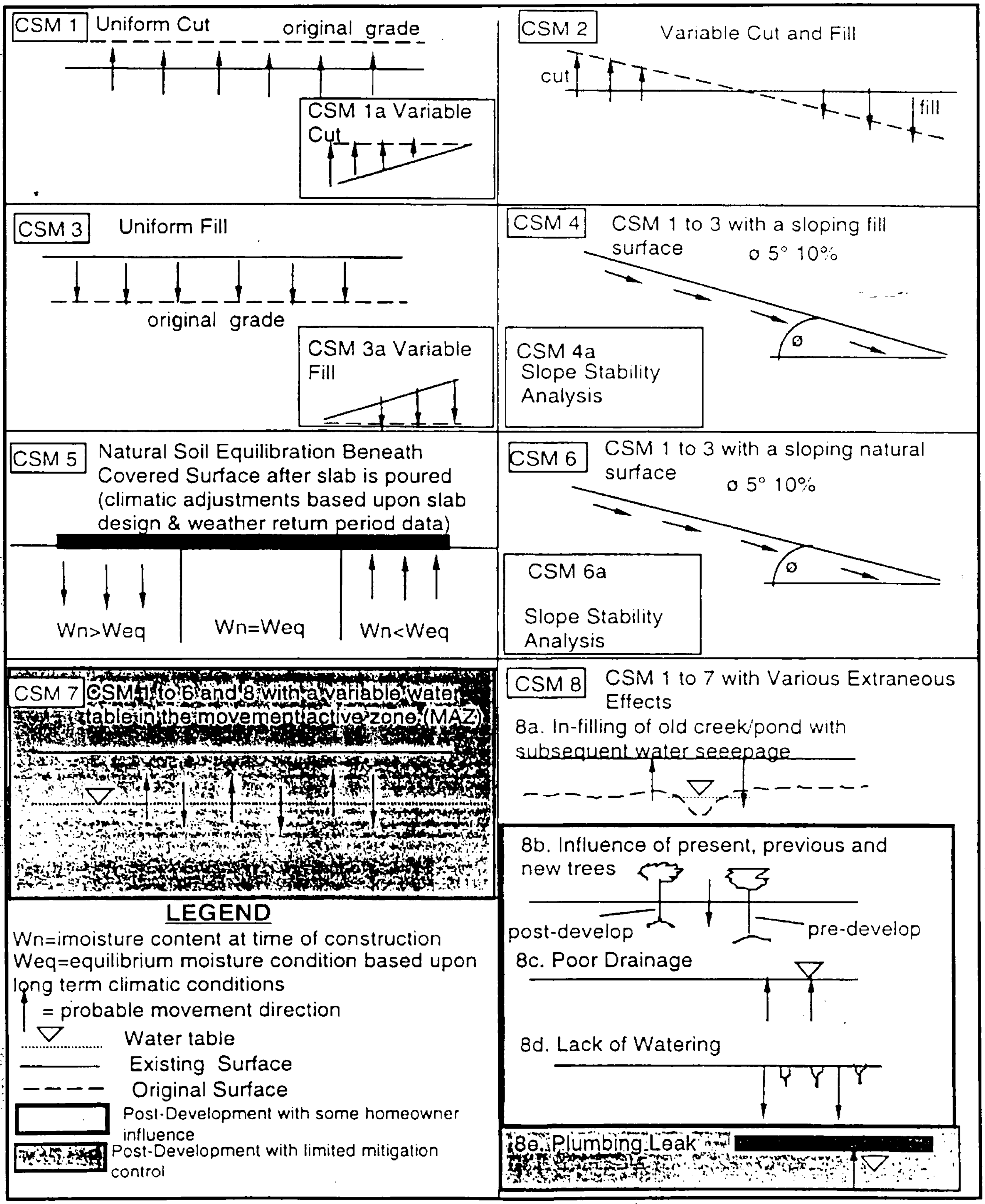
SITE DEVELOPMENT DETAILS



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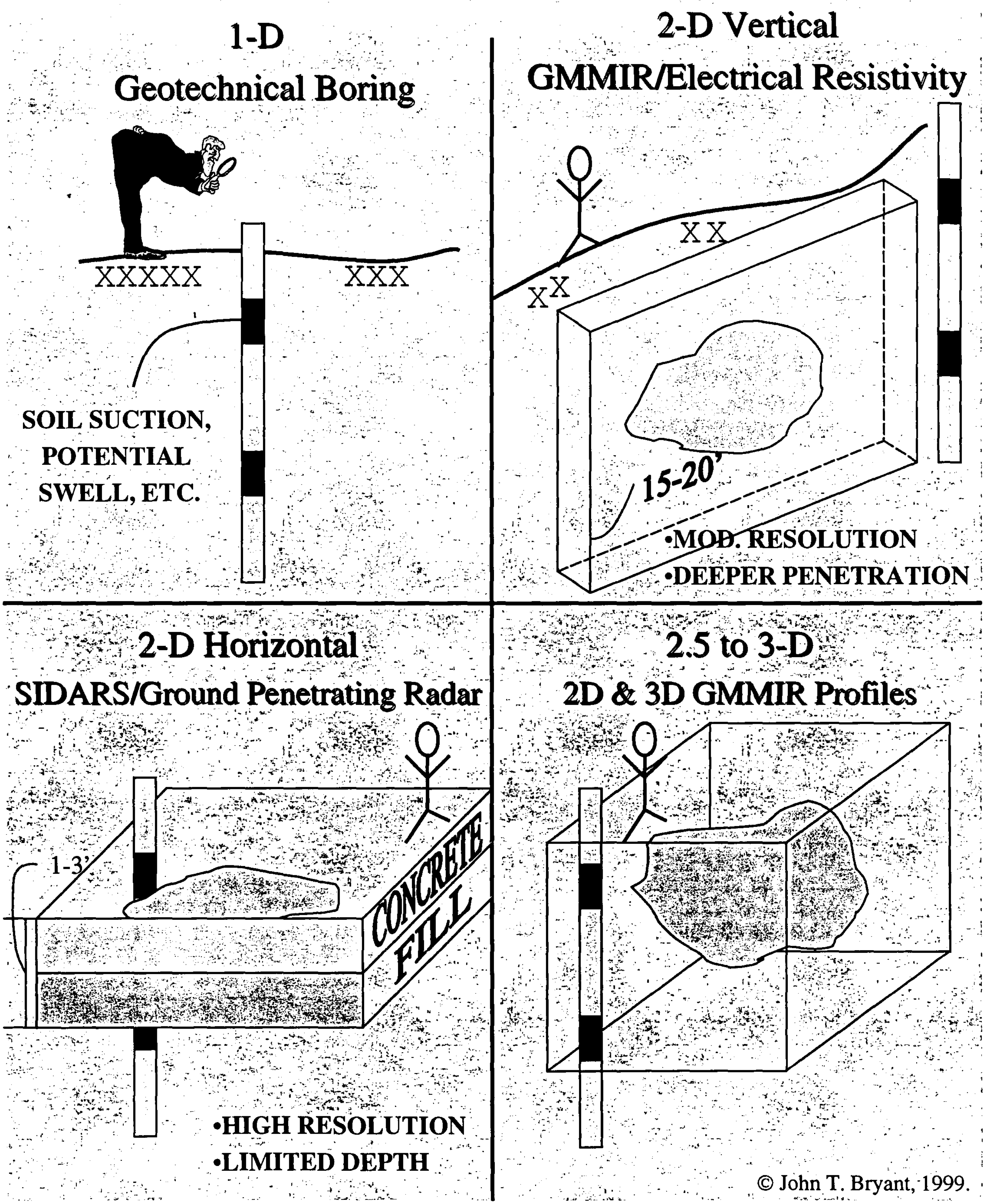
SUBSURFACE EXPLORATIONS



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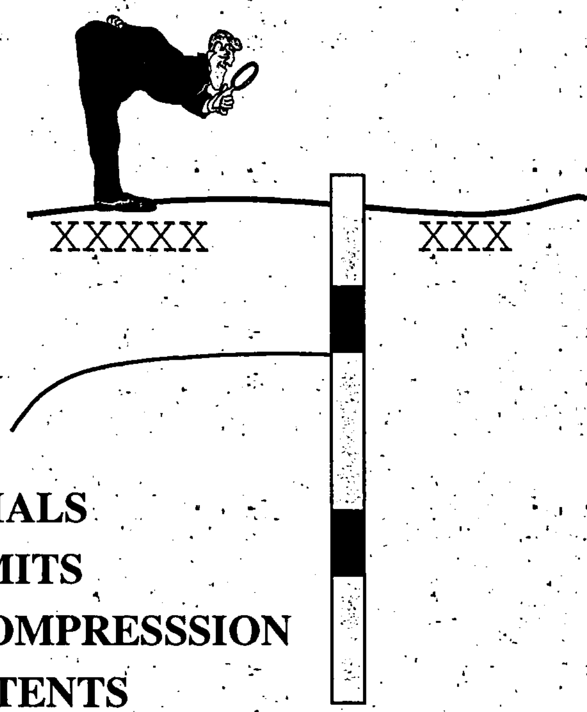
1-D FIELD EXPLORATION

- Geotechnical Boring
- Trial Pit
- Trial Trench



- SOIL SUCTION
- SWELL POTENTIALS
- ATTENBERG LIMITS
- UNCONFINED COMPRESSSION
- MOISTURE CONTENTS
- STRATA IDENTIFICATION
- HAND PENETROMETER
- THD PENETRATION
- SPLIT SPOON, ETC.

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DRIELLARS??!!??

FIELD LOG of BORING

B-1 See Figure 1

PROJECT: ~~92506A~~ 92506 M BORING #: B-A1
DATE: 11-27-92 JOB #: 92-2856M GROUND ELEVATION:
TYPE: LOCATION:
DRILLER: LOGGER: DRILL RIG: B-53
Pushed to ____ feet Drilled with ____ Mud used ____ Cased to ____ ft.

Depth ft.	Sample Type	Std. Pen. 1st 6"	2nd 6"	3rd 6"	Hand Pen. tsf	% REC.	% RQD	LEGEND: ■ -SHELBY TUBE ☒ -STD. PEN. ▼ -WATER ◫ -N/X CORE ◪ -THD ⧅ -NO RECOVERY ◧ -AUGER
	■				2.5			0-4 Fill
	☒	6	10	11				
5	■				4.5+			4-6 Cal. Dark Clay
	☒		50/10"					6 to 9 Drove spoon 50 for 0 Hard rock Took Hogger Sample
10								
15	☒	50 for 3						9-15 Alter Rock, Tan Limestone
20	☒ THD	50/4	50/4 CORE					15-20 Blue Shelly Limestone
25								
30								T.D. 20
35								
40								0-4 Very stiff to hard, brown to orange brown clay with embedded chalky white calcareous fragments, gravel, trace silt. Some soft layers 2-4" thick
								4-6 Very stiff Dark brown clay with limestone fragments
45								6-15 Tan to olive brown weathered limestone with clay seams and layers
								15-20 Gray to ~~dark~~ olive gray ~~limestone~~ with shale seams and layers
50								

WATER INFORMATION:
Seepage at ____ feet at ____ Water at ____ feet at ____
Bailed to ____ feet at ____ Water at ____ feet at ____

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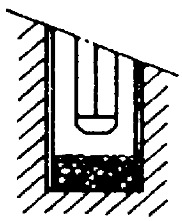
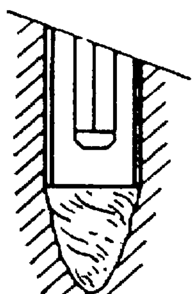
SAMPLING TECHNIQUES vs. STRATUMS

INSPECTION IS PROVIDED TO CONTROL:

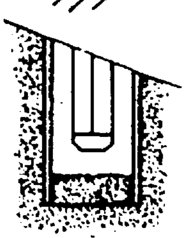
1. Correct Driving Energy
2. Sampler Type
3. Sampler Condition
4. Sampling Sequence
5. Sample Identification
6. Sample Preservation
7. Condition at Sampling Depth



Sampler dropping on gravel or cinders not cleaned from casing, results in high blow count.

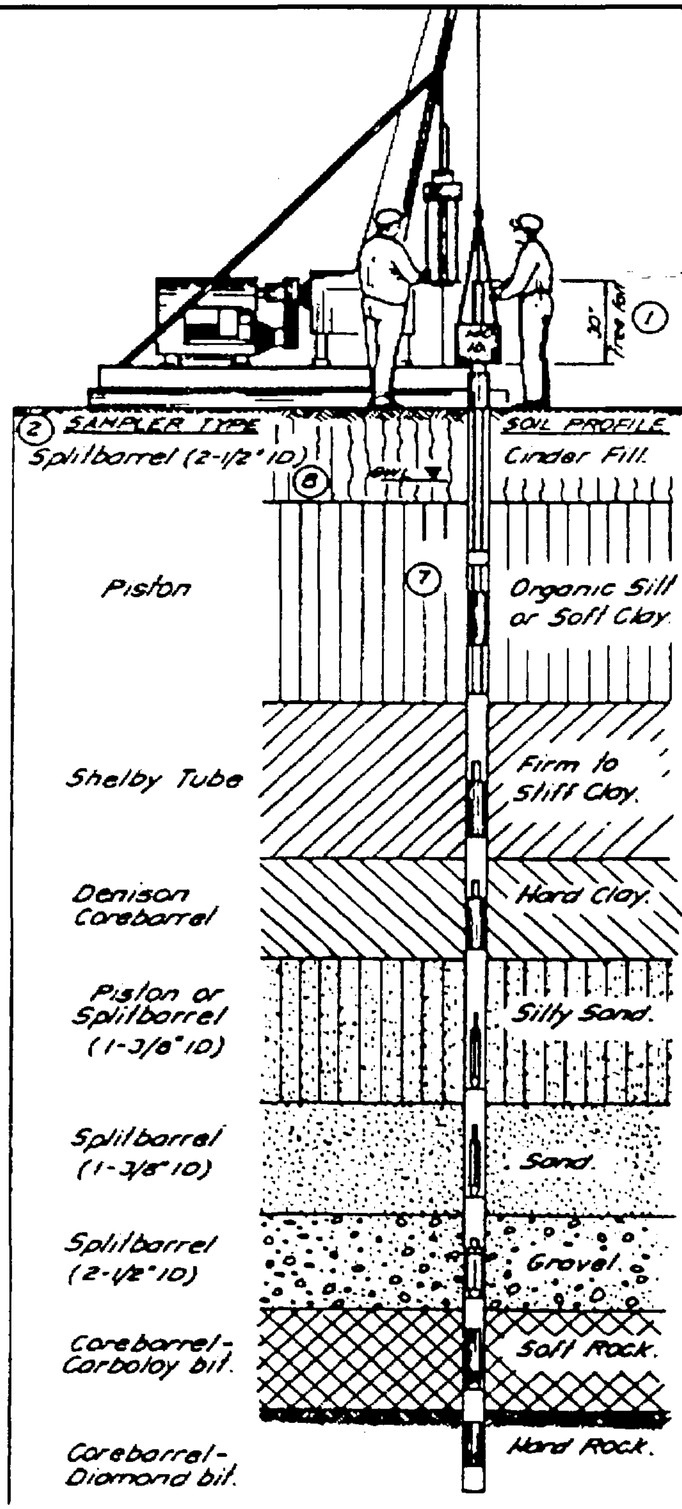


Soils loosened by overwashing. Blow count will be lower than true count.



Sand under hydrostatic pressure plugging casing. Blow count will be higher than true.

8. Groundwater Measurements
9. Depth of Boring
10. Sample Recovery - Percentage



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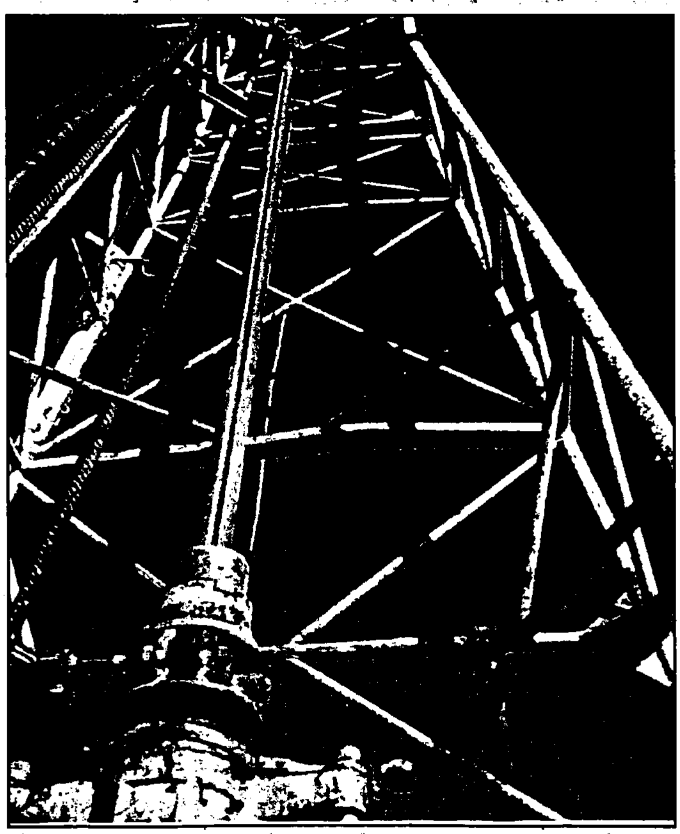
GEOTECHNICAL DRILLING RIG

Oblique view of a typical drilling rig with mud pit.



Close-up of the Kelly bar connection to mud swivel and the supporting raised mask.

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ASTM Laboratory Testing
for

•Potential Vertical Swell
•Soil Suction

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SOIL SUCTION

Understanding the relationship between unsaturated soil mechanics and foundation design and performance, expansive and compacted soils, etc. is critical. Soil suction is perhaps the most important factor in unsaturated soil mechanics.

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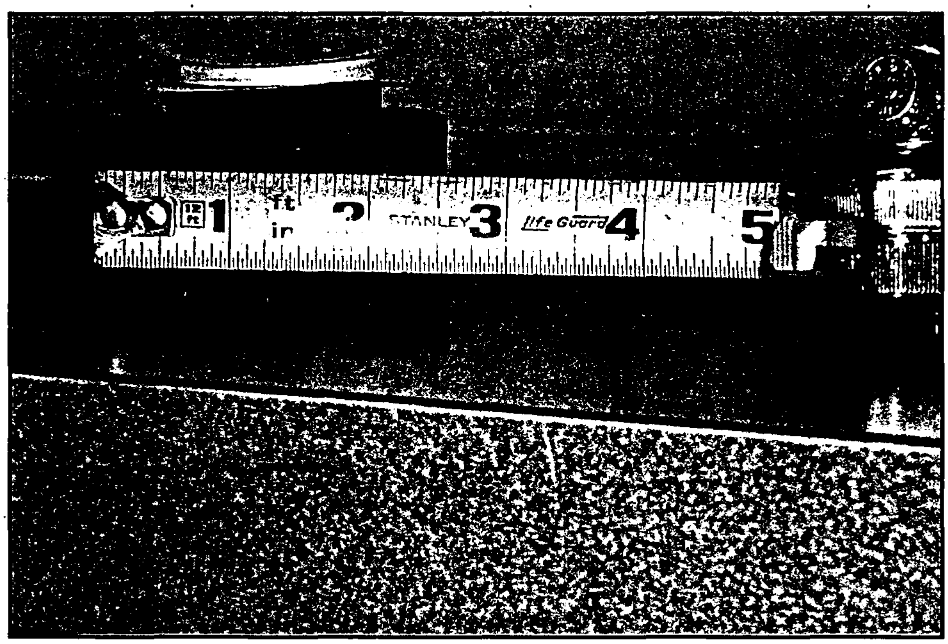
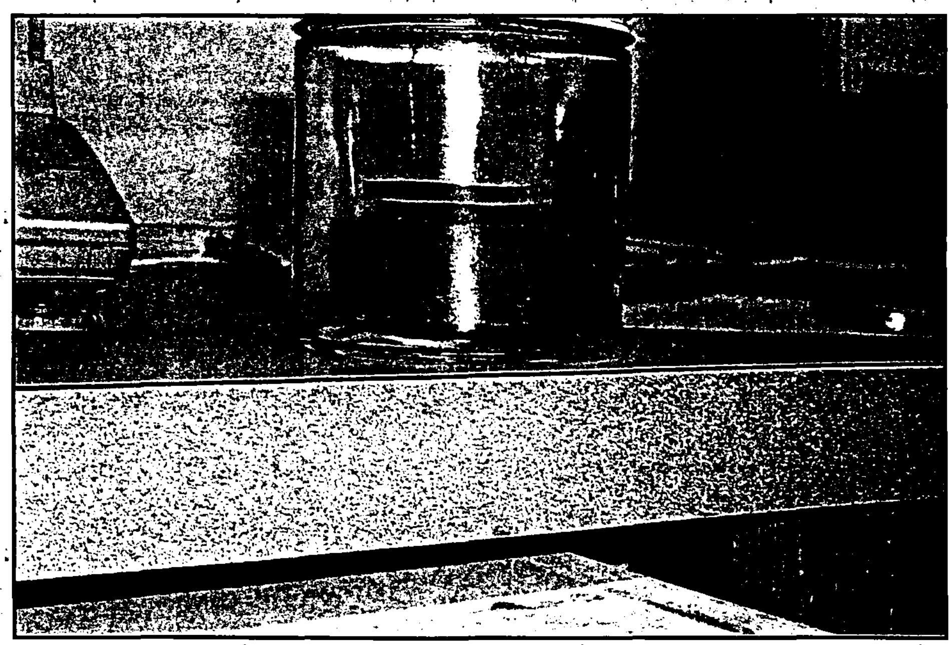
ASTM D 5298

Basic Procedures to determine Total Soil Suction



An intact 115 to 230 gram soil specimen is required.



Isolate two filter papers on top of the soil specimen
and place in air tight container.

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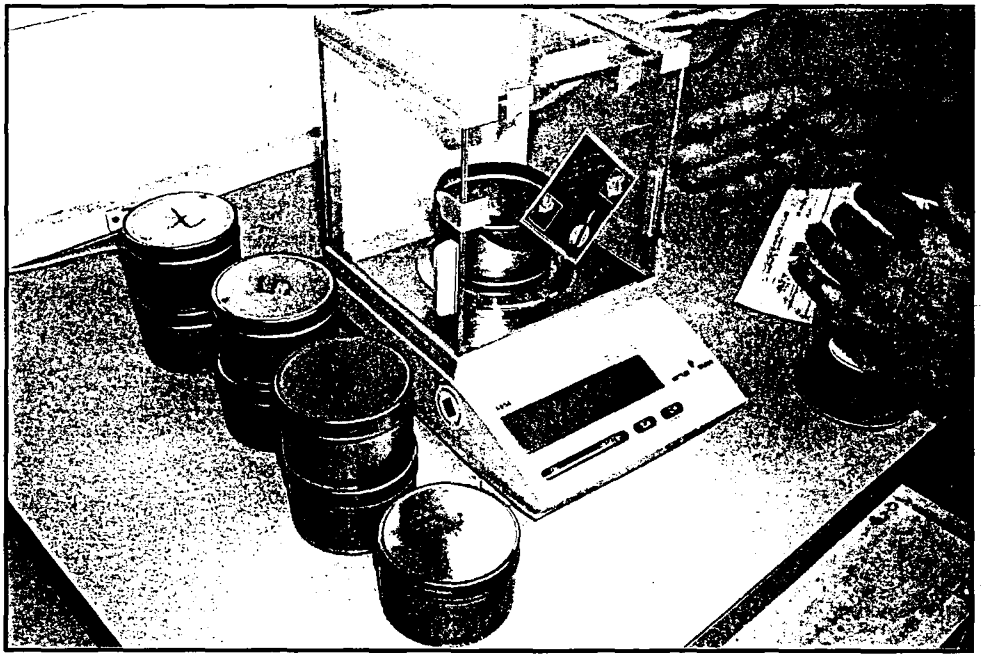
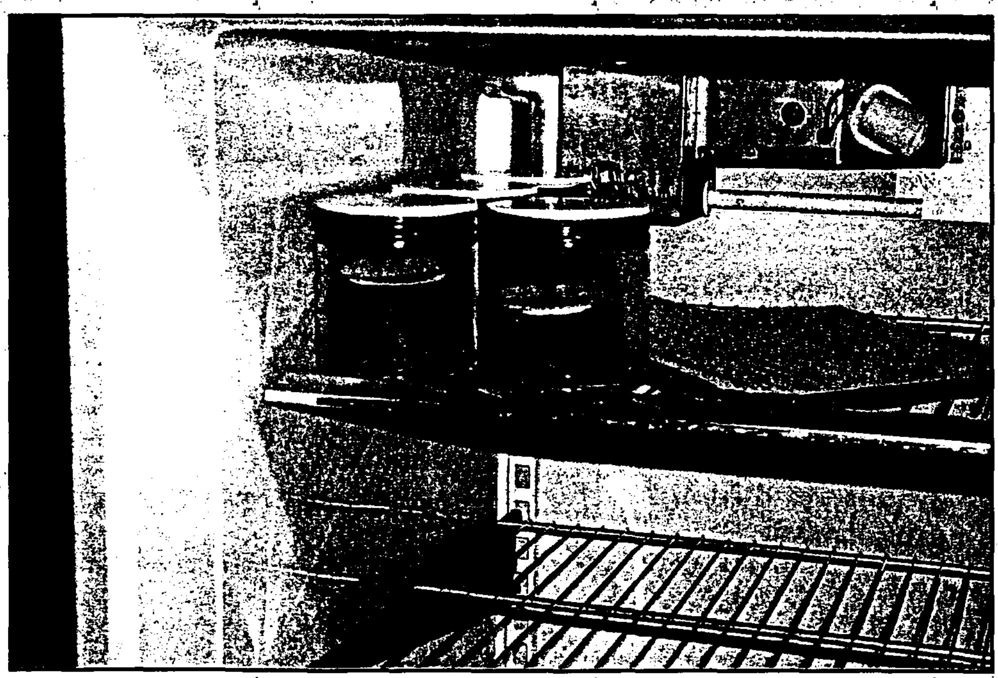
ASTM D 5298



Accurate measurements are a necessity (±0.0001) in determining total soil suction values.



Place in samples in desiccator for seven days to achieve equilibration.

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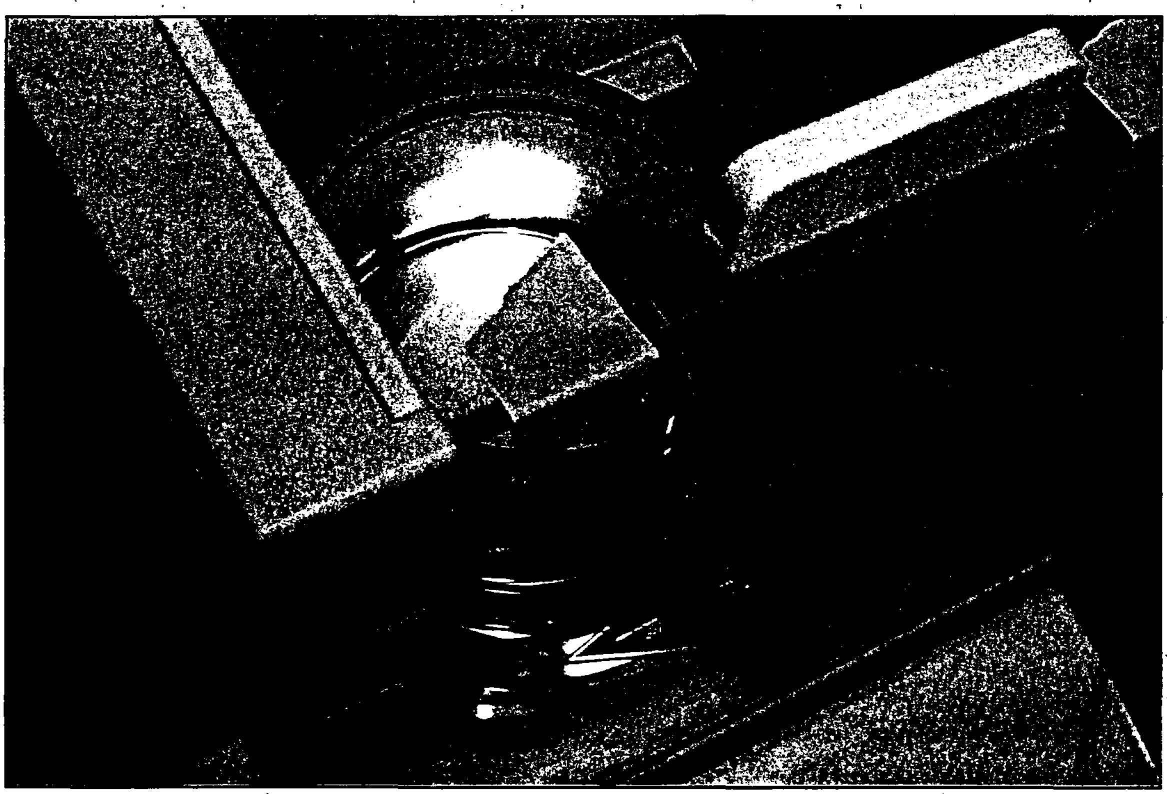
ASTM D 5298



Average moisture content from the two filter papers is used in determining total suction values.

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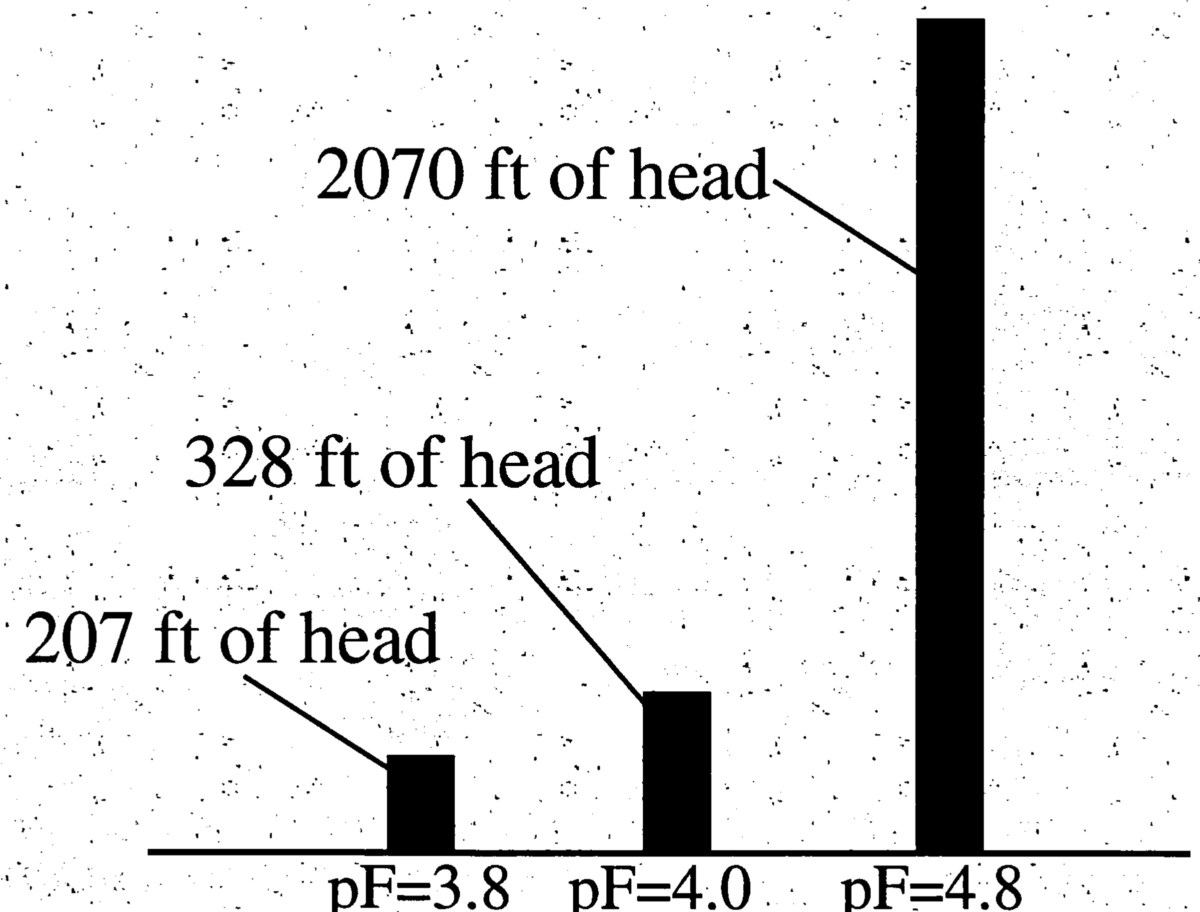
UNITS OF SUCTION VALUES

Soil suction value is normally reported in pF. This is a unit of negative pressure expressed as the $\log_{10}$ of head of water.



Average Soil Suction for DFW area = 4.0 pF*

*based on Bryant, J.T., Variations of Soil Suction with Depth in DFW, *TRB*, No. 1615, 1997.

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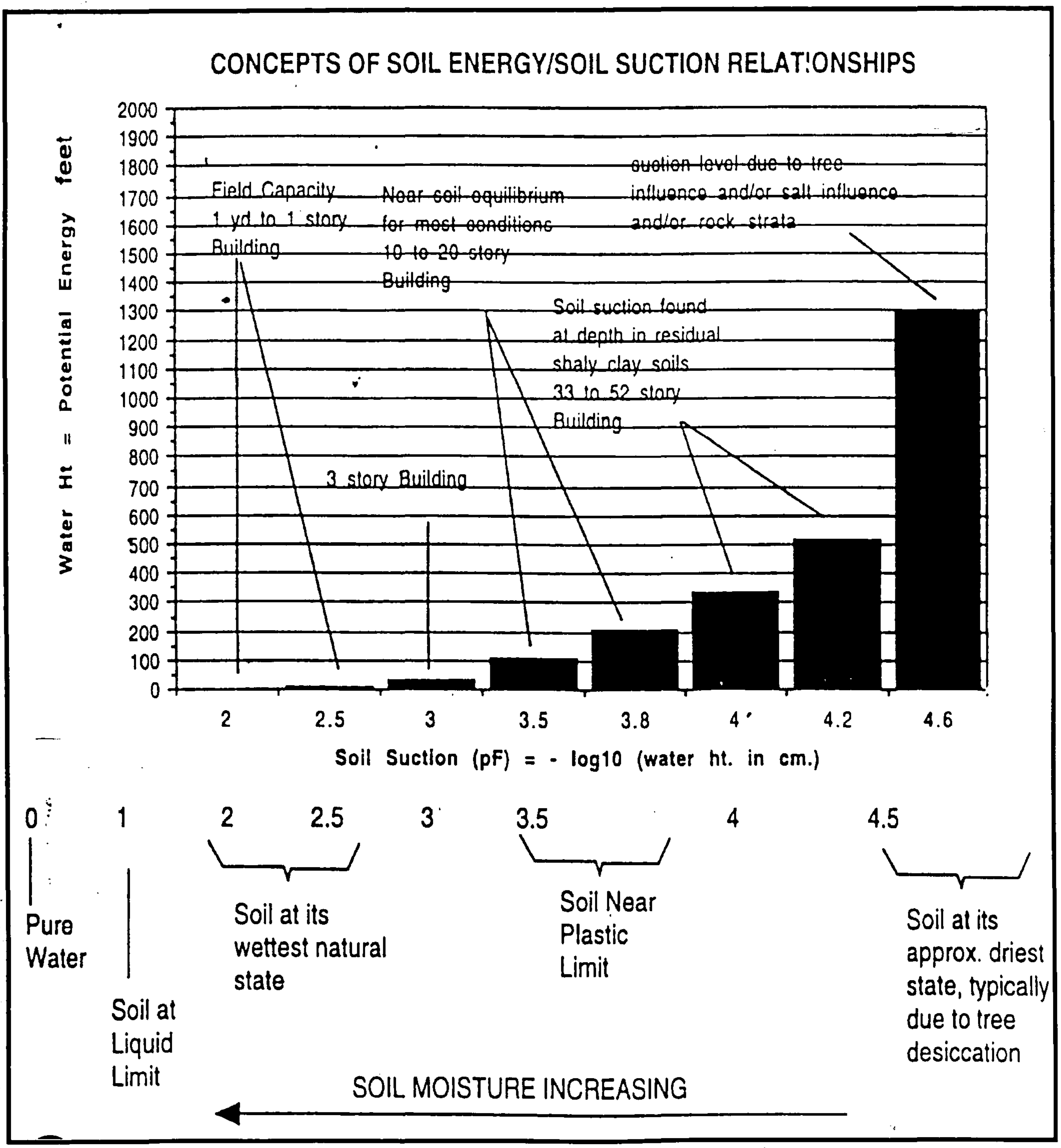
SOIL ENERGY/SOIL SUCTION RELATIONSHIPS



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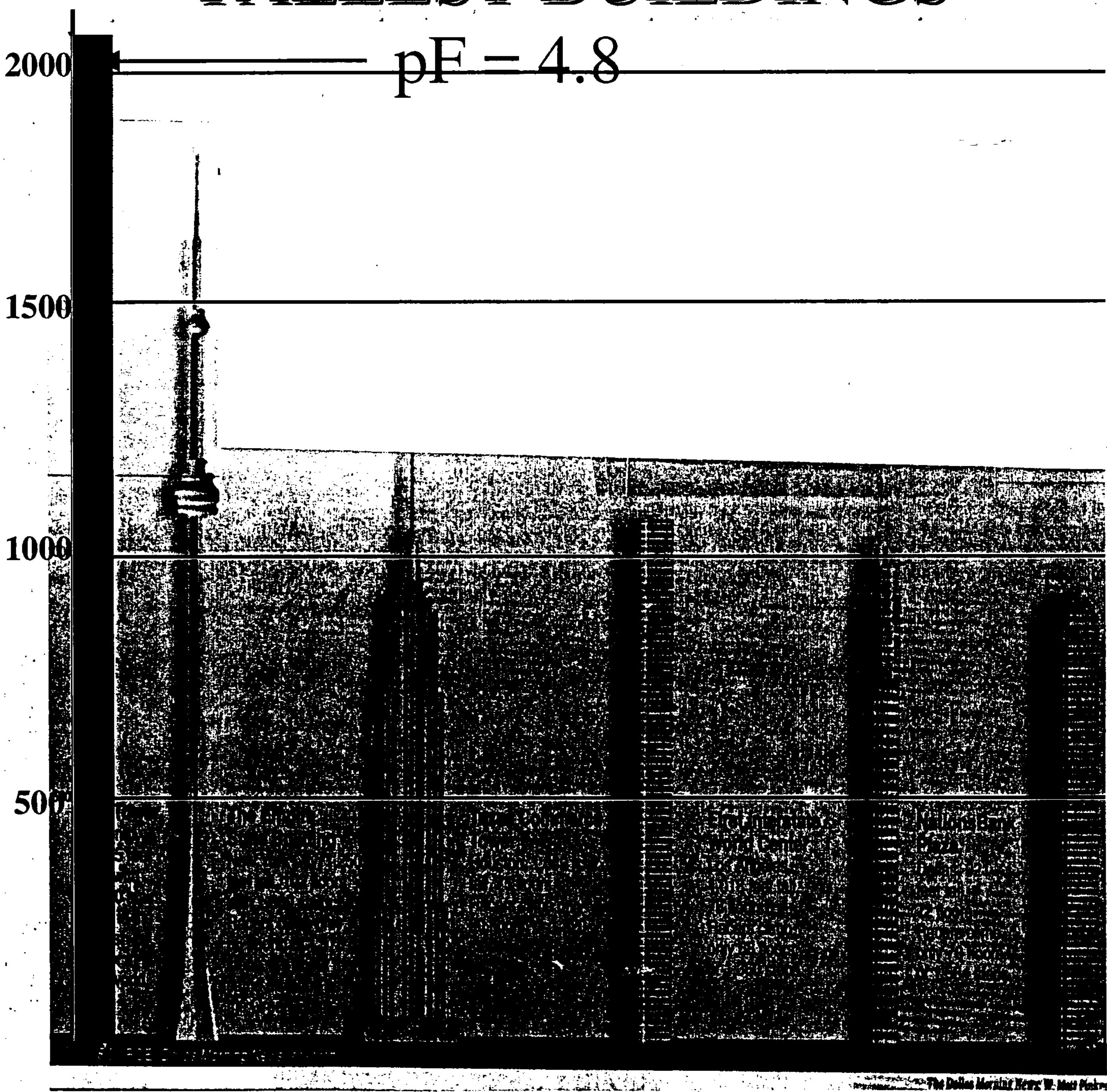
SOIL ENERGY COMPARED AGAINST THE WORLD'S TALLEST BUILDINGS



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CONCLUDING POINTS on SOIL SUCTION

- Soil suction is a measure of the free energy content of soil water.
- Determining soil suction values based on ASTM D 5298 is relatively easy and inexpensive.
- pF is a value equal to the $\log_{10}$ of the pressure head (in centimeters)
- Understanding soil suction and its vital role in unsaturated soil mechanics is essential for proper foundation design.

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FREE SWELL

The potential free swell/settlement of soil can be used to develop estimates of heave or settlement for a given moisture and loading conditions.

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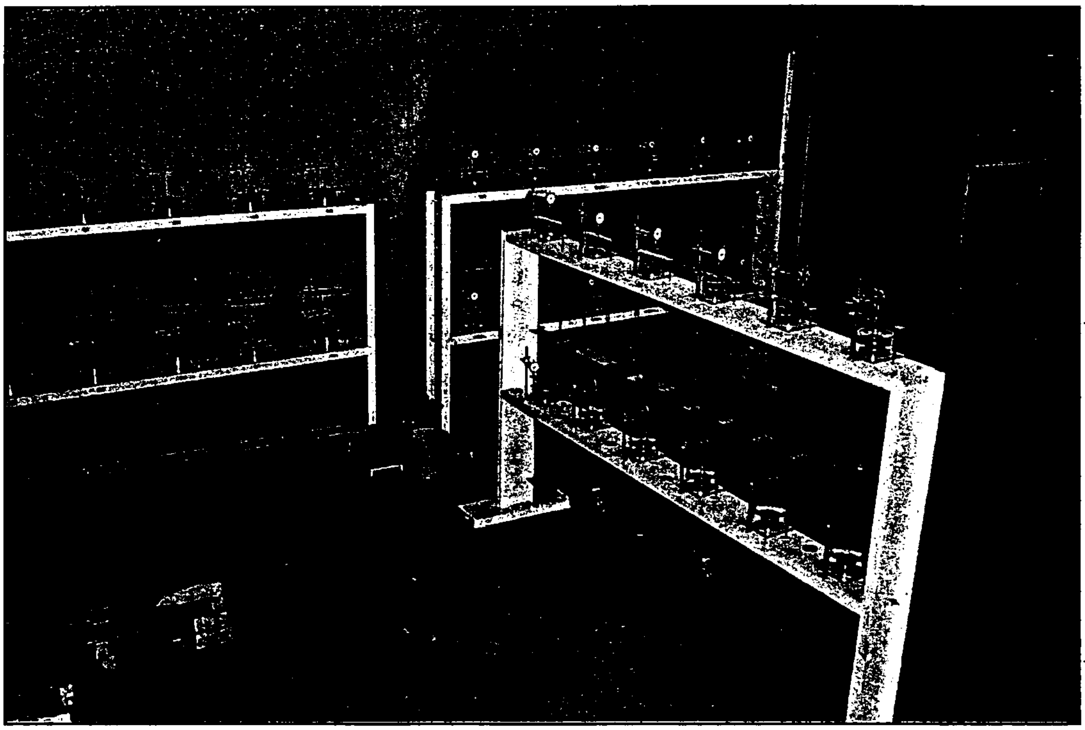
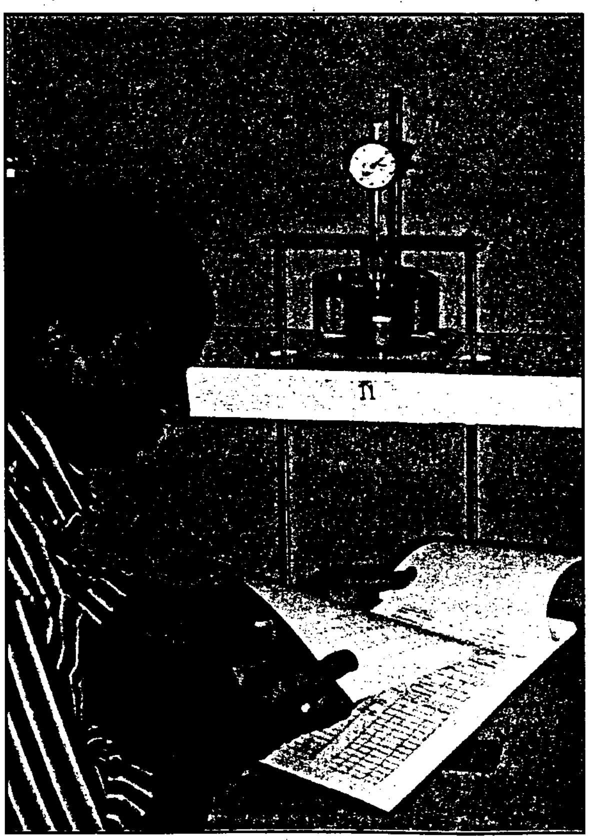
ASTM D 4546: Method B



Swell devices that measure the percent heave/settlement for vertical pressure equivalent to in situ vertical overburden.



Close-up of swell device.

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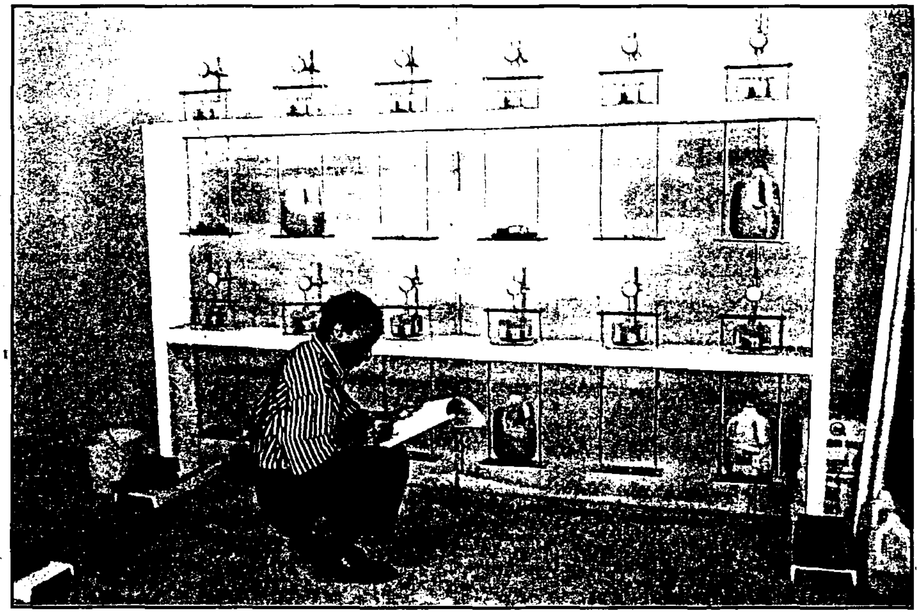
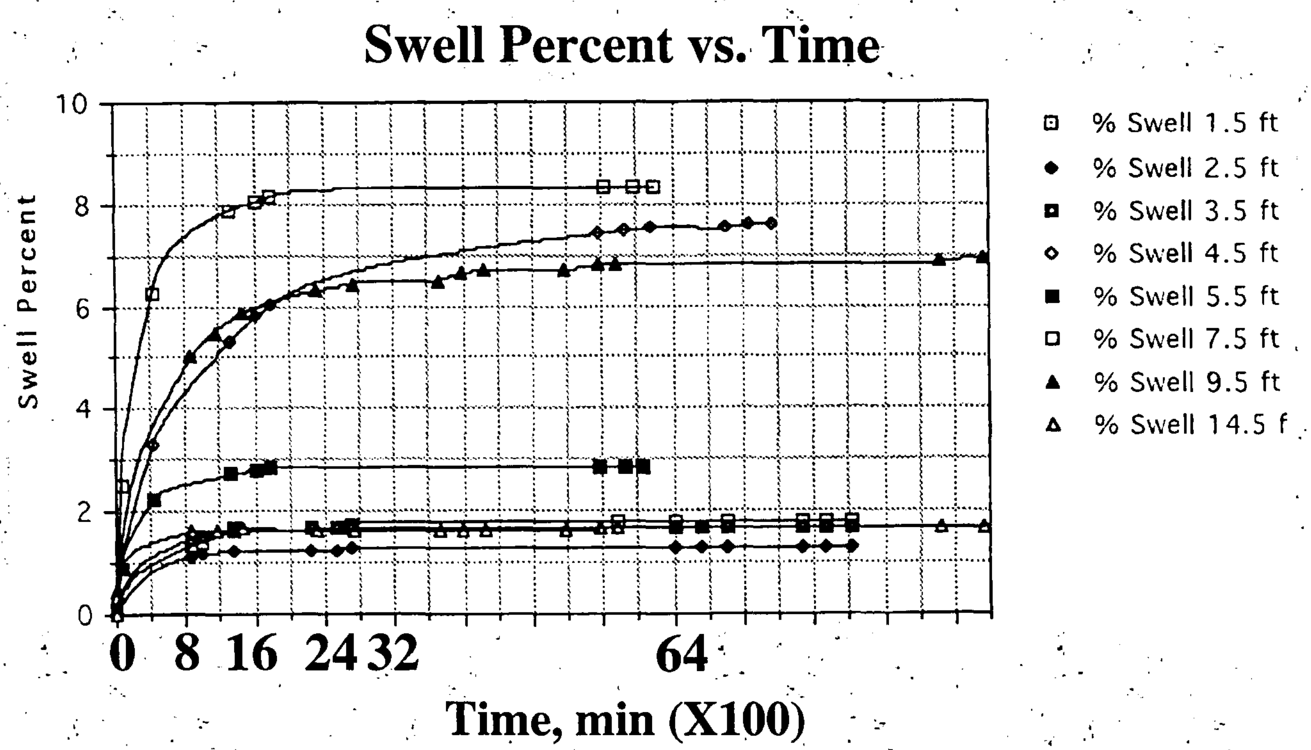
ASTM D 4546: Method B



After initial deformation, readings are usually taken at 0.1, 0.2,0.5, 1, 2, 3, 8, 15 and 30 min and 1, 2, 4, 8, 24, 48 and 72 hrs (or until constant swell).



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GEOPHYSICAL EXPLORATIONS

•Two-dimensional resistivity profiling utilizing a patent pending process called Geoelectrical Moisture/Material Resistivity (*GMMIR*). US Patent Application No. S/N 09/071,577. All rights reserved.

•With an licensed agreement with Lyric Technologies, Bryant Consultants, Inc has the capabilities of performing ground penetrating radar tests and analyzing raw data with a patent process called System Identification and Analysis of Subsurface Radar Signals (*SIDARS*). US Patent No. 5,384,715. All rights reserved.

•COMING SOON - Three-Dimensional Resistivity Profiling!

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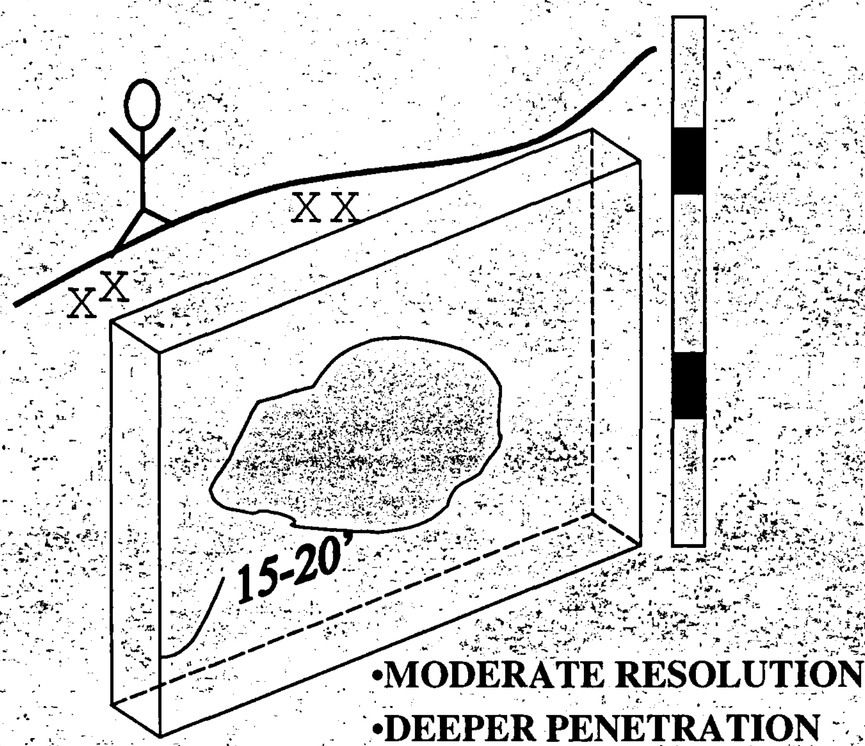
Electrical Resistivity and the GMMIR Process



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What Is Electrical Resistivity ?

- Electrical resistivity is the **resistance** for electrical current **to flow** through various materials

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Water conducts Electricity Readily.....

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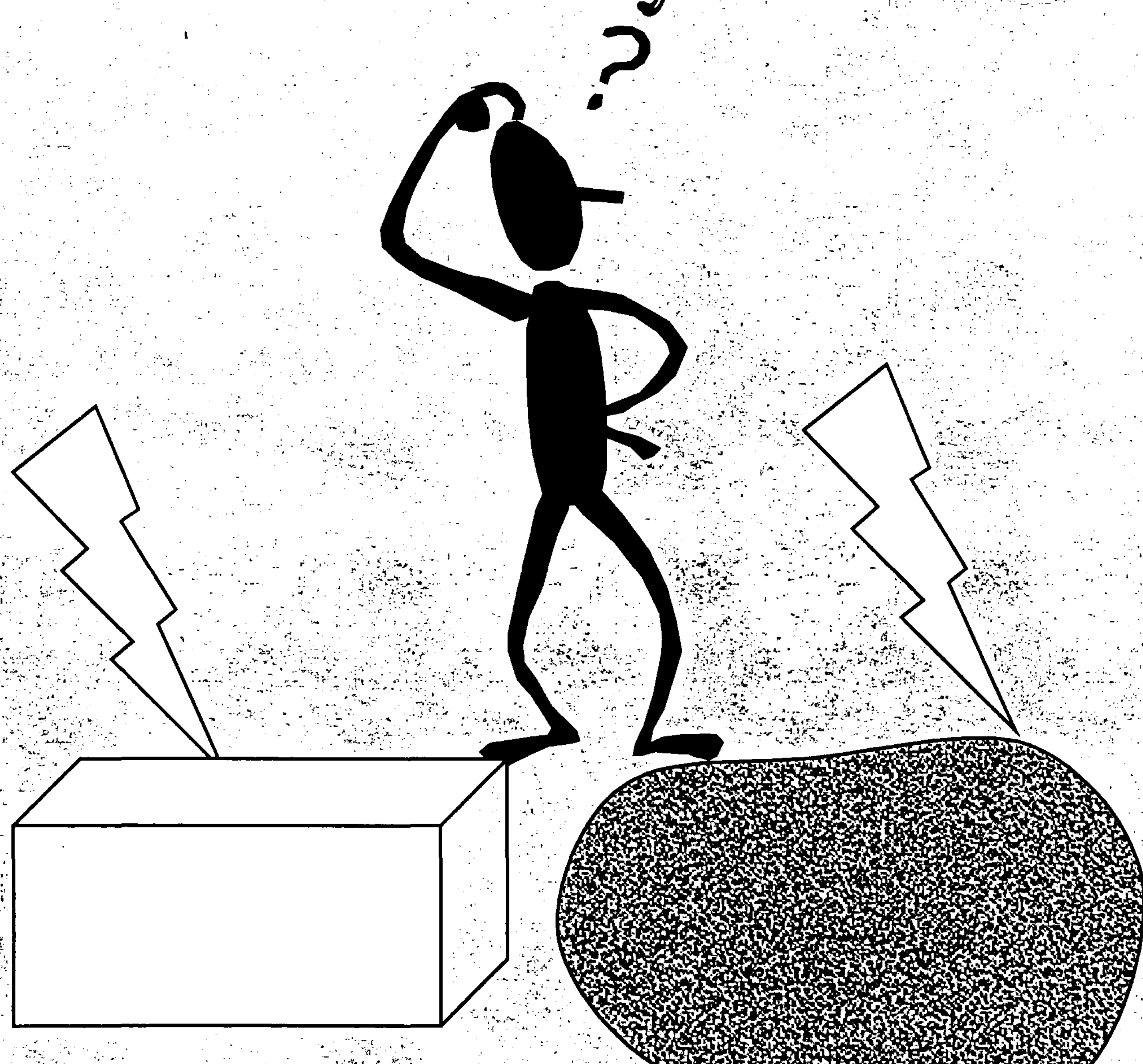
Concrete or Rock does not conduct electricity readily.......



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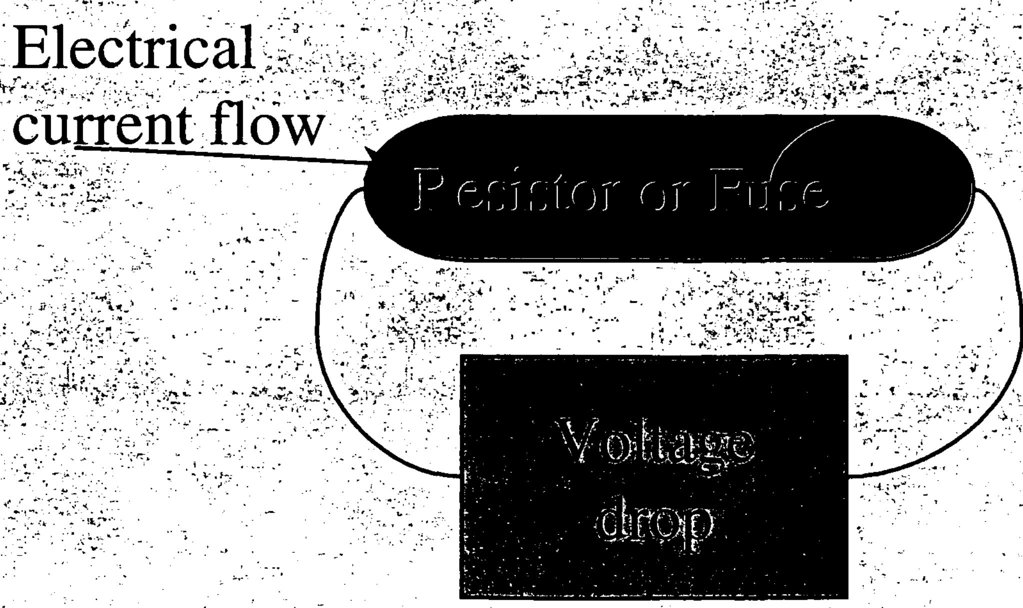
Units of Electrical Resistivity Measurement

OHM-METER

1 OHM = 1 volt/1 amp



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What is GMMIR Profiling

- Geo-electrical
- Moisture
- Material
- Imaging
- Resistivity

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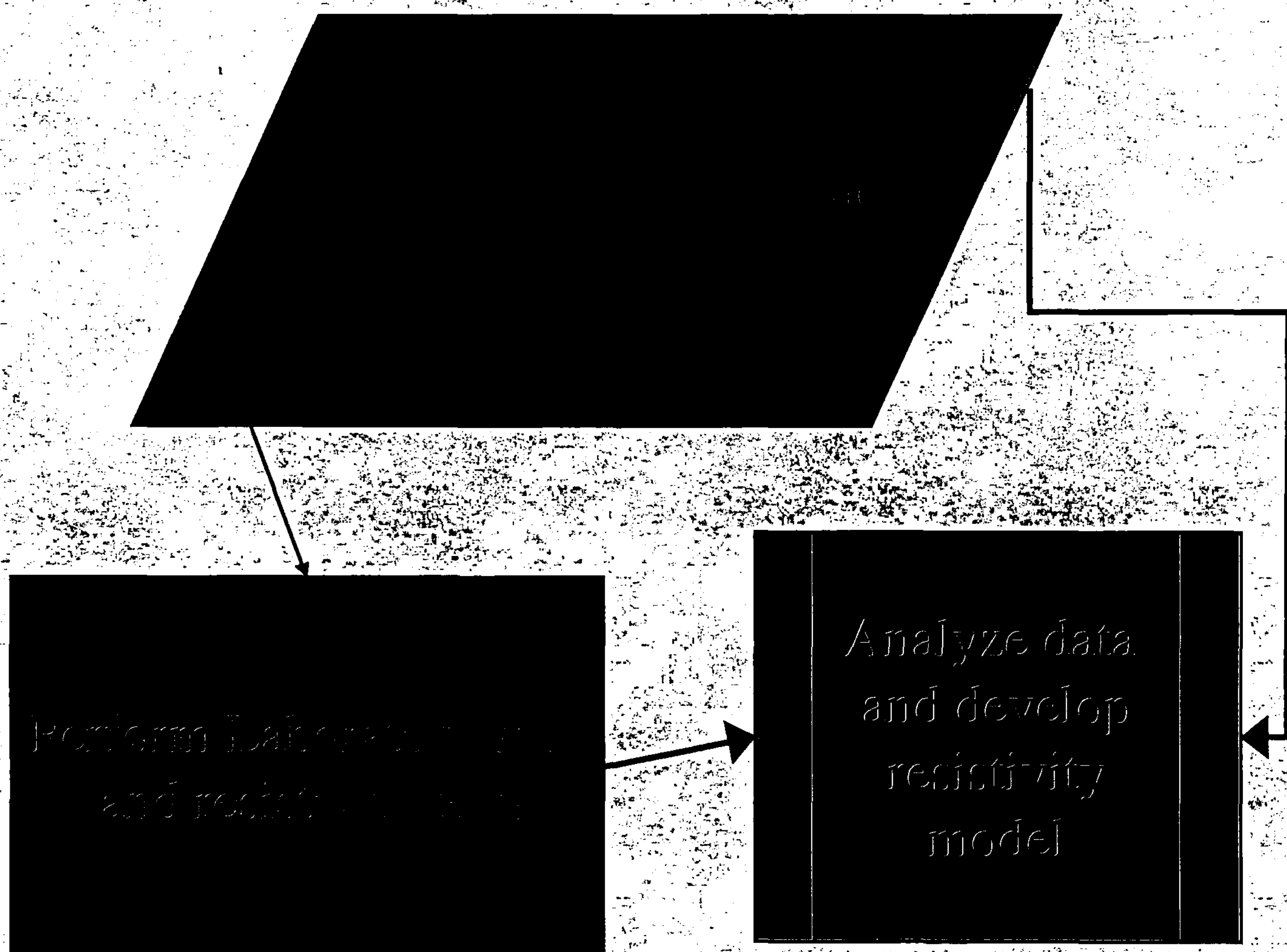
GMMIR PROCESS



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GMMIR USES AND APPLICATIONS

- Evaluating leaks occurring beneath buildings
- Evaluating slope stability along embankments
- Verification of water injection for pre-swelling of expansive clays
- Evaluating the effects of vegetation beneath buildings and pavement structures
- Location of buried objects such as steel tanks and pipes

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GMMIR ADVANTAGES

- ALLOWS BETTER DELINATION OF LEAK INFLUENCE AREA
- ALLOWS MORE QUANTFIABLE IDENTIFICATION OF ENVIRONMENTAL EFFECTS
- HELPS TO PROVIDE MORE COMPLETE CLAIM EVALUATION

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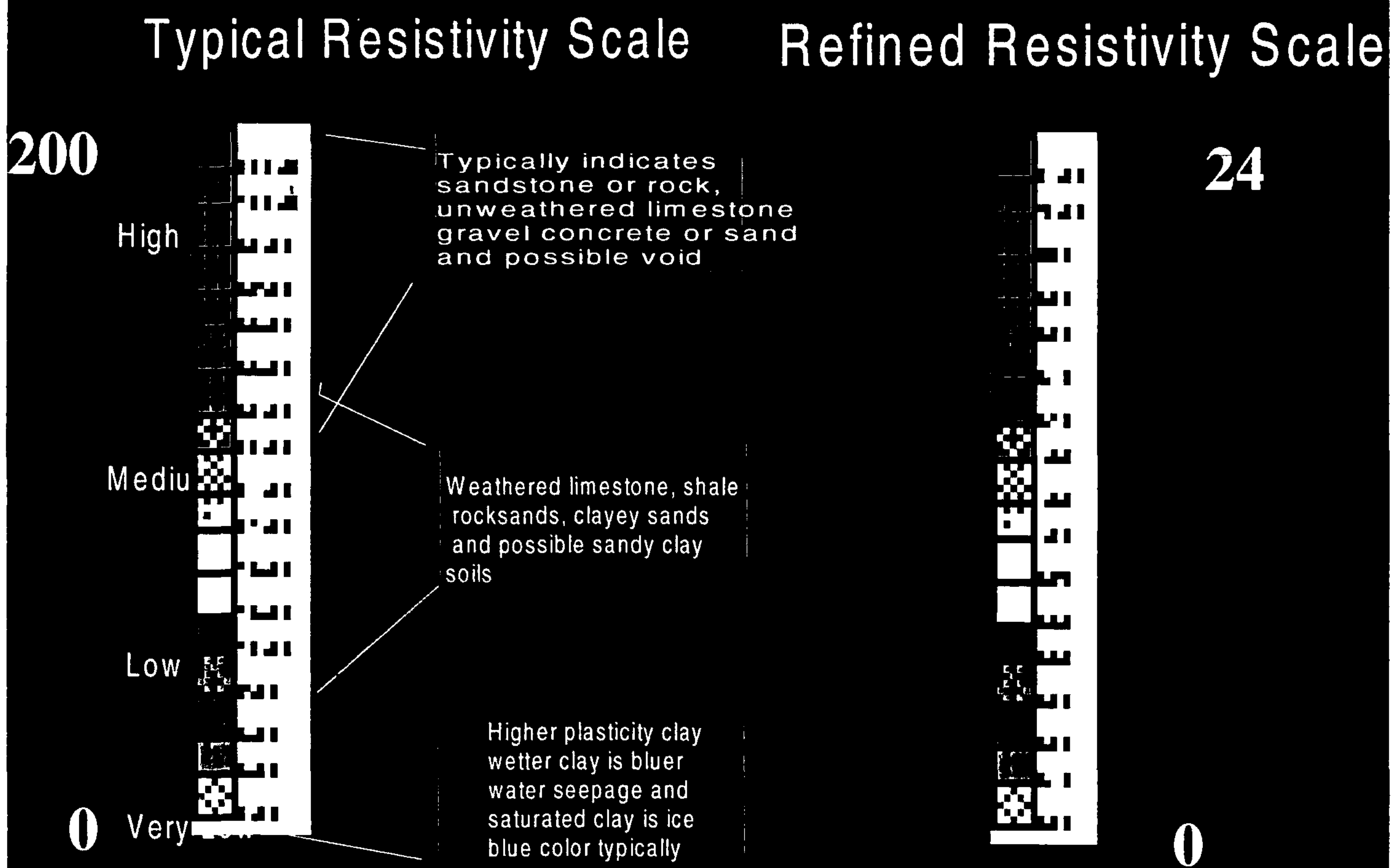
Electrical Resistivity Scales



Resistivity Values measured in Ohm-meters

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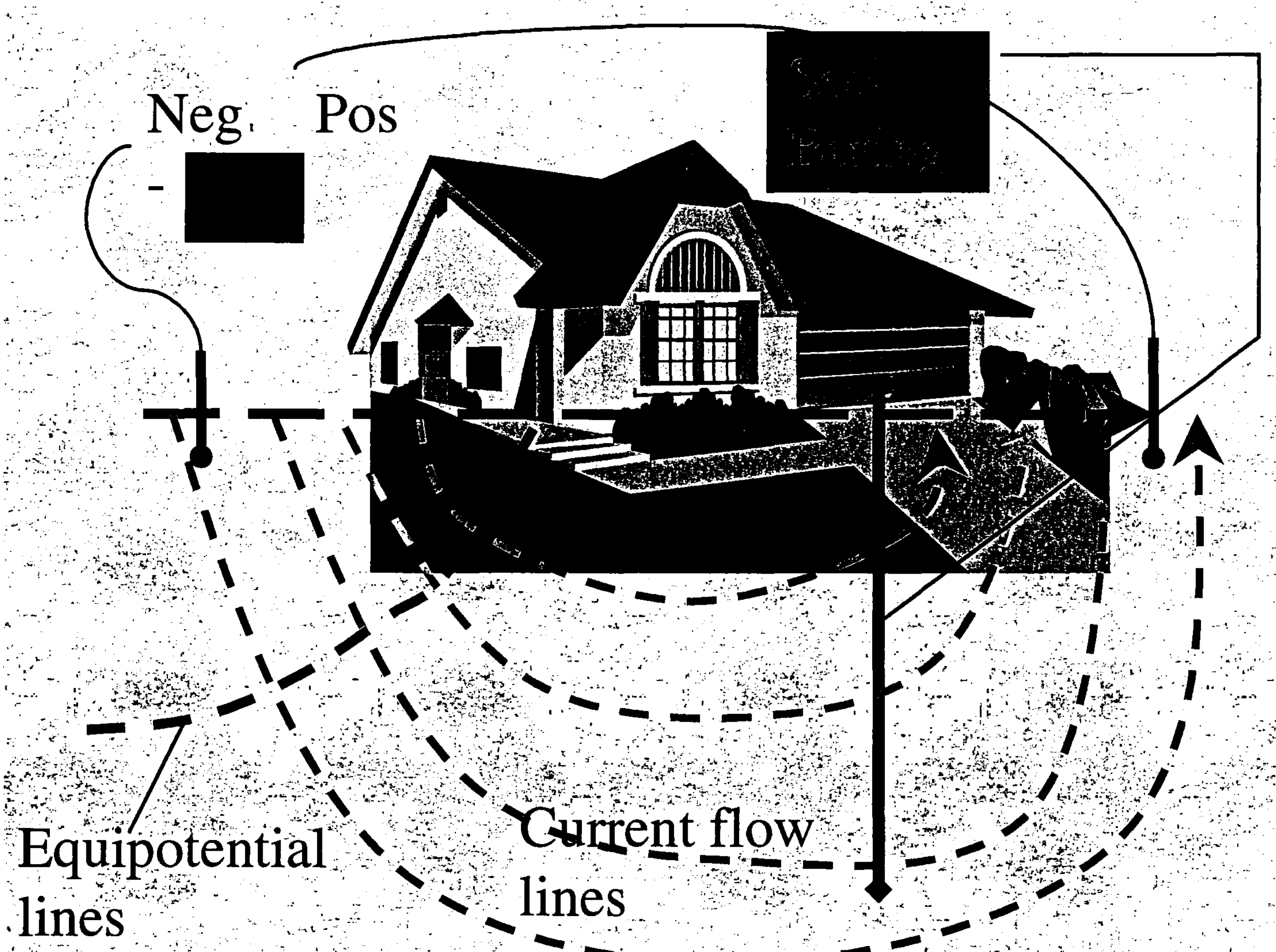
GMMIR Process
Collecting Field Data and Information



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No Anomaly
Existing at Depth

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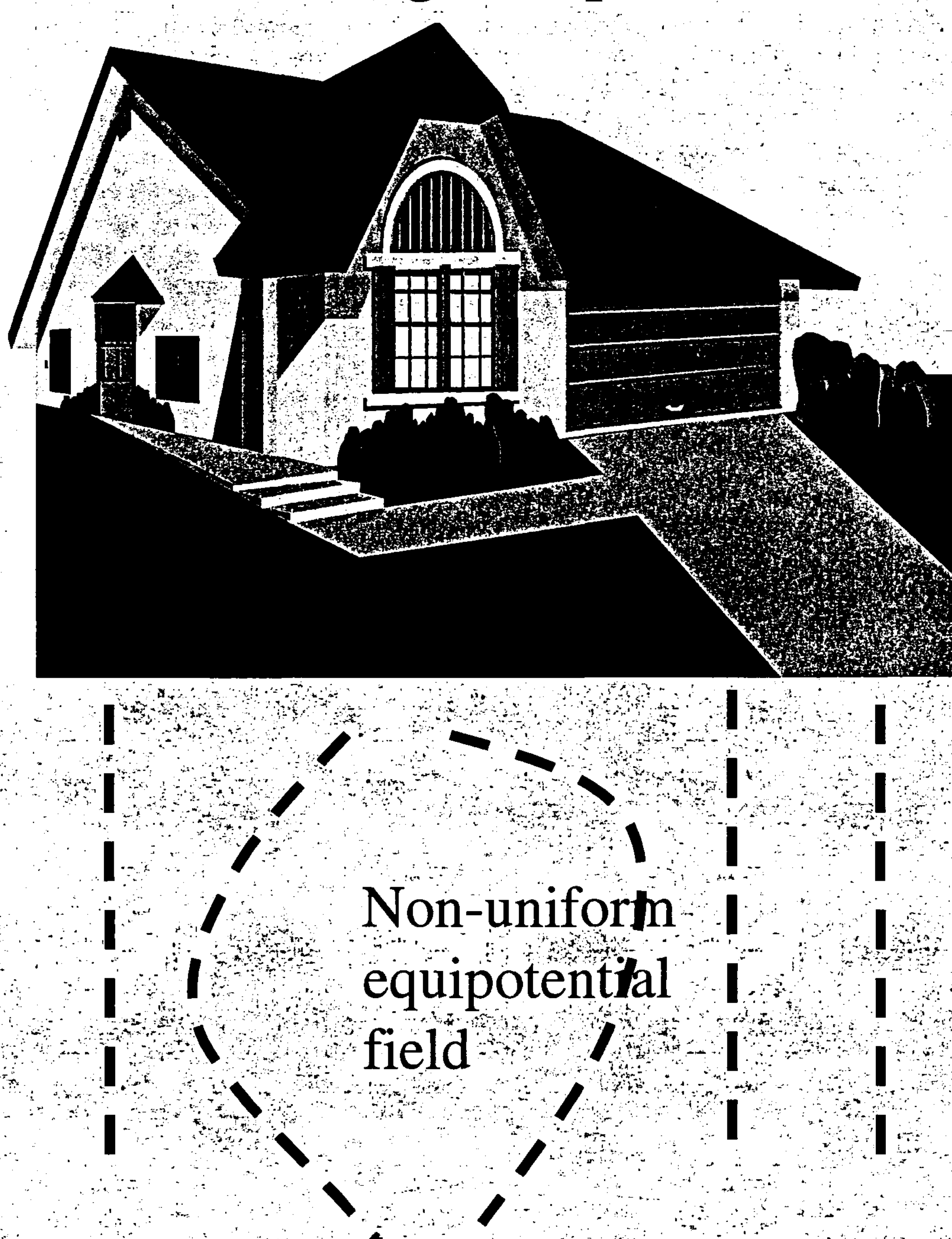
Anomaly
Existing at Depth



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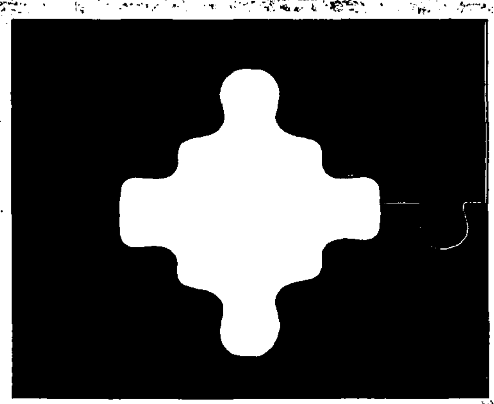
Understanding House and Structure Performance Requires solving a puzzle:

- Understanding Soil Structure Interaction
- Understanding the Subsurface Conditions
- Evaluating the soil and rock properties accurately



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Primary Objective of Plumbing Leak Assessment is an:

ACCURATE EVALUATION OF THE CLAIM

This involves:

- Where did the plumbing leak affect the structure?
- Where did the plumbing leak not affect the structure?
- How can you support the effects of environmental causes other than leaks?

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EVALUATION OF LEAK INFLUENCE

- •Soil-Structure Interaction Problem
- •Must Explain Site Conditions Clearly
- •2D and 3D GMMIR and other Geophysical Methods (SIDARS) Coupled with Accurate Soil Testing is Answer to Establish Causation of Movement and Distress

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Water Line Leak Examples

- Utility line leaks often occur around and beneath residential and commercial structures

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EXAMPLE A
Relatively Severe Sanitary Sewer Line Leak with Other Water Influences

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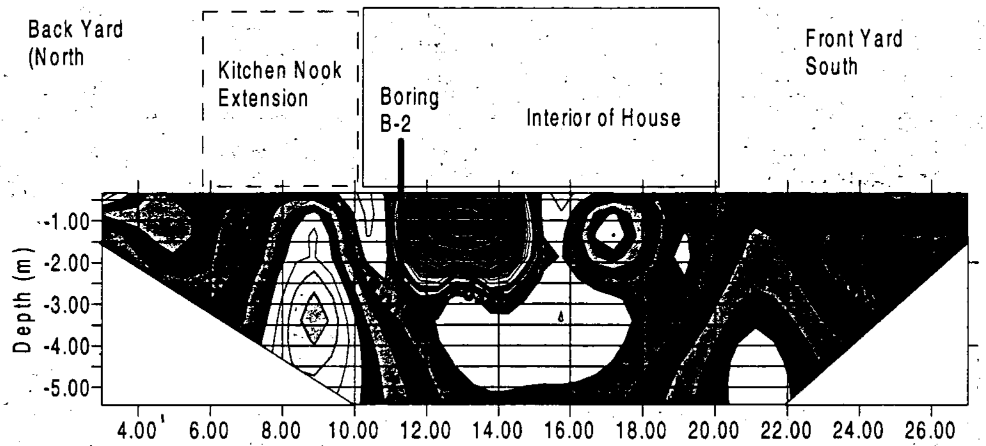
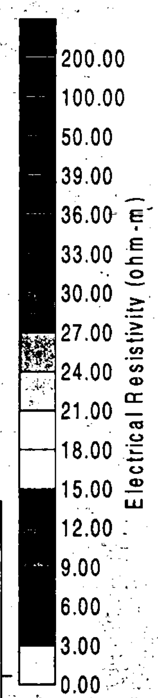
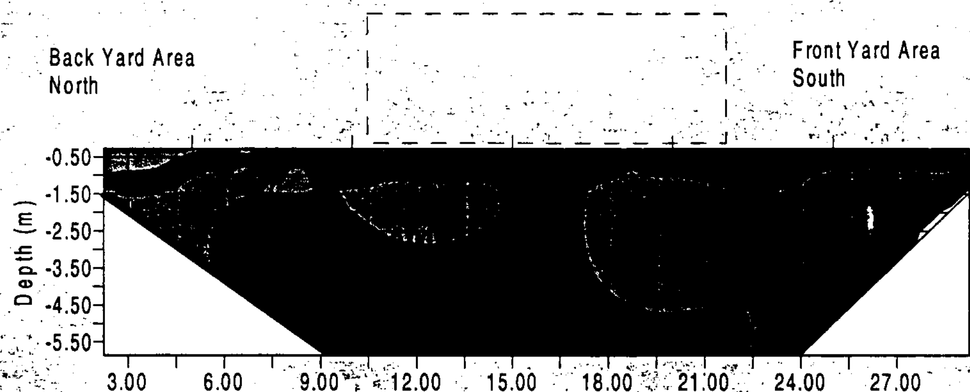
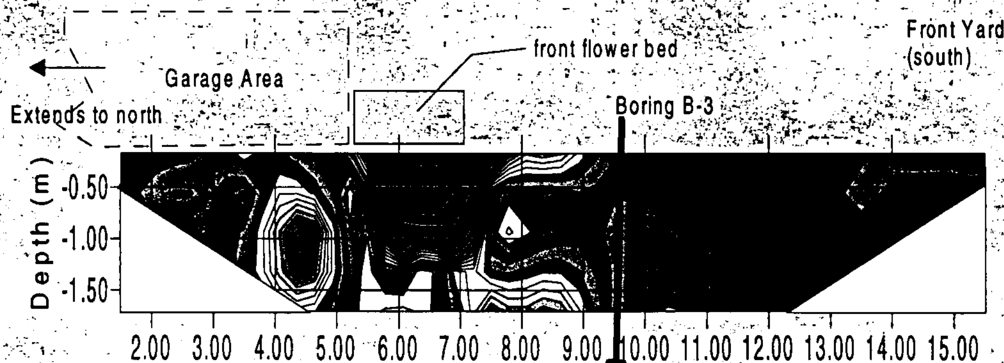


Profile R1. GMMIR Profile Taken from north to south through central portion of structure.





Profile R4. GMMIR profile taken on the exterior east side of the structure.
Looking towards the east.



Profile R3. GMMIR Profile on the west side of structure looking east.

Notes: 1. Data at lower corners is interpolated.
2. Structure, Boring and Vegetation positions are approximate.
3. Patent Pending Process, All Rights Reserved. US Patent Application S/N 09/071,577.

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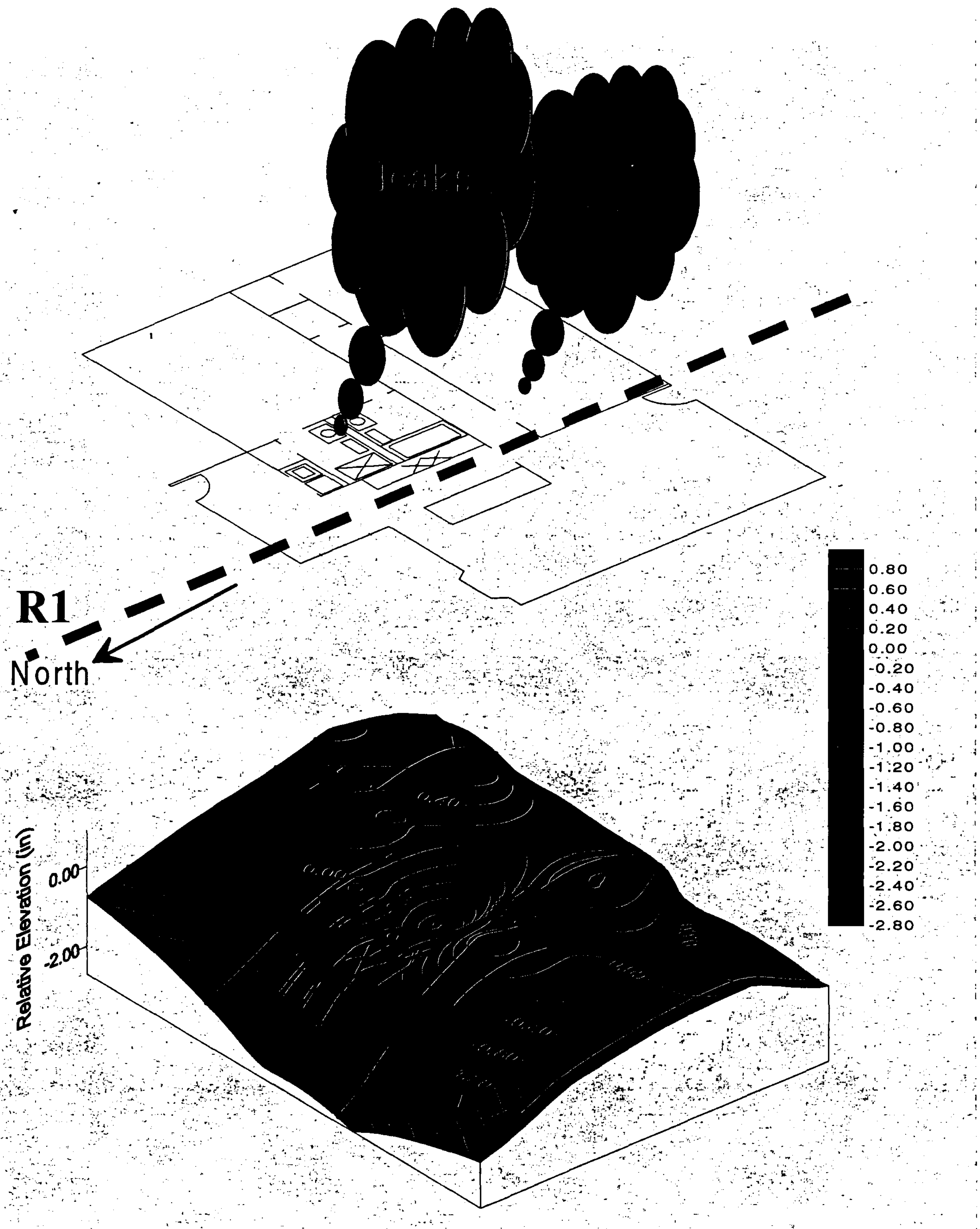

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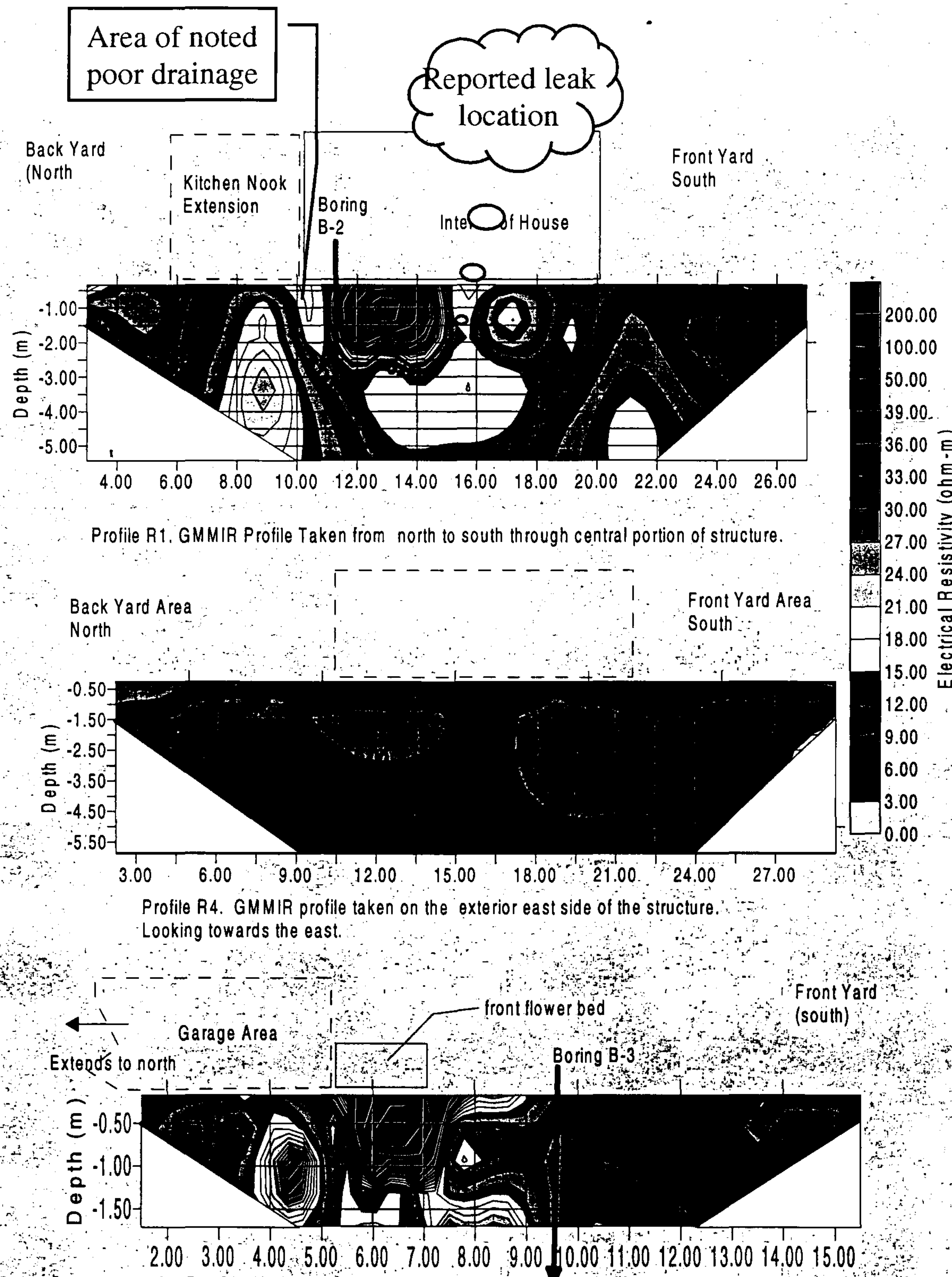

Profile R1. GMMIR Profile Taken from north to south through central portion of structure.

Profile R4. GMMIR profile taken on the exterior east side of the structure. Looking towards the east.

Profile R3. GMMIR Profile on the west side of structure looking east.

Notes: 1. Data at lower corners is Interpolated.
2. Structure, Boring and Vegetation positions are approximate.
3. Patent Pending Process, All Rights Reserved. US Patent Application S/N 09/071,577.

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EXAMPLE B
Relatively Small Sanitary Sewer Line Leak with Other Vegetation Influences

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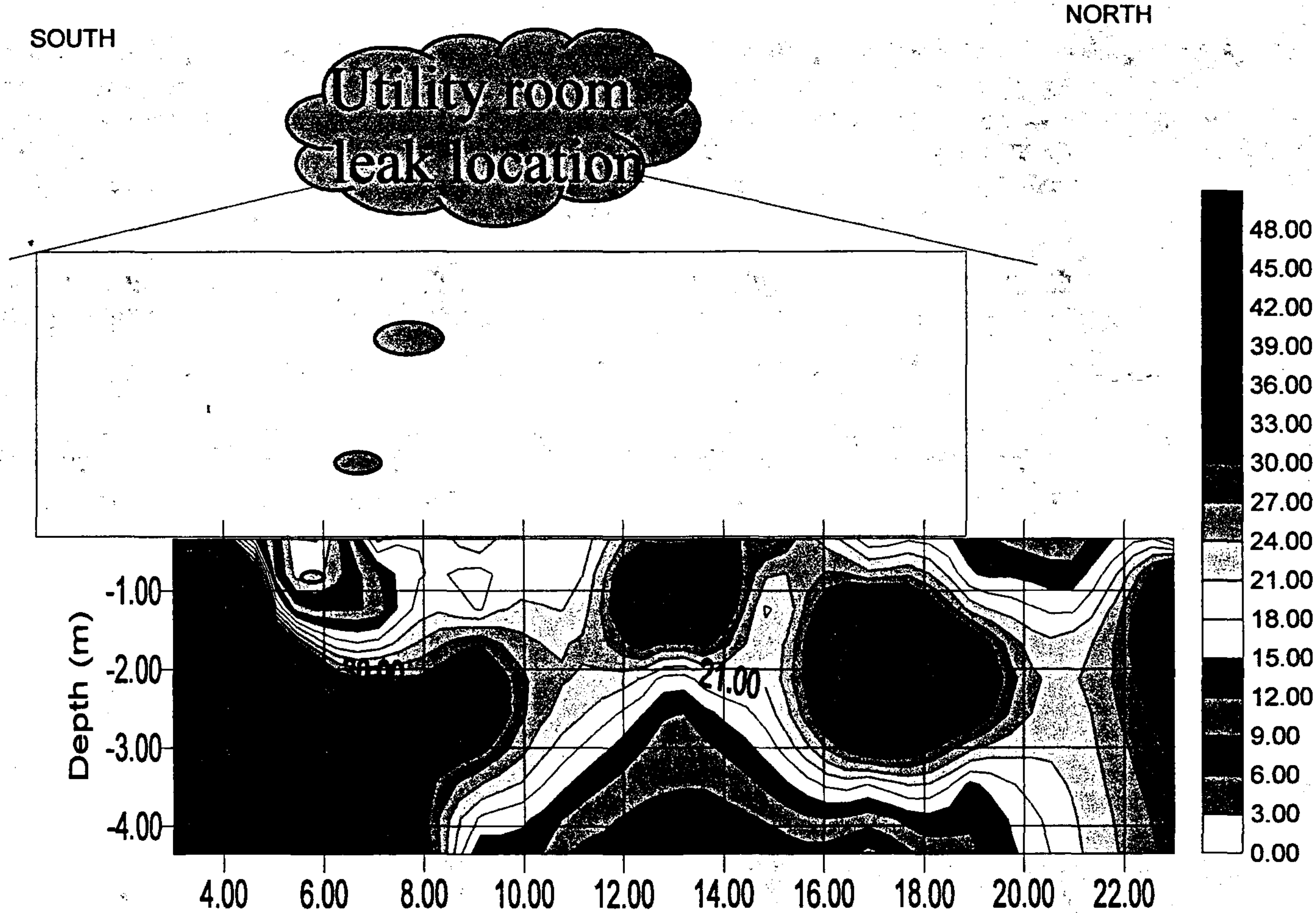
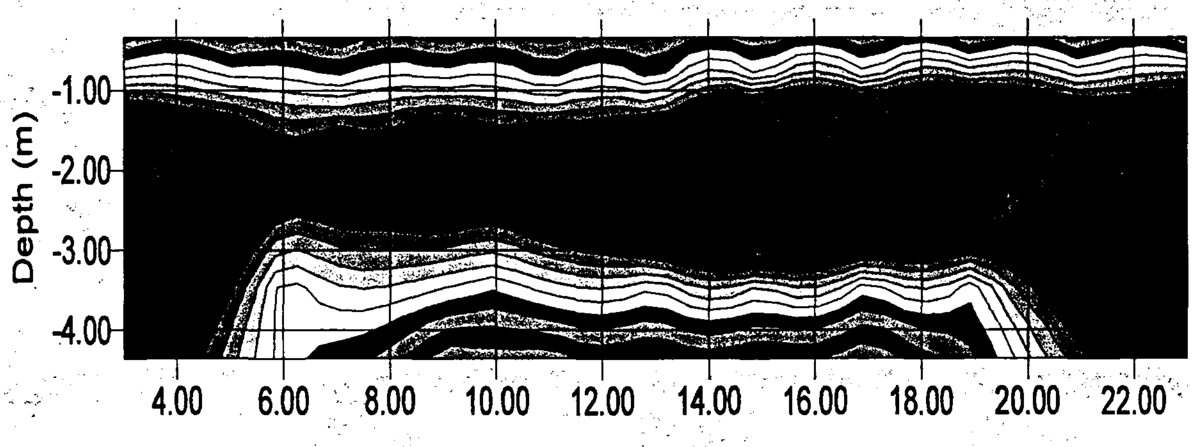


59

Profile P1 Through Garage and House



Profile P2 on Exterior West Side of Structure

Notes:
1. Data at lower corners is interpolated.
2. Structure, Boring and Vegetation positions are approximate.
3. Patent Pending Process, All Rights Reserved.
US Patent Application S/N 09/071,577.

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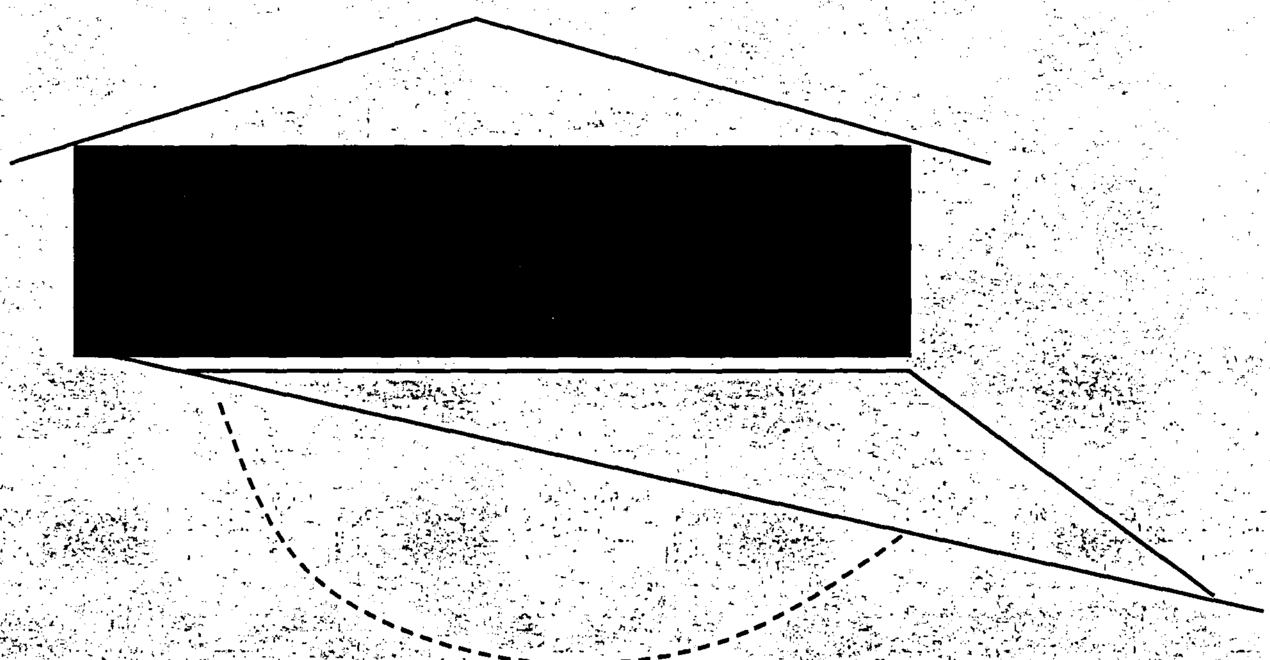
Slope Stability Example



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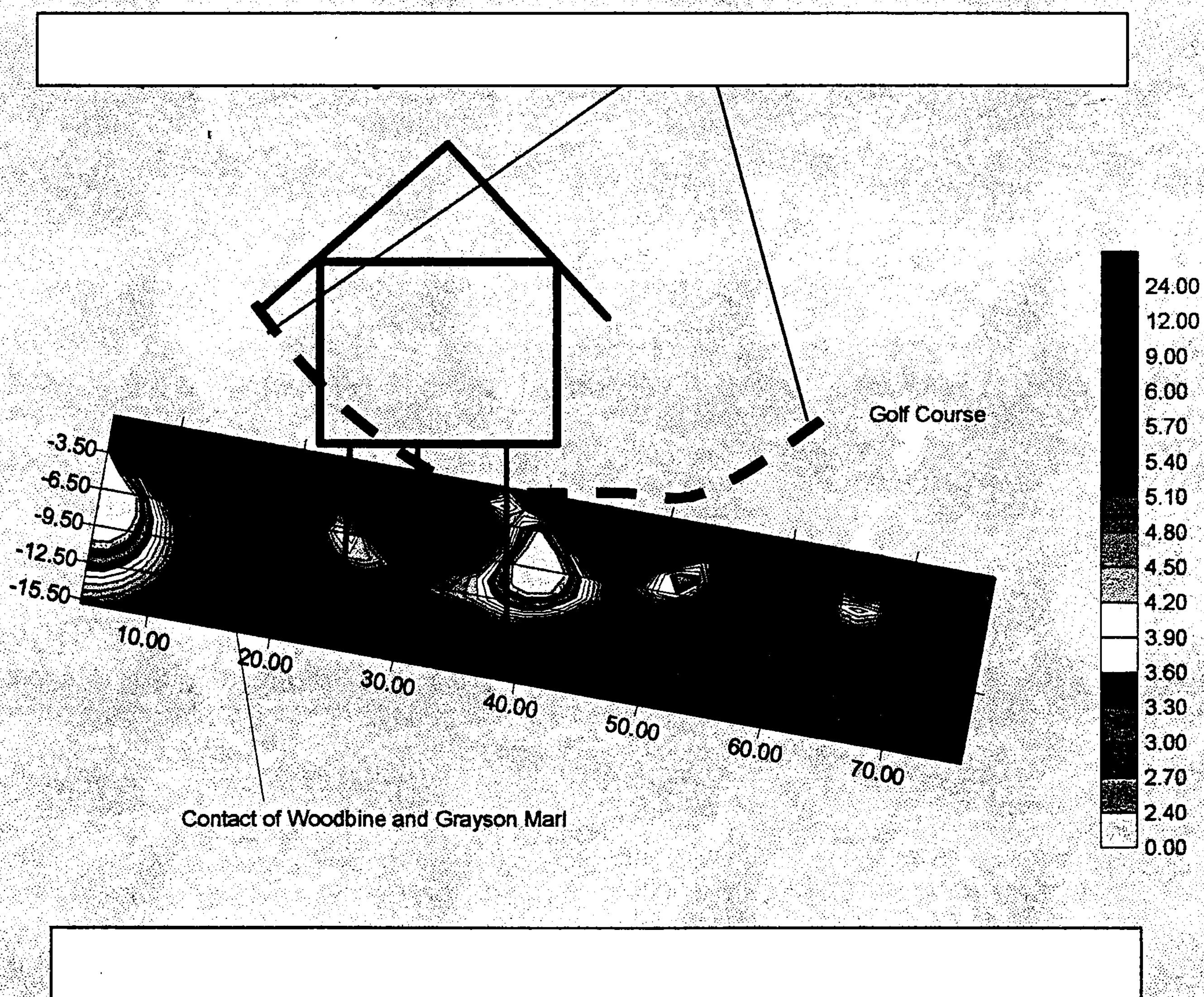
SLOPE FAILURE
NOT A PLUMBING LEAK



Notes: 1. Data at lower corners is interpolated.
2. Structure, Boring and Vegetation positions are approximate.
3. Patent Pending Process, All Rights Reserved.
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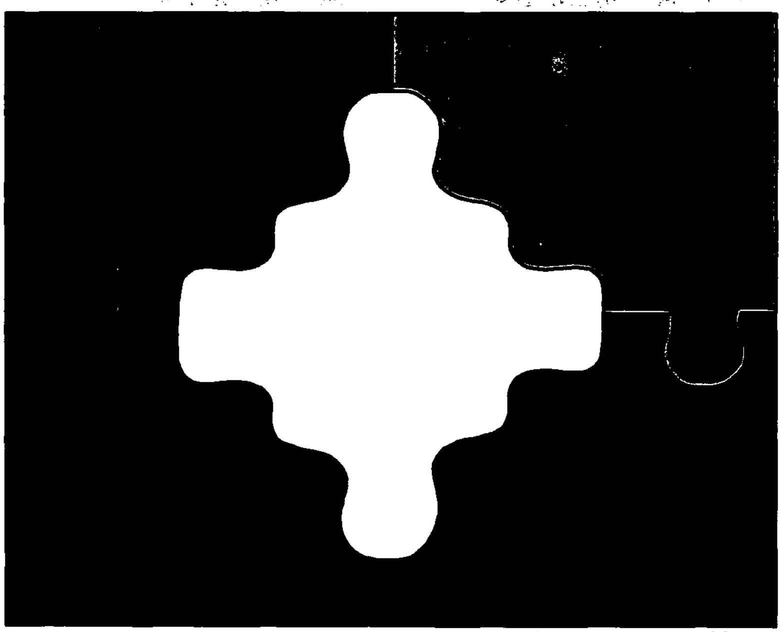
Environmental Effects of Creek and Tree for estimating depth of Root Barrier/Drain System



."... a puzzle wrapped in a riddle inside of an enigma."

Sir Winston Churchill

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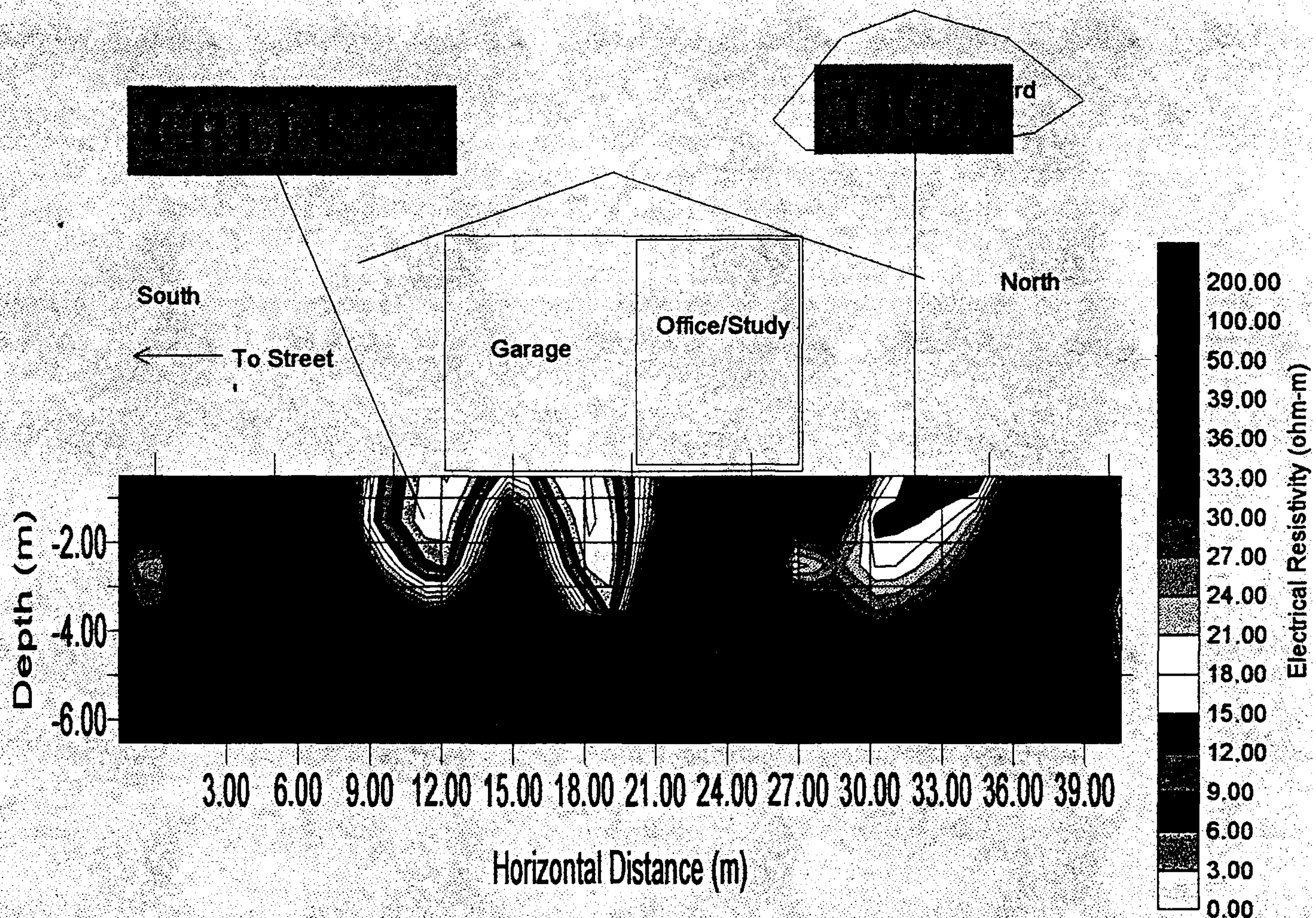

Notes: 1. Data at lower corners is interpolated.
2. Structure, Boring and Vegetation positions are approximate.
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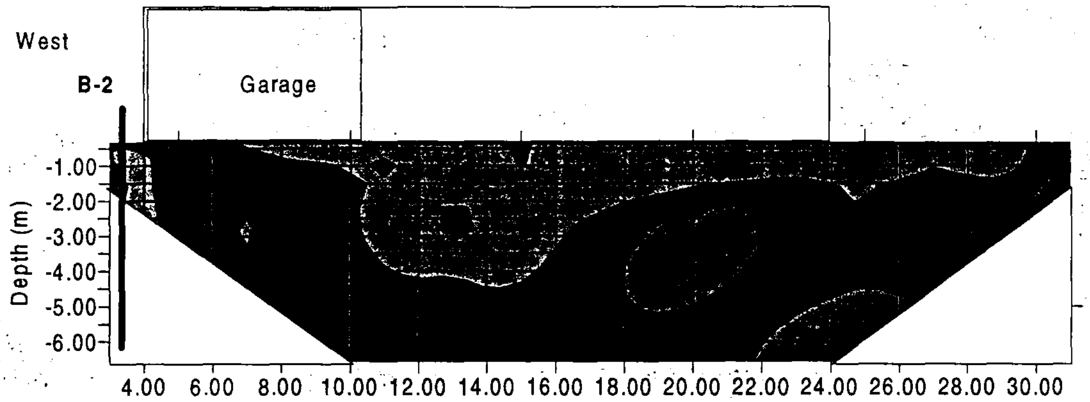
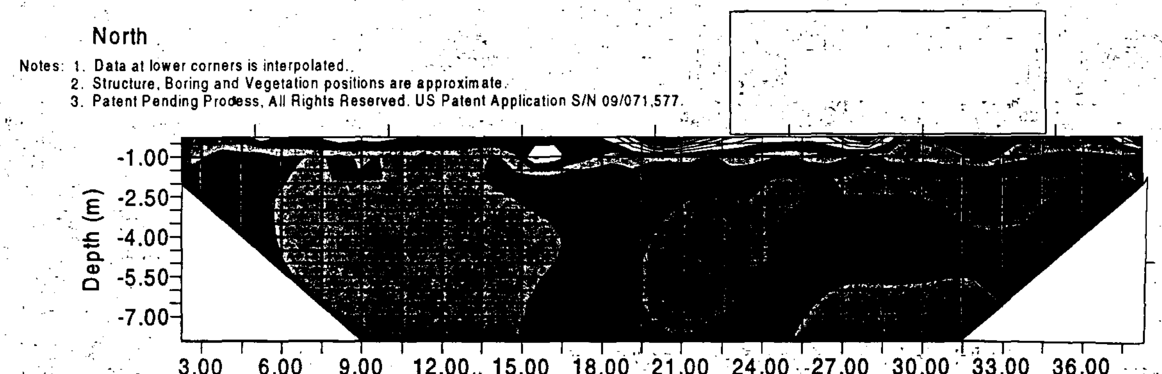
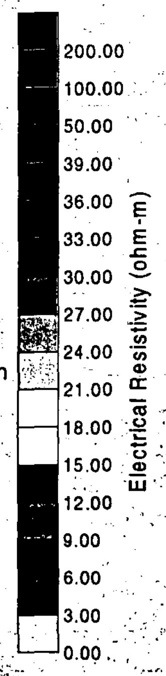
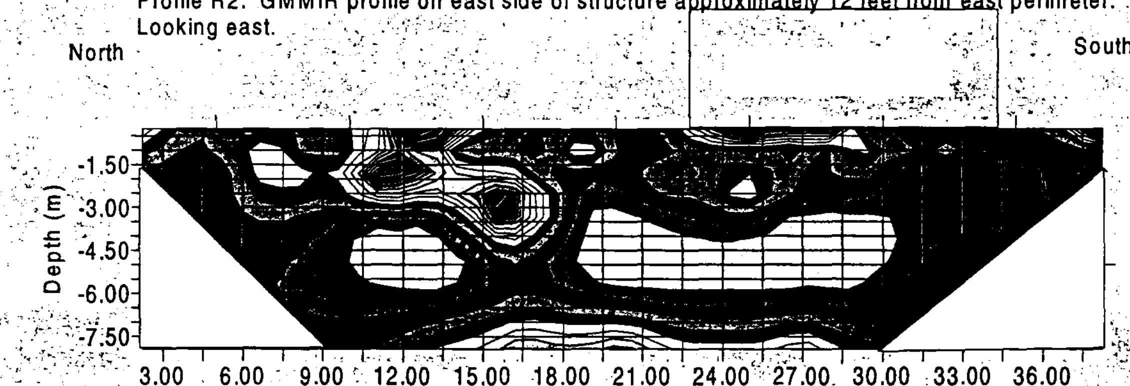


Profile R1. GMMIR profile on north side of structure looking north.

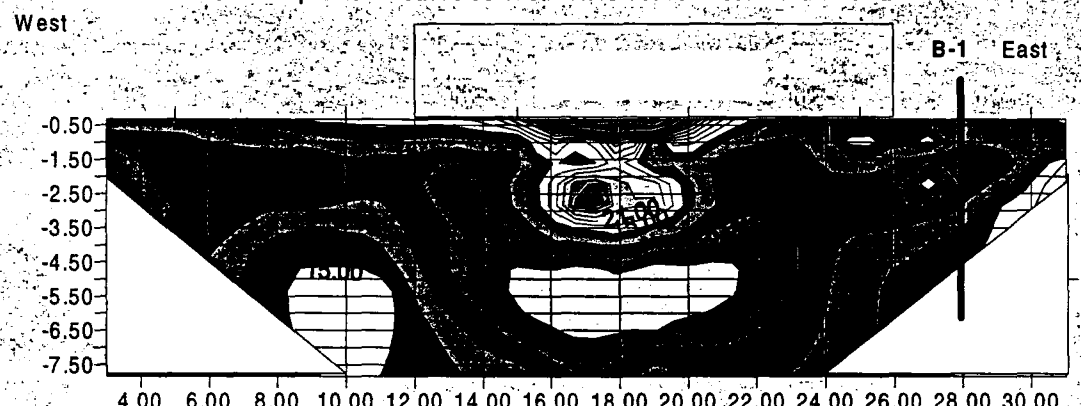


Profile R2. GMMIR profile on east side of structure approximately 12 feet from east perimeter. Looking east.





Profile R3. GMMIR profile on east side of structure between curb and sidewalk.



Profile R5. GMMIR profile on south side of structure looking to the north.

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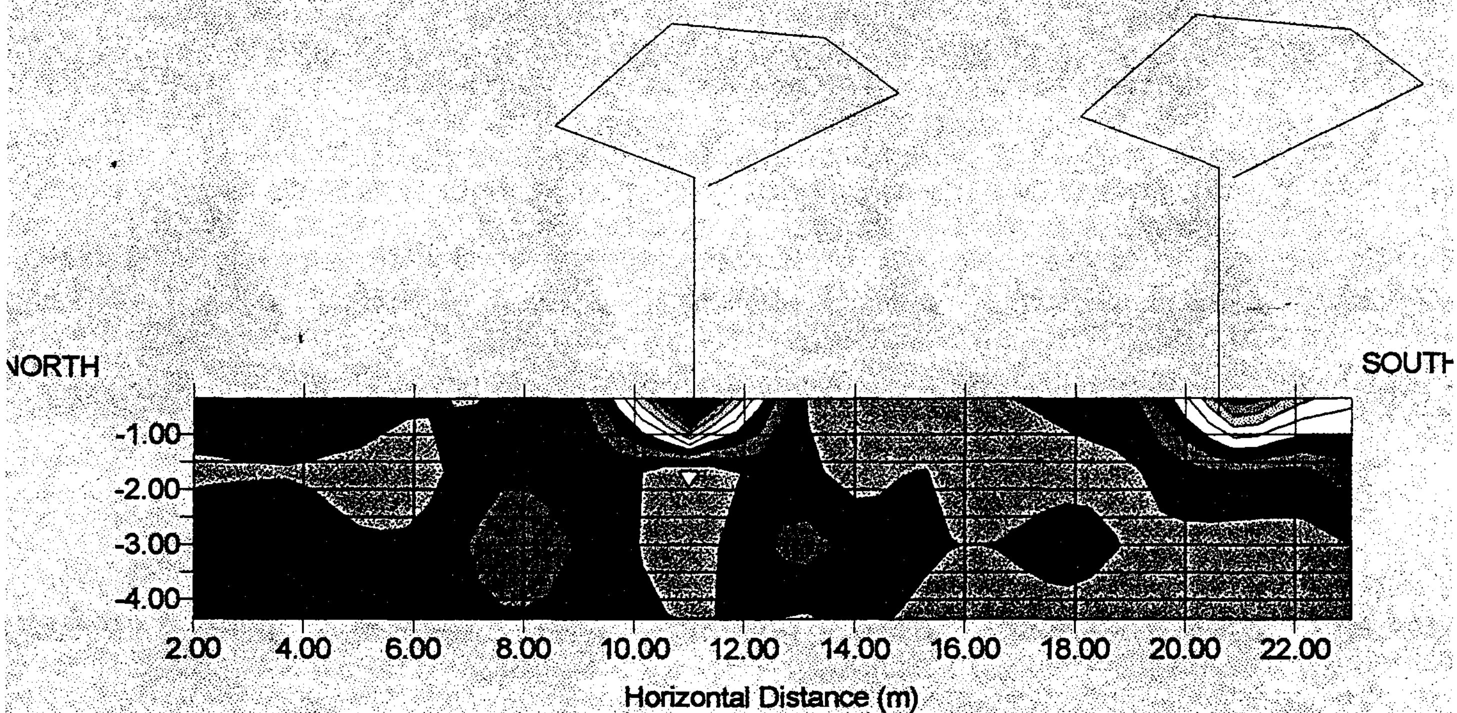

Notes: 1. Data at lower corners is interpolated.
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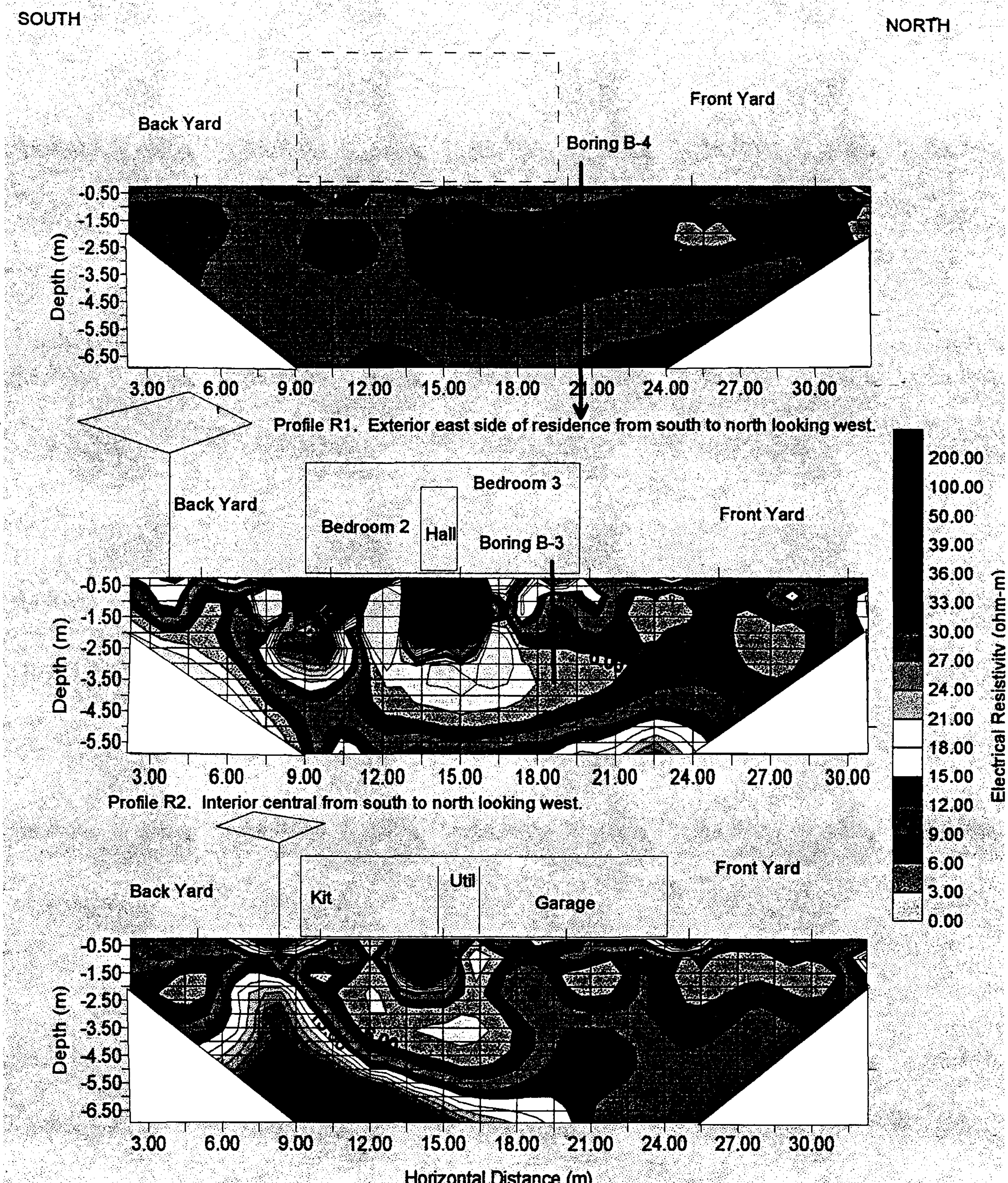

Profile R1. Exterior east side of residence from south to north looking west.

Profile R2. Interior central from south to north looking west.

Profile R3. Interior west side of residence north to south looking west.

Notes: 1. Data at lower corners is interpolated.
2. Stucture, Vegetation and Boring locations are approximate.
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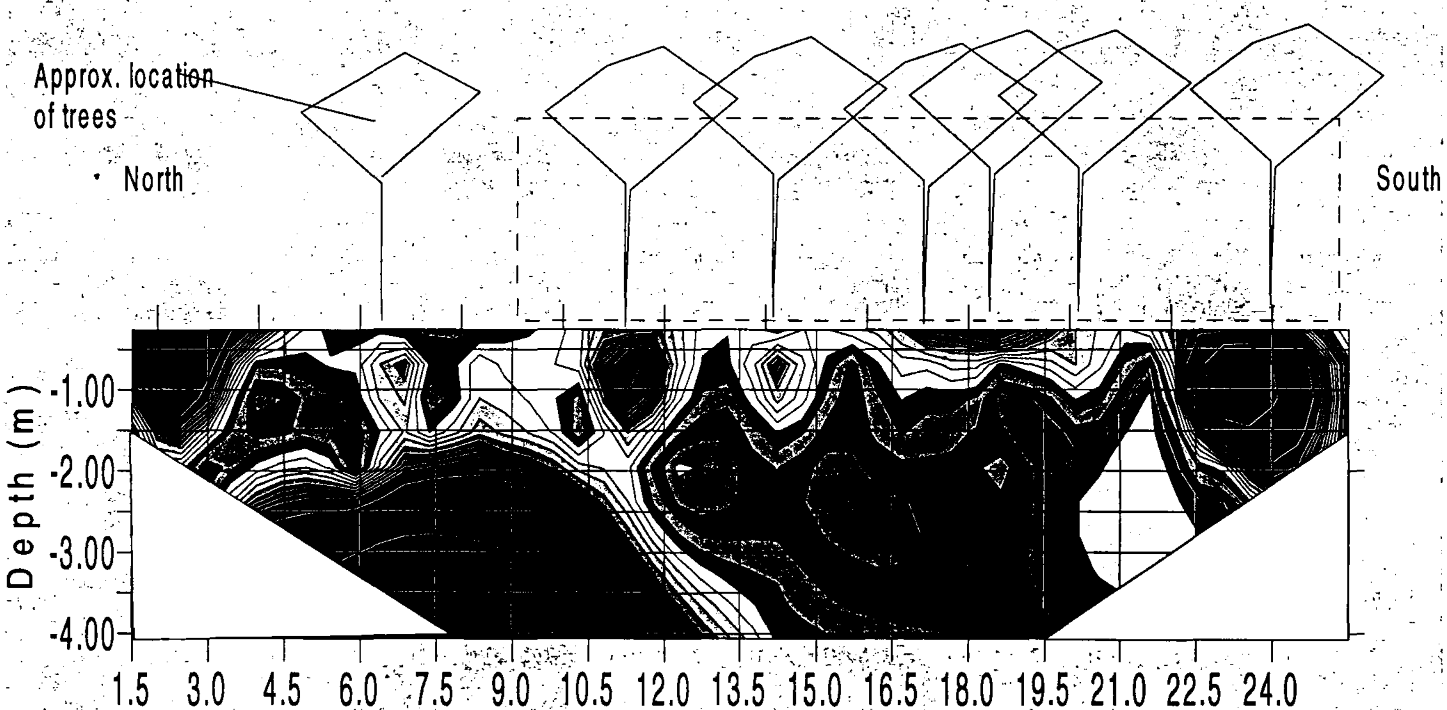

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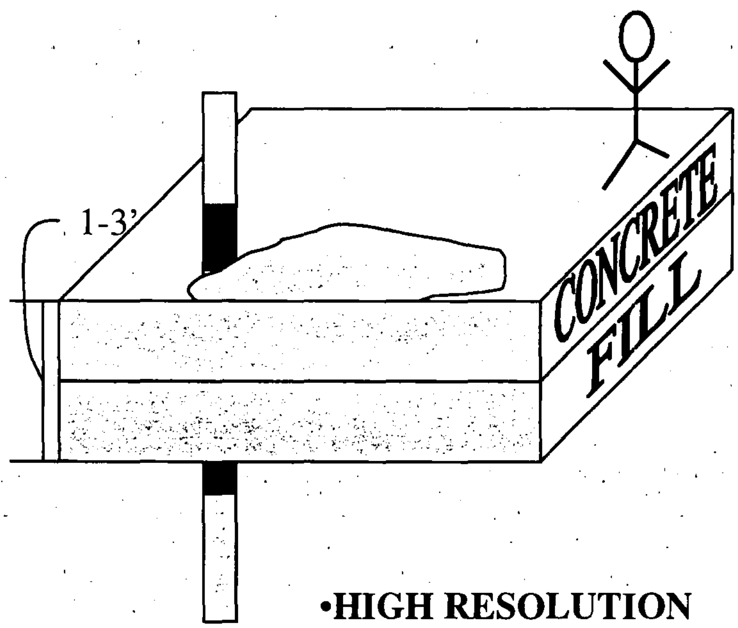
GROUND PENETRATING RADAR and the SIDAR PROCESS



•HIGH RESOLUTION
•LIMITED DEPTH

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What Is Ground Penetrating Radar ?

- GPR is a short pulse of **electromagnetic energy** which is radiated into the subsurface. When this pulse strikes an interface between layers of materials with different electrical properties, part of the wave reflects back, and the remaining energy continues to next interface.

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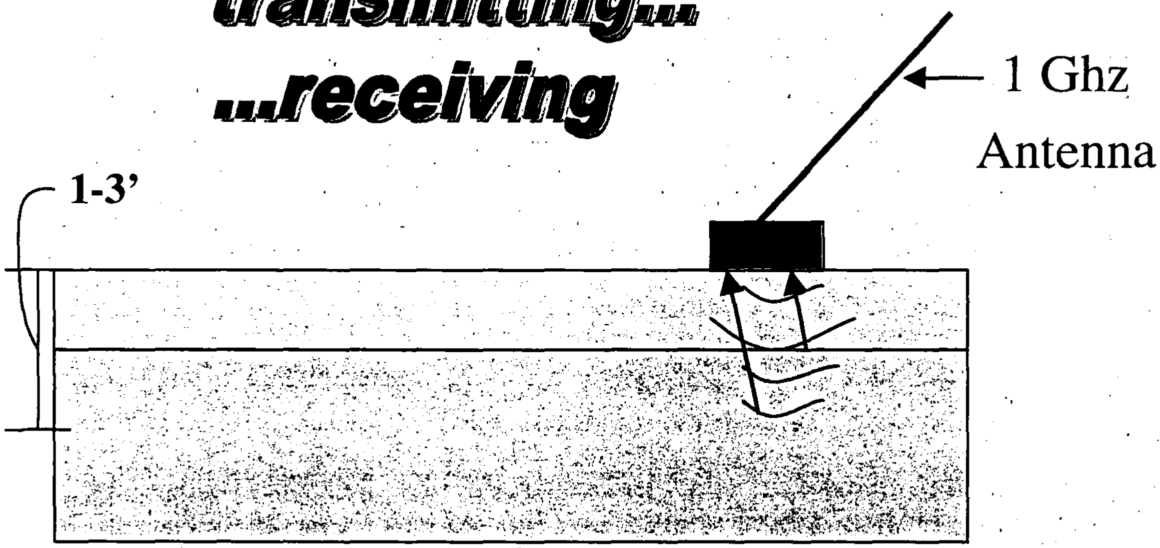
GROUND PENETRATING RADAR

transmitting...

...receiving



•HIGH RESOLUTION
•LIMITED DEPTH

Actual Scan = 200 reflection/sec!

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What is SIDARS?

- System
- IDentification
- Analysis of
- Radar
- Signals

SIDARS calculates and numerically quantifies the densities, liquid content and voids within each layer of the subsurface system

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SIDARS USES AND APPLICATIONS

- Evaluating the severity of leaks occurring beneath buildings
- Evaluating slope stability along embankments
- Determining thickness, density, liquid content, air voids and porosity of one or more interfaces
- Location of buried objects such as steel tanks and pipes
- Providing a high resolution picture of shallow subsurface conditions

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EXAMPLE
of a Plumbing Leak
detected by SIDARS

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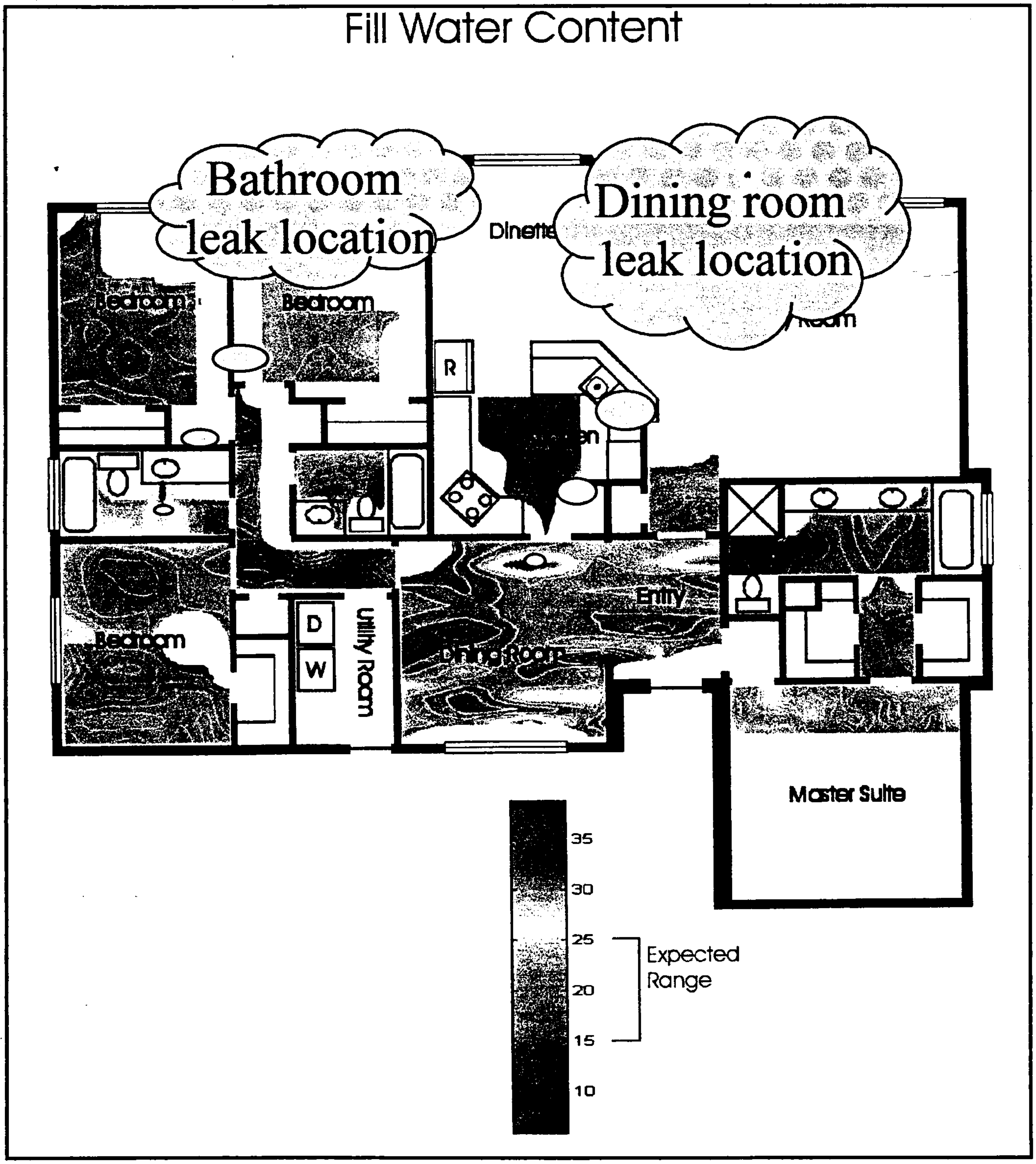
Leak Detection



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US Patent No. 5,384,715

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SUMMARY OF MAIN POINTS

- Soil-Structure Interaction Problem
- Must Explain Site Conditions Clearly
- 2D and 3D GMMIR and other Geophysical Methods (SIDARS) Coupled with Accurate Soil Testing is Answer to Establish Causation of Movement and Distress

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??QUESTIONS??

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EDUCATION

Ph.D. Civil Engineering, University of Texas, 1967
M.S. Civil Engineering, University of Texas, 1961
B.S. Civil Engineering, University of Texas, 1960

PATENTS

"Systems Identification and Analysis of Subsurface Radar Signals," U.S. Patent No. 5,384,715.

HONORS, AWARDS AND LISTINGS

Hamilton Watch Award, University of Texas, College of Engineering, 1960
Honorary Fellow, University of Texas, 1960-61
Graduate Fellow, National Science Foundation, 1960-61, 1965-67
John B. Hawley Award, Texas Section ASCE, 1966
Post-doctoral Fellow, National Science Foundation, 1969-70
Everite Bursary Award, Council for Scientific and Industrial Research, Pretoria, South Africa, 1984
Who's Who in Texas, 1986
Who's Who in the South and Southwest, 1988 and afterward
American Men and Women in Science, 1989 and afterward
Who's Who in America, 1993 and afterward
Who's Who in the World, 1994 and afterward
Fellow, American Society of Civil Engineers, 1992
Texas A&M University Association of Former Students Distinguished Achievement Award in Research, 1996

COURSES TAUGHT(T) AND DEVELOPED(D)

CVEN 365	Soil Mechanics (T)
CVEN 435	Foundation Engineering (T)
CVEN 616	Systems Design of Pavements (T,D)
CVEN 646	Foundations on Expansive Clays (T,D)
CVEN 647	Numerical Methods in Geotechnical Engineering (T,D)

LICENSES

Registered Professional Engineer, Texas #27657
Registered Professional Engineer, Louisiana #9620
Registered Land Surveyor, Louisiana #2434

PROFESSIONAL ACTIVITIES

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.

Papers/Handouts In Order

1. Soil Suction Measurements with Filter Paper Method.
2. Prediction of Movement in Expansive Clays. (R. Bulyt) Oct 1999
3. Foundations and Pavements on Unsaturated Soils (R. L. Lytton) Sept, 1995.
4. Volume Change and Flow Calculations in Expansive Soils (R. D. Ullrich and Assoc.) June, 1995 (Volflo Users Guide).
5. Soil Suction Conversion Factors (1 page).
6. The Prediction of Total Heave Using Soil Suction Profiles, Atterberg Limits, Hydrometer, and Filter Paper Suction Measurements (R. L. Lytton), Nov , 1993.
7. Appendix A. Measurement of Suction with Filter Paper. (ASTM D5298-92).
8. Appendix B. Sample Calculations of Heave and Shrinkage (R. L. Lytton) June 1995.
9. Appendix C. Measurement of Suction Compression Index (R. G. McKeen) July 1985.
10. GAMMA 100 - A Re-examination for Estimating Soil Swelling Properties: Preliminary Final Charts (A. P. Covar) Oct 1999.
11. Engineering Structues in Expansive Soils (R. L. Lytton) April 1997.
12. Copies of Transparencies from Engineering Structures in Expansive Soils (R. L. Lytton) April 1997.
13. Variation of Soil Suction with Depth in Dallas and Fort Worth, Texas (J. T. Bryant) 1998.
14. Method of Designing Slab-on-Ground Foundations in Arid and Semi-Arid climates for Irrigation, Lawn Watering, and Flower Bed Conditions (R. L. Lytton) May 1999.

15. Nomographic Calculation of Linear Extensibility in Soils Containing Coarse Fragments (G. G. S. Holmgren) 1968.

16. Edge Moisture Variation Distance as Determined by Thornthwaite Moisture Index and Soil Diffusion Properties (R. L. Lytton) Sept 1997 (Graph)

PAPER NO. 1

SOIL SUCTION MEASUREMENTS WITH FILTER PAPER METHOD

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 engineering applications in unsaturated soils. The filter paper method is a laboratory test method, and it is inexpensive and relatively simple. It is also the only known method that covers the full range of suction. With the filter paper method, both total and matric suction can be measured. If the filter paper is allowed to absorb water through vapor flow (non-contact method), then only total suction is measured. However, if the filter paper is allowed to absorb water through fluid flow (contact method), then only matric suction is measured. 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.

DISCUSSION

The working principle behind the filter paper method is that the filter paper will come to equilibrium with the soil either through vapor flow or liquid flow, and at equilibrium, the suction value of the filter paper and the soil will be the same. If the filter paper and soil are not in direct contact, then only total suction is measured. However, if the filter paper and soil are in intimate contact, then only matric suction is measured.

In engineering practice, soil suction is composed of two components: matric and osmotic suction. The sum of the matric and osmotic suction is called the total suction:

$$h_t = \frac{RT}{V}\ln\left(\frac{P}{P_O}\right) \tag{1}$$

where,

h_t	=	total suction (kPa)
R	=	universal gas constant [8.31432 J/(mol K)]
T	=	absolute temperature (in Kelvin)
V	=	molecular volume of water (m^3/kmol)
P/P_O	=	relative humidity (in percent)
P	=	partial pressure of pore water vapor (kPa)
P_O	=	saturation pressure of water vapor over a flat surface of pure water at the same temperature (kPa).

Suction is frequently represented in cm of negative head. The conversion from kPa to cm is 1 kPa = 10,198 cm. Suction is also frequently represented on a pF – scale. The pF is $\log_{10}$ (cm |suction|). Matric suction comes from the capillarity, texture, and surface adsorption forces of the soil. Osmotic suction arises from the salts that are present in the soil pore water. In the filter paper method, the soil specimen and filter paper are brought to equilibrium either in a contact (matric suction measurement) or in a non-contact (total suction measurement) method 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 a filter paper calibration curve of water content versus suction, the corresponding suction value is found from the curve, so the filter paper method is an indirect method of measuring soil suction. Therefore, a calibration curve should be constructed or be adopted (i.e., the two curves presented for different filter papers in *ASTM D 5298 – 94 Standard Test Method for Measurement of Soil Potential (Suction) Using Filter Paper*) in soil suction measurements.

REQUIRED APPARATUS

For Calibration Procedure:

a. Filter papers; the ash-free quantitative Schleicher & Shuell No. 589 White Ribbon or Whatman No. 42 type filter papers.
b. Salt solutions; sodium chloride (NaCl) solutions in a range between 0 (i.e., distilled water) to about 2.7 molality.
c. Sealed containers; 250 ml glass jars with lids which work nicely.
d. Small aluminum cans; the cans with lids are used as carriers for filter papers during moisture content measurements.
e. A balance; a balance with an accuracy to the nearest 0.0001 g. is used for moisture content determination.
f. An oven; an oven for determining the moisture contents of the filter papers by leaving them in it for 24 hours at 105 ± 5°C temperature in the *aluminum moisture cans* (as in the standard test method for water content determinations of soils).
g. A temperature room; a controlled temperature room in which the temperature fluctuations are kept below ±1°C is used for the equilibrium period.
h. Pressure plates and tensiometers; pressure plates and tensiometers are used for the low suction range in the calibration process.
i. An aluminum block; the block is used as a heat sink to cool the aluminum cans for about 20 seconds after removing them from the oven.

In addition, latex gloves, tweezers, plastic tapes, plastic bags, ice-chests, scissors, and a knife are used to set up the test.

For Soil Suction Measurements:

a. Filter papers; the ash-free quantitative Schleicher & Shuell No. 589 White Ribbon or Whatman No. 42 type filter papers.
b. Sealed containers; glass jars with lids which work nicely.
c. Small aluminum cans; the cans with lids are used as carriers for filter papers during moisture content measurements.
d. A balance; a balance with an accuracy to the nearest 0.0001 g. is used for moisture content determination.
e. An oven; an oven for determining the moisture contents of the filter papers by leaving them in it for 24 hours at $105 \pm 5^{o}C$ temperature in the *aluminum moisture cans* (standard test method for water content determinations of soils).
f. A temperature room; a controlled temperature room in which the temperature fluctuations are kept below $\pm 1^{o}C$ is used for the equilibrium period.
g. An aluminum block; the block is used as a heat sink to cool the aluminum cans for about 20 seconds after removing them from the oven.

In addition, latex gloves, tweezers, plastic tapes, plastic bags, ice-chests, scissors, and a knife are used to set up the test.

FILTER PAPER CALIBRATION PROCEDURE

The filter paper water content measurements are performed by two persons in order to decrease the time during which the filter papers are exposed to the laboratory atmosphere and, thus, the amount of moisture lost and gained during measurements is kept to a minimum. All the items related to filter paper testing are cleaned carefully. Gloves and tweezers are used to handle the materials in nearly all steps of the calibration. The filter papers and aluminum cans are never touched with bare hands.

The filter paper calibration curve is constructed using salt solutions as an osmotic potential source for suctions above about 2.5 pF and a combination of pressure plates and tensiometers for suctions below 2.5 pF. The procedure that is adopted for the calibration is as follows:

I. When using salt solutions:

a. NaCl solutions are prepared from 0 (i.e., distilled water) to 2.7 molality. The definition of molality is the number of moles of NaCl in 1000 ml of distilled water. For example, one mole of NaCl is 58.4428 g. Thus, 2 molality NaCl means 2 times 58.4428 g. or 116.8856 g. NaCl in 1000 ml distilled water. Table 1 gives the NaCl weights at different suction values.

Table 1. Osmotic suction values of NaCl solutions at 25°C.

NaCl Concentration (in molality)	Suction in cm units	Suction in pF units	Suction in kPa units	NaCl amount in grams (in 1000 ml distilled water)
0.000	0	0.00	0	0
0.003	153	2.18	15	0.1753
0.007	347	2.54	34	0.4091
0.010	490	2.69	48	0.5844
0.050	2,386	3.38	234	2.9221
0.100	4,711	3.67	462	5.8443
0.300	13,951	4.14	1,368	17.5328
0.500	23,261	4.37	2,281	29.2214
0.700	32,735	4.52	3,210	40.9099
0.900	42,403	4.63	4,158	52.5985
1.100	52,284	4.72	5,127	64.2871
1.300	62,401	4.80	6,119	75.9756
1.500	72,751	4.86	7,134	87.6642
1.700	83,316	4.92	8,170	99.3528
1.900	94,228	4.97	9,240	111.0413
2.100	105,395	5.02	10,335	122.7299
2.300	116,857	5.07	11,459	134.4184
2.500	128,625	5.11	12,613	146.1070
2.700	140,699	5.15	13,797	157.7956

b. A 250 ml glass jar is filled with approximately 150 ml of a solution of known molality of NaCl and the glass jar is labeled with the solution molality used for that jar.

c. Then, a small plastic cup is inserted into the glass jar. Holes are made in plastic cups in order for the filter papers to interact with and absorb water from the air in the closed jar. The configuration of the setup is shown in Fig. 1. Two filter papers are put on the plastic cup one on top of the other in order to double check the errors in the balance readings and in a case when one of the filter paper is accidentally dropped, the other filter paper is used. The glass jar lid is sealed with plastic tapes very tightly to ensure air tightness.

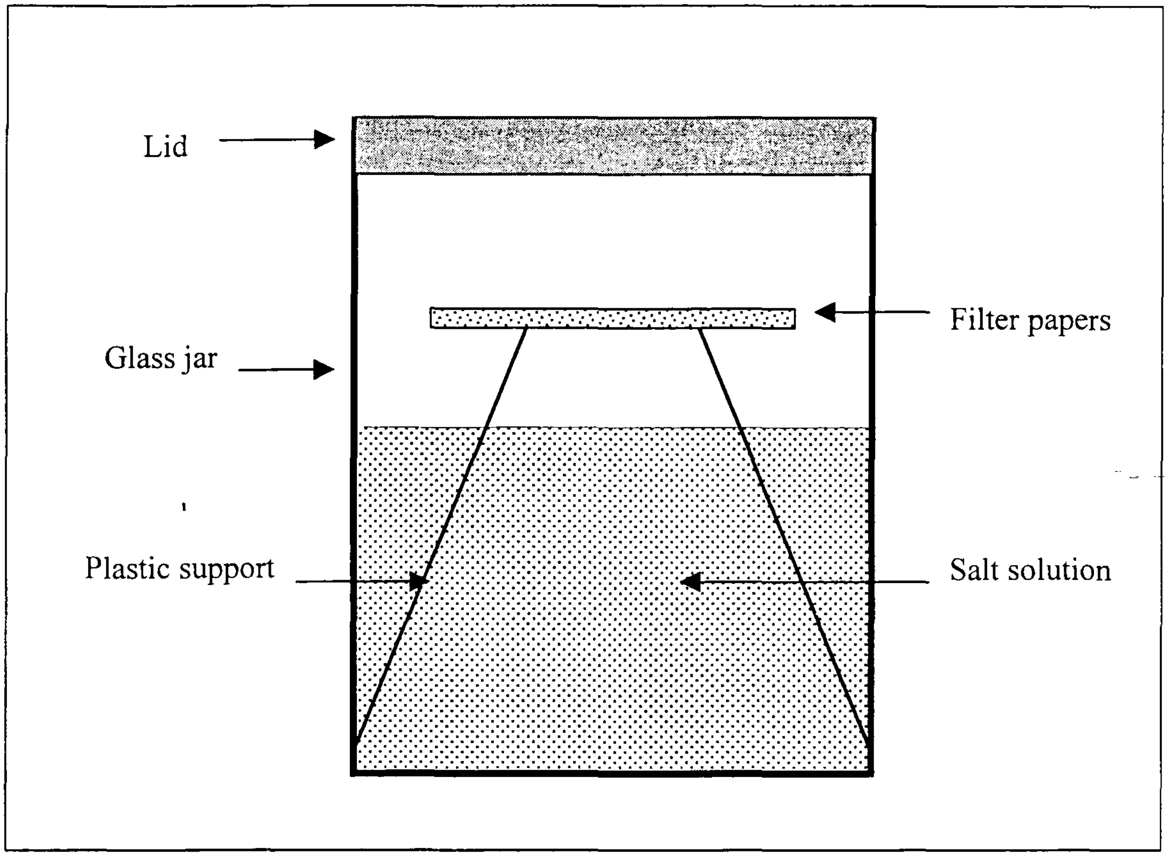

Fig. 1. Total suction calibration test configuration.

d. Steps b. and d. are repeated for each of the different NaCl concentrations.

Then, the prepared containers are put into plastic bags for extra protection. After that, the containers are put into the ice-chests in a controlled temperature room. The suggested equilibrium period is at least one week.

After the equilibrium period, the procedure for the filter paper water content measurement is as follows:

a. Before starting to take measurements, all the items related to the calibration process are cleaned carefully and latex gloves are used throughout the process. Before taking 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 on a filter paper water content measurement data sheet as shown in Fig. 2.

MEASUREMENT OF SOIL SUCTION USING FILTER PAPER

BORING NO.:____________ DATE TESTED:___________
DATE SAMPLED:_________ TESTED BY:______________
SAMPLE NO.:___________

Depth						
Moisture Tin No.						
Top Filter Paper/Bottom Filter Paper (circle)		Top/ Bot.	Top/ Bot.	Top/ Bot.	Top/ Bot.	Top/ Bot.
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					
Filter Paper Water Content, % (M_w/M_f)	w					
Suction, cm of water	h					
Suction, pF	h					

Fig. 2. Data sheet for filter paper water content measurements.

b. After that, all measurements are carried out by two persons. For example, while one person is opening the sealed glass jar, the other person is putting the filter paper into the aluminum can very quickly (i.e., in a few seconds, usually less than 5 seconds) using the tweezers.

c. Then, the weights of each can with wet filter papers inside are taken very quickly. The weights of cans and wet filter papers are recorded with the corresponding can numbers and whether the top or bottom filter paper is inside.

d. Step c. is 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 a $105 \pm 5^{o}C$ temperature for 24 hours inside the oven.

e. Before taking measurements on the dried filter papers, the cans are closed with their lids and allowed to equilibrate for 5 minutes in the oven. 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 acts as a heat sink and expedites the cooling of the can. After that, the can with the dry filter paper inside is weighed again very quickly. The dry filter paper is taken from the can and the cold can is weighed in a few seconds. Finally, all the weights are recorded on the data sheet shown in Fig. 2.

f. Step e. is repeated for every can.

II. When using pressure plates:

In the calibration process at low suction values (i.e., below about 2.5 pF) salt solutions can not be used, so for this part of the calibration (i.e., suction values less than about 2.5 pF) pressure plates and tensiometers should be employed. In the calibration process with pressure plates, the filter papers are either directly put on the porous disks or embedded in soil specimens on the porous disks. However, when the filter papers are embedded in the soil samples, protective filter papers need to be used in order to avoid any contamination of the filter paper on which the measurement relies. In other words, one filter paper from which the measurements will be taken is sandwiched between two larger size protective filter papers. The configuration of the pressure plate setup is shown in Fig.3. The suggested equilibrium period is about 3 to 5 days. The procedure is as follows:

a. Before starting to take measurements, all of the items related to the calibration process are cleaned carefully and latex gloves are used throughout the process. Before opening the pressure plate apparatus, all aluminum cans that are used for moisture content measurements are weighed to the nearest 0.0001 g. accuracy and recorded on a filter paper water content measurement data sheet as shown in Fig. 2.

b. After that, all measurements are carried out by two persons. For example, while one person is holding the aluminum can, the other person is putting the filter paper into the can very quickly (i.e., in a few seconds, usually less than 5 seconds) using the tweezers.

c. Then, the weights of each can with wet filter papers inside are taken very quickly. The weights of cans and wet filter papers are recorded with the corresponding can numbers.

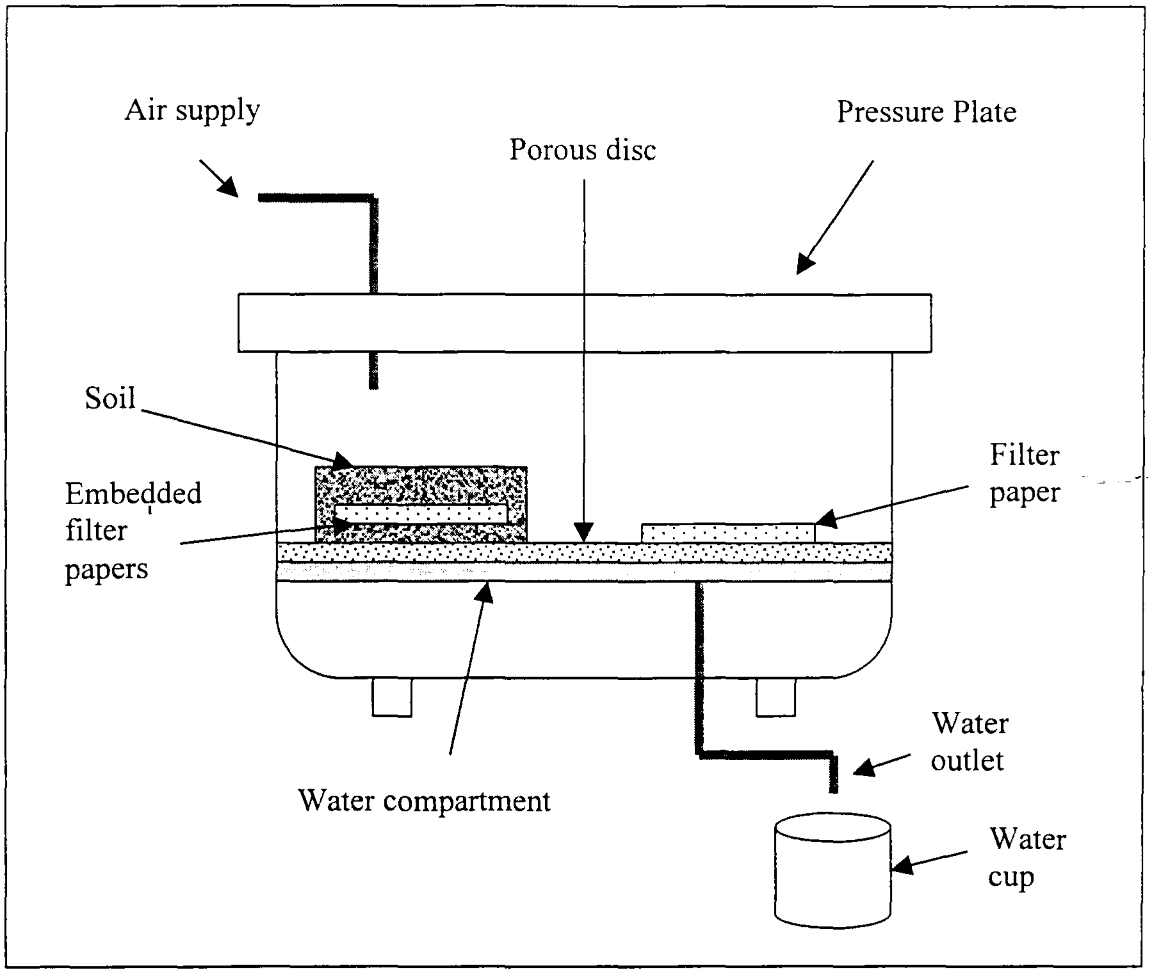

Fig. 3. Matric suction calibration test configuration using pressure plate.

d. Step c. is followed for every pressure plate. Then, all cans are put into the oven with the lids half-open to allow evaporation. All filter papers are kept at a 105 ± 5°C temperature for 24 hours inside the oven.

e. Before taking measurements on the dried filter papers, the cans are closed with their lids and allowed to equilibrate for 5 minutes in the oven. 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 acts as a heat sink and expedites the cooling of the can. After that, the can with the dry filter paper inside is weighed again very quickly. The dry filter paper is taken from the can and the cold can is weighed in a few seconds. Finally, all the weights are recorded on the data sheet shown in Fig. 2.

f. Step e. is repeated for every can.

The filter paper calibration curve of water content versus corresponding suction values is obtained from the calibration testing procedure. If suction values in pF or log (kPa) units are plotted with corresponding filter paper water content values a calibration curve for that *specific type filter paper* is obtained. Such a curve for Schleicher & Schuell No. 589 White Ribbon and Whatman No. 42 type filter papers is given by ASTM D 5298 (1994) and is reproduced in Fig. 4, on which the suction values are plotted as log (kPa).

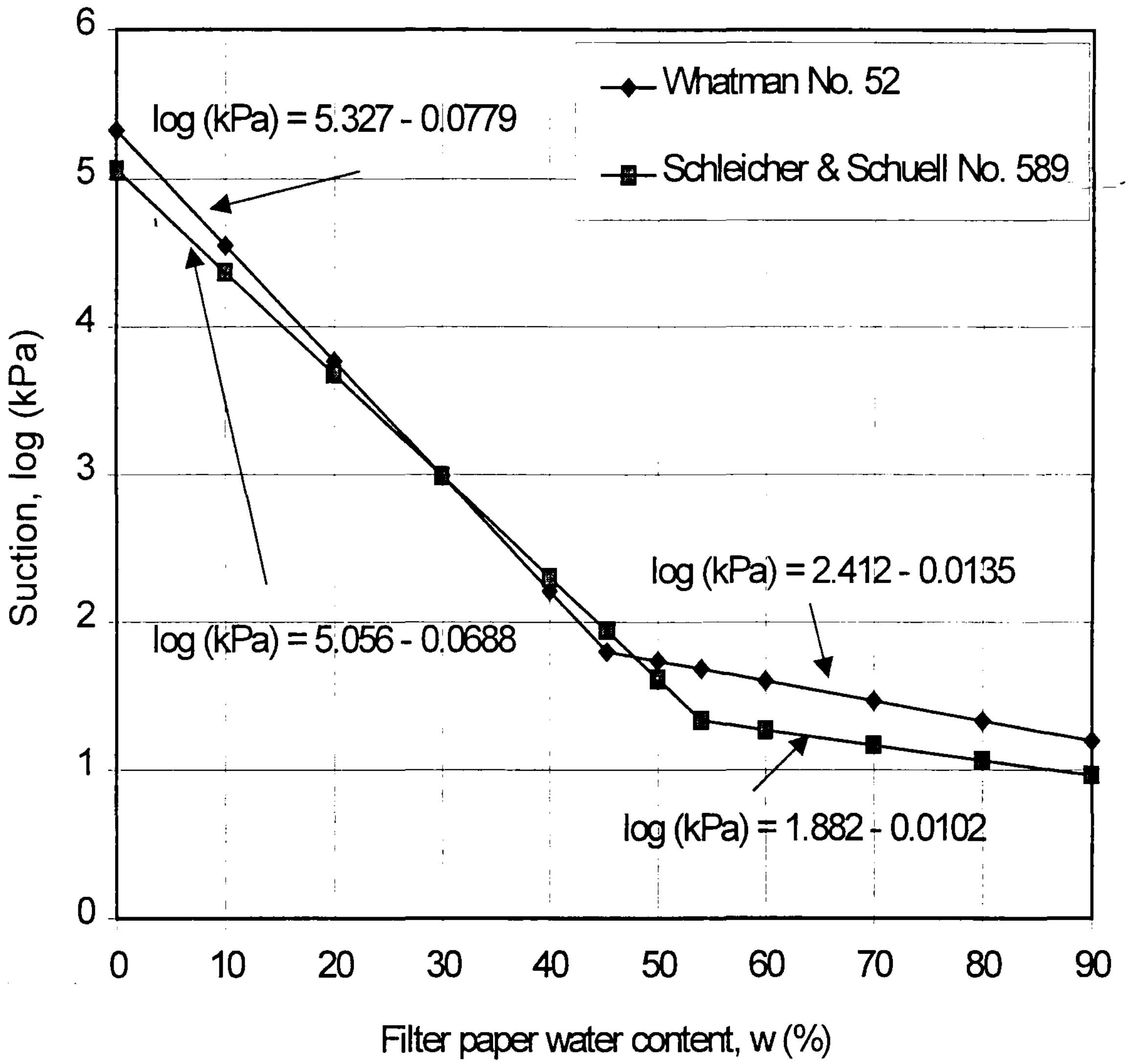

Fig. 4. Calibration curves for two types of filter papers (reproduced from ASTM D5298).

SOIL SUCTION MEASUREMENTS

Both total and matric suction measurements are possible from any type of soils and soils at any conditions (i.e., natural unprocessed and uncompacted, loose, compacted, treated soils, etc.) using the filter paper method. However, care must be taken when measuring matric suction because intimate contact between the filter paper and the soil is very important. If a good contact is not provided between the filter paper and the soil, then it is possible that the result will be total suction measurement rather than matric suction measurement.

The filter paper water content measurements are performed by two persons in order to decrease the time during which the filter papers are exposed to the laboratory atmosphere and, thus, the amount of moisture lost and gained during measurements is kept to a minimum. All the items related to filter paper testing are cleaned carefully. Gloves and tweezers are used to handle the materials in nearly all steps of the experiment. The filter papers and aluminum cans are never touched with bare hands. From 250 to 500 ml volume size glass jars are readily available in the market and can be adopted for suction measurements. Especially, the glass jars with 3.5" to 4" diameter in size can contain the 3" diameter Shelby tube samples very nicely. A typical setup for both the soil total and matric suction measurements is depicted in Fig. 5. The procedure that is adopted for the experiment is as follows:

Soil Total Suction Measurements:

a. At least 75 percent 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.
b. A ring type support (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.
c. 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.*
d. Then, the glass jar lid is sealed very tightly with plastic type electrical tape.
e. Steps a., b., c., and d. are repeated for every soil sample.
f. After that, the containers are put into the ice-chests in a controlled temperature room for equilibrium.

The suggested equilibrium period is at least one week. After the equilibrium period, the procedure for the filter paper water content measurement is as follows:

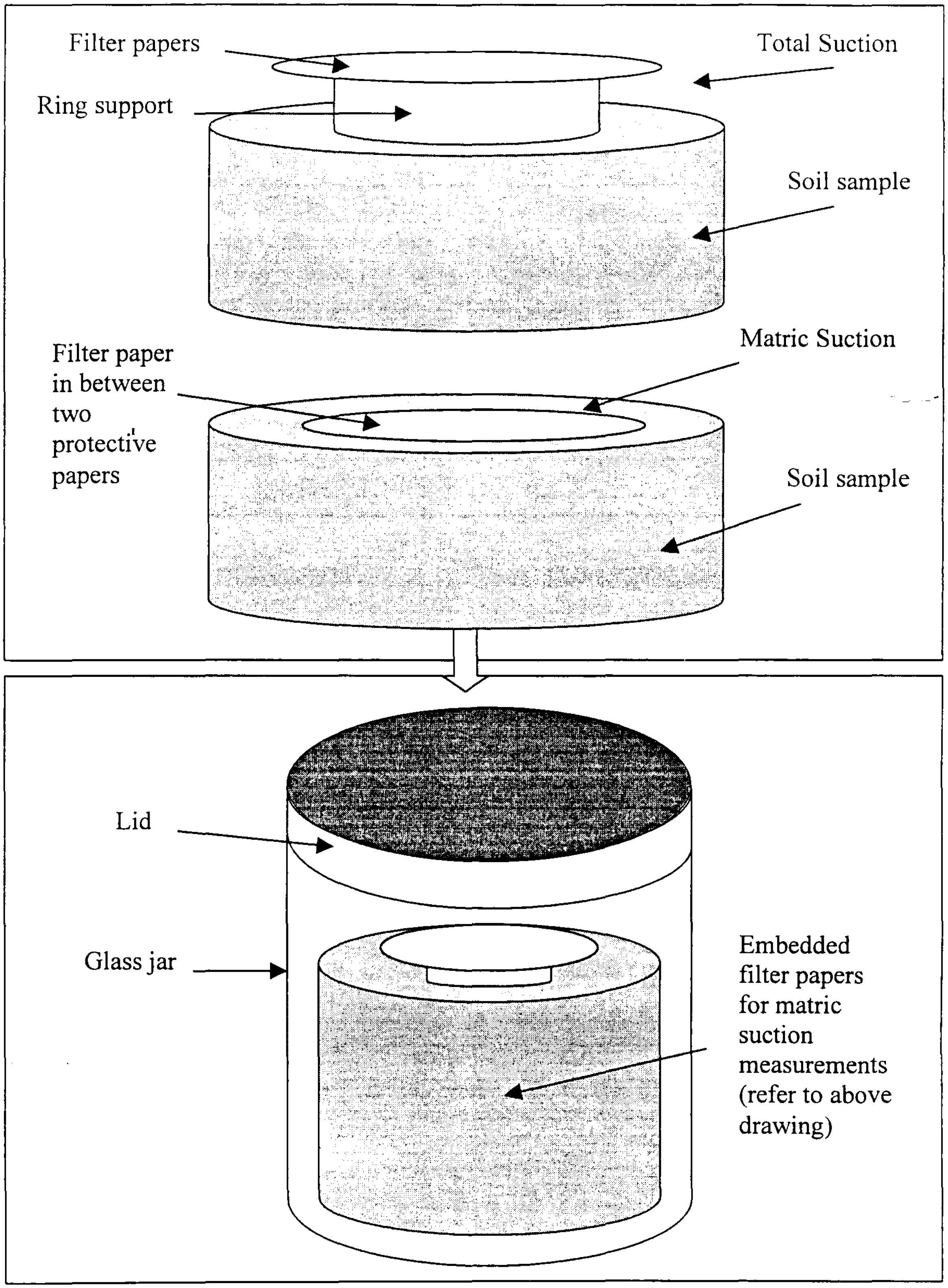

Fig. 5. Contact and noncontact filter paper methods for measuring total and matric suction.

a. Before starting to take measurements, all the items related to the measurement process are again cleaned carefully and latex gloves are used throughout the process. Before taking the glass jar containers from the temperature room, all aluminum cans that are used for moisture content measurements are weighed to nearest 0.0001 g. accuracy and recorded on a filter paper water content measurement data sheet as shown in Fig. 2.
b. After that, all measurements are carried out by two persons. For example, while one person is opening the sealed glass jar, the other person is putting the filter paper into the aluminum can very quickly (i.e., in a few seconds, usually less than 5 seconds) using the tweezers.
c. Then, the weights of each can with wet filter papers inside are taken very quickly. The weights of cans and wet filter papers are recorded with the corresponding can numbers and whether the top or bottom filter paper is inside.
d. Step c. is 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 a $105 \pm 5^{o}C$ temperature for 24 hours inside the oven.
e. Before taking measurements on the dried filter papers, the cans are closed with their lids and allowed to equilibrate for 5 minutes in the oven. 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 acts as a heat sink and expedites the cooling of the can. After that, the can with the dry filter paper inside is weighed again very quickly. The dry filter paper is taken from the can and the cold can is weighed in a few seconds. Finally, all the weights are recorded on the data sheet shown in Fig. 2.
f. Step e. is repeated for every can.

After obtaining all of the filter paper water content values 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:

a. A filter paper is sandwiched between two bigger 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.
b. Then, these sandwiched filter papers are inserted into the soil sample, which can fill up the glass jar, in *a very good contact manner.* *An intimate contact between the filter paper and the soil is very important*.
c. After that, this soil sample with embedded filter papers is put into the glass jar container.
d. The glass container is sealed up very tightly with electrical tape.
e. Steps a., b., c., and d. are repeated for every soil sample.

f. The prepared containers are put into the ice-chests in a controlled temperature room for equilibrium.

The suggested equilibrium period is 3 to 5 days. After the equilibrium period, the procedure for the filter paper water content measurement is as follows:

a. Before starting to take measurements, all the items related to the measurement process are again cleaned carefully and latex gloves are used throughout the process. Before taking the glass jar containers from the temperature room, all aluminum cans that are used for moisture content measurements are weighed to nearest 0.0001 g. accuracy and recorded on a filter paper water content measurement data sheet as shown in Fig. 2.
b. After that, all measurements are carried out by two persons. For example, while one person is opening the sealed glass jar, the other person is putting the filter paper into the aluminum can very quickly (i.e., in a few seconds, usually less than 5 seconds) using the tweezers.
c. Then, the weights of each can with wet filter papers inside are taken very quickly. The weights of cans and wet filter papers are recorded with the corresponding can numbers.
d. Step c. is 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 a $105 \pm 5^{\circ}C$ temperature for 24 hours inside the oven.
e. Before taking measurements on the dried filter papers, the cans are closed with their lids and allowed to equilibrate for 5 minutes in the oven. 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 acts as a heat sink and expedited the cooling of the can. After that, the can with the dry filter paper inside is weighed again very quickly. The dry filter paper is taken from the can and the cold can is weighed in a few seconds. Finally, all the weights are recorded on the data sheet shown in Fig. 2.
f. Step e. is repeated for every can.

After obtaining all of the filter paper water content values an *appropriate* calibration curve, such as the one in Fig. 4, is employed to get matric suction values of the soil samples.

PAPER NO.2

PREDICTION OF MOVEMENT IN EXPANSIVE CLAYS

Robert L. Lytton,[1] 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. Graphs of 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 modes. 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

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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 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 are 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; and
H = relative humidity, in percent.

"Suction" is a term used principally by engineers for the thermodynamic quantity, Gibbs free energy which is inherently negative, as seen in Eq. (1), and generates tension in the pore water stretching between soil particles.

Total suction may have two components: matrix suction, which is due to the attraction of water to the soil particle surfaces and osmotic suction, which is due to dissolved salts or other solutes in the pore water. A complete discussion of suction and its measurement is found in the book by Fredlund and Rahardjo (1993), 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 in cm of water, a positive value.

Fig. 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 confined 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 conceptual graph of suction-versus-volume can be drawn using the relations of each to water content. This is illustrated in Fig. 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 mechanical pressure or total stress.
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:

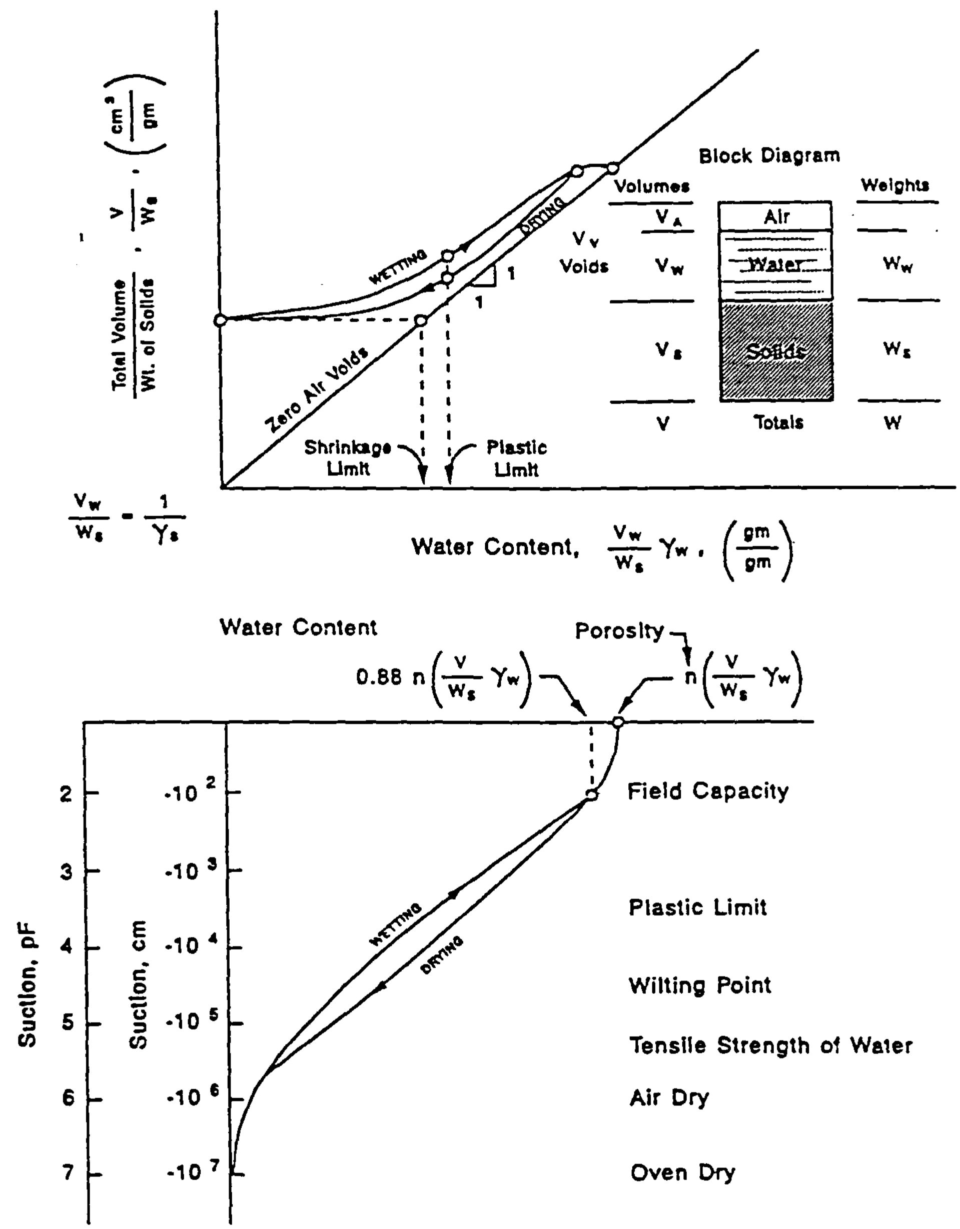

FIG. 1. Suction-Water-Content-Volume Relationships in the Absence of Total Stress

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left[\frac{h_f}{h_i}\right] - \gamma_\sigma \log_{10}\left[\frac{\sigma_f}{\sigma_i}\right] - \gamma_\pi \log_{10}\left[\frac{\pi_f}{\pi_i}\right] \qquad (3)$$

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; and
γ_π = the osmotic suction compression index.

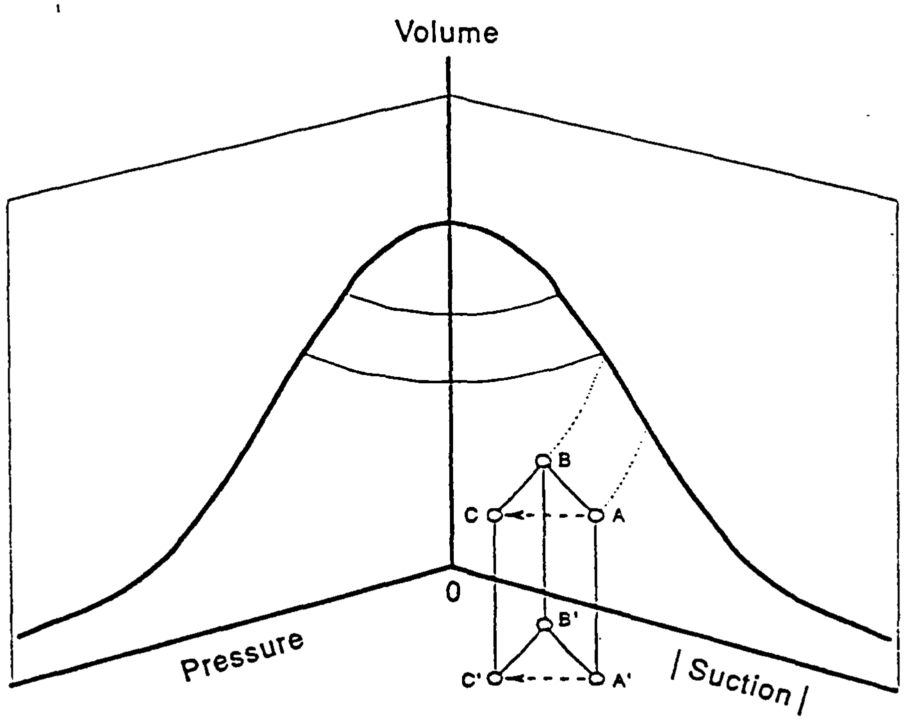

FIG. 2. Pressure-Suction-Volume Surface for Expansive Soil

The mean principal stress compression index is related to the commonly used compression index, C_c, by:

$$\gamma_\sigma = \frac{C_c}{1 + e_o} \qquad (4)$$

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 Eq. (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 Fig. 3.

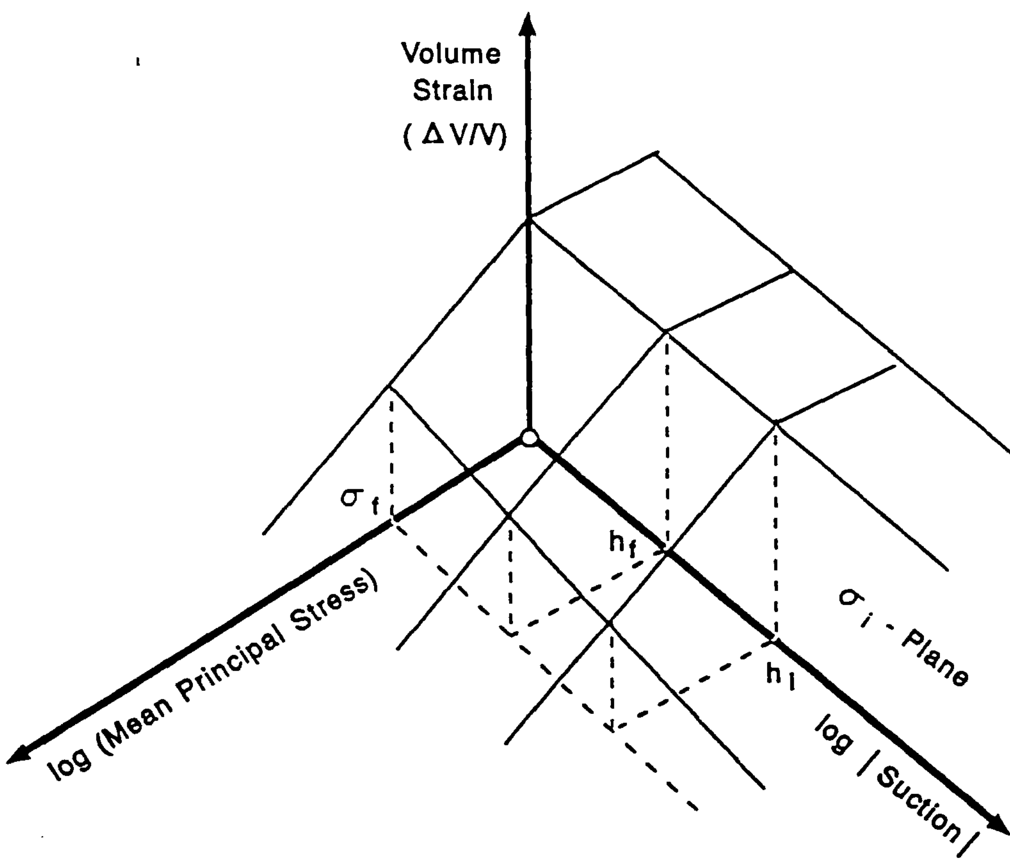

FIG. 3. Graph of Volume Strain as a Function of Log | Suction | and Log (Mean Principal Stress)

The mean principal stress is estimated by:

$$\sigma = \left[\frac{1+2K_o}{3}\right]\sigma_z$$

where σ_z = the vertical stress at a point below the surface in a soil mass; and
K_o = 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 coefficient, K_o, can vary between 0.0 and passive earth pressure levels. Typical values that have been back-calculated 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 closed and the soil is swelling.

The vertical strain is estimated from the volume strain by using a crack fabric factor, f.

$$\frac{\Delta H}{H} = f\left[\frac{\Delta V}{V}\right] \tag{6}$$

Back-calculated 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.

$$\Delta = \sum_{i=i}^{n} f_i \left[\frac{\Delta V}{V}\right]_i \Delta z_i \tag{7}$$

where n = the number of depth increments;
Δz_i = the i^{th} depth increment; and
$(\Delta V/V)_i$ = 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_h. This may be estimated with the chart developed by McKeen (1981), shown in Fig. 4. The two axes are given by the activity ratio, Ac, and the Cation Exchange Activity ratio, CEAc, which are defined as follows:

$$Ac = \frac{PI\,\%}{\frac{(\%\ -2\ \text{micron})}{(\%\ -\text{No.}200\ \text{sieve})} \times 100} \tag{8}$$

$$CEAc = \frac{CEC\ \frac{\text{milliequivalents}}{100\ \text{gm of dry soil}}}{\frac{(\%\ -2\ \text{micron})}{(\%\ -\ \text{No. }200\ \text{sieve})} \times 100} \tag{9}$$

where PI = the plasticity index in percent.

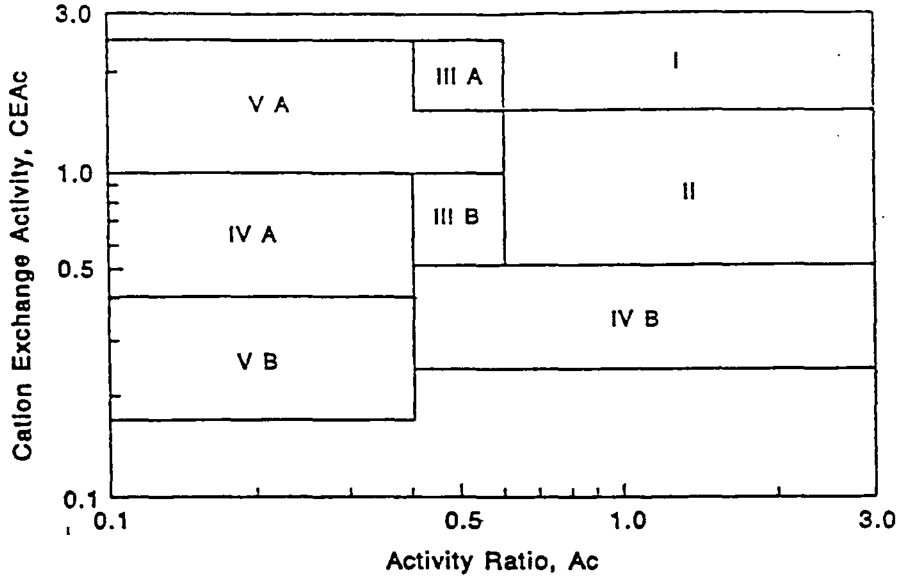

FIG. 4. Chart for the Prediction of Suction Compression Index Guide Number

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 (Mojeckwu 1979) or it may be estimated with sufficient accuracy by Eq. (10) which was developed by Mojeckwu (1979):

$$CEC \cong (PL\%)^{1.17} \tag{10}$$

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:

$$\gamma_h = \gamma_o \times \left[\frac{\% - 2\ \text{micron}}{\% - \text{No. } 200}\right] \tag{11}$$

for the soil portion finer than the No. 200 sieve. A method for estimating γ_h for soils containing coarse-grained particles was developed by Holmgreen (1968).

The mean principal stress compression index, γ_σ, is related to γ_h by the following equation:

$$\gamma_\sigma = \gamma_h \frac{1}{1 + \dfrac{h}{\Theta\left[\dfrac{\partial h}{\partial \Theta}\right]}} \tag{12}$$

where Θ = the volumetric water content; and

$\partial h/\partial \Theta$ = the slope of the suction-versus-volumetric water content curve.

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 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 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 for the unsaturated permeability (Gardner, 1958) 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; and
k_o = saturated permeability, cm/s.

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 for the unsaturated permeability (Mitchell 1980) gives:

$$\Delta h = -\Delta Z \left\{ 1 + \frac{v}{k_o} \left[\frac{h}{h_o} \right] \right\} \tag{18}$$

where h_o = about -100 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 Fig. 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. It is speculated that in general, the pF-level where cracks begin to form is the equilibrium pF-value which corresponds to the local value of the Thornthwaite Moisture Index (Thornthwaite 1948). The velocity of water entering or leaving the soil may be estimated from Thornthwaite Moisture Index moisture balance computations.

The suction profiles for two transient states can be predicted approximately using Mitchell (1980):

$$U(Z,t) = U_e + U_o \exp\left\{-Z\sqrt{\frac{n\pi}{\alpha}}\right\} \cos\left\{2\pi nt - Z\sqrt{\frac{n\pi}{\alpha}}\right\} \tag{19}$$

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/s); and
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.

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.

Eq. (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.

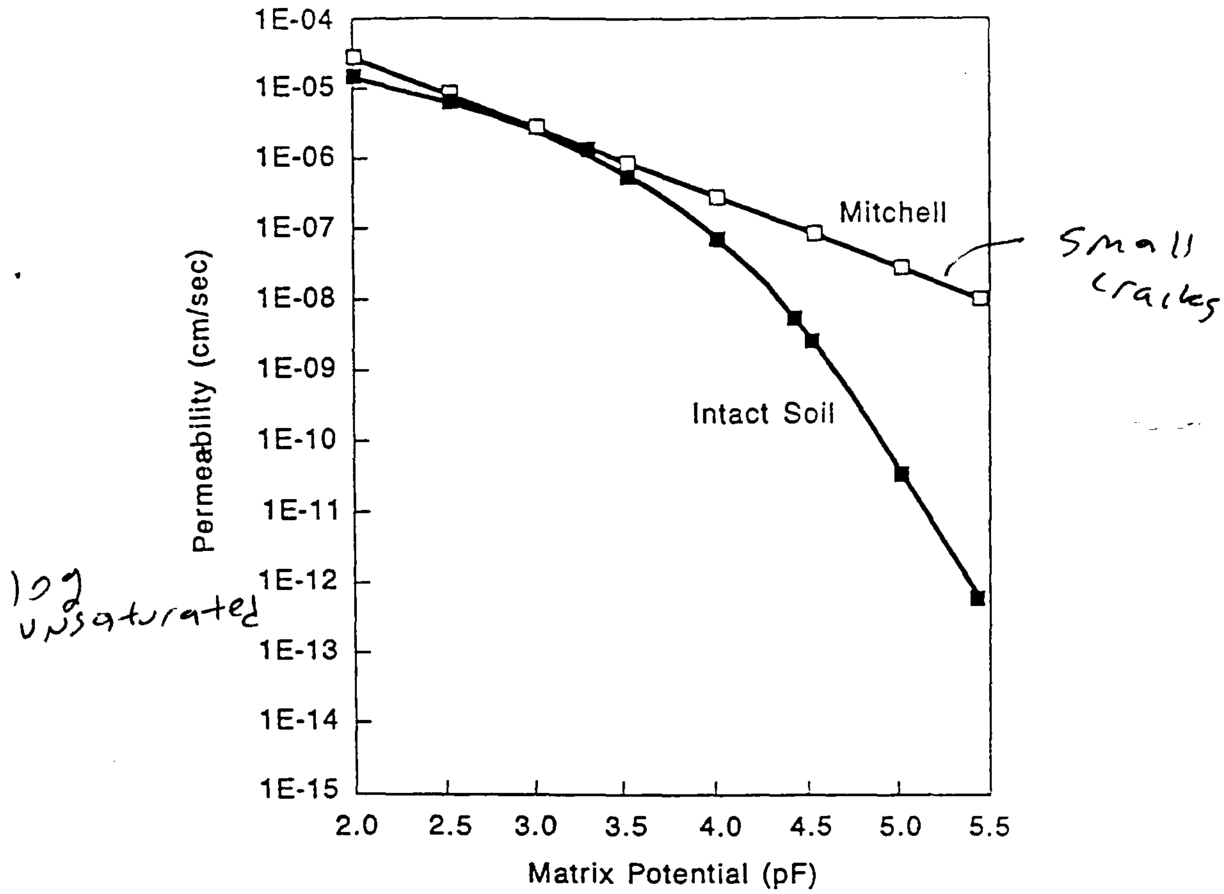

FIG. 5. Permeability Relationships for Intact and Cracked Clay Soil

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 Fig. 6. Actual data for a monitoring site near Seguin, Texas are shown in Fig. 7, (Jayatilaka et al. 1993). 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 relationship for 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 given 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.

TABLE 2. Wet Suction Profile Values

Thornthwaite Moisture Index	Mitchell Unsaturated Permeability (cm^2/s)	U_e (pF)	U_o (pF)
-46.5	5×10^{-5} 1×10^{-3}	4.43 4.27	0.25 0.09
-11.3	5×10^{-5} 1×10^{-3}	3.84 2.83	1.84 0.83
26.8	5×10^{-5} 1×10^{-3}	3.47 2.79	1.47 0.79

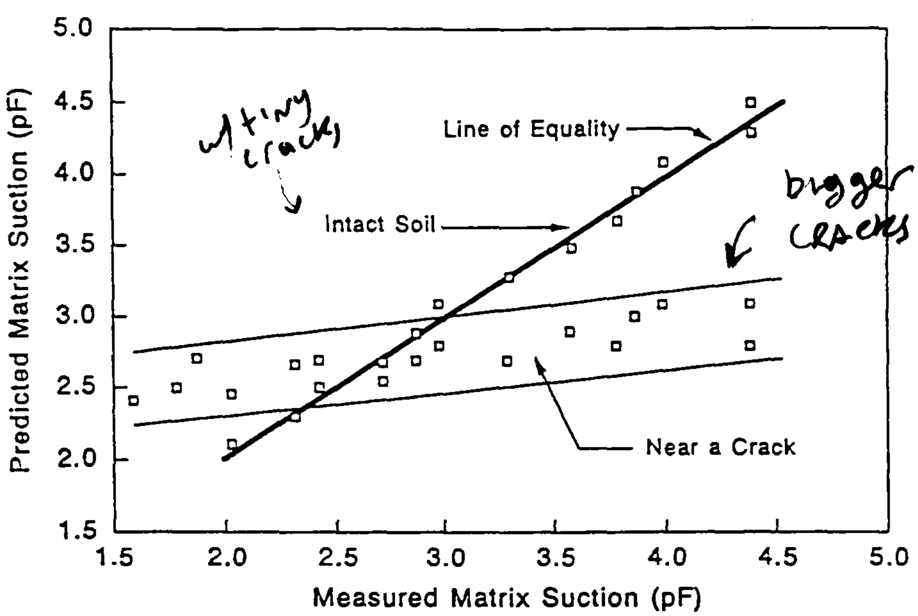

FIG. 6. A Typical Pattern of Measured Soil Suction vs. Predicted Soil Suction

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 surface evapotranspiration, then it is controlled by a high water table. This will usually be within about 10 m (30 feet) below the surface.

If the suction is higher than expected then there is osmotic suction present. Osmotic suction levels may be measured with vacuum desiccators.

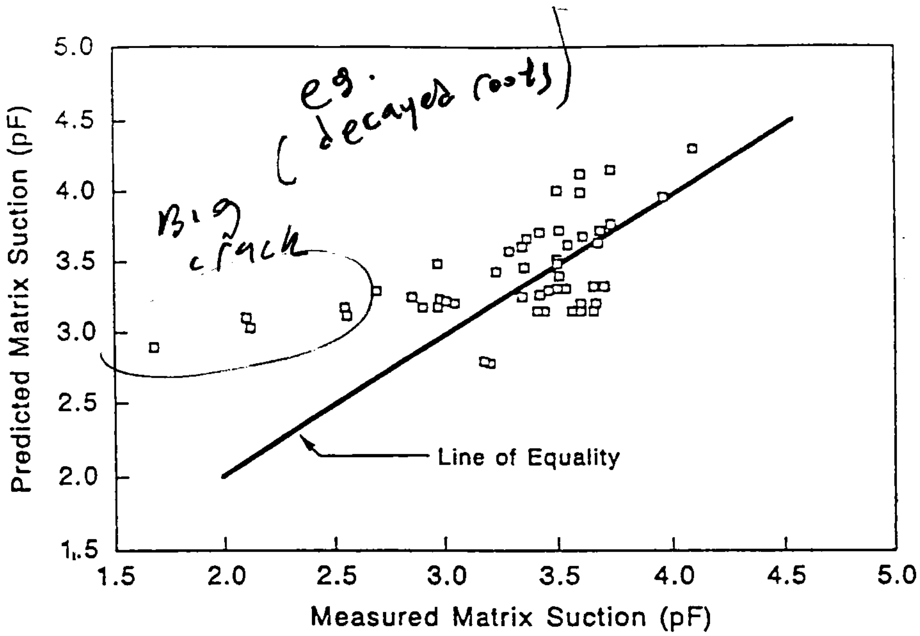

FIG. 7. **Measured Soil Suction vs. Predicted Soil Suction at Seguin (Jayatilaka et al. 1993)**

TABLE 3. Equilibrium Suction Values, U_e

TMI	Mitchell Unsaturated Permeability, cm^2/s		
	5×10^{-5}	2.5×10^{-4}	1.0×10^{-3}
-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

Estimates of Unsaturated Soil Properties

The fundamental definition of p is :

$$p = \frac{k_o|h_o|}{0.4343} \qquad (20)$$

where $|h_o| = 100$ cm for clays.

The units of k_o, the saturated permeability, (cm/s), and $|h_o|$, the suction at which the soil desaturates (cm) produce units of (cm^2/s) for the Mitchell unsaturated permeability.

big cracks $> .001$
small cracks = other
tight $< .0005$

The Mitchell unsaturated permeability, p, is estimated by:

$$p = \frac{\alpha\gamma_d}{|S|\gamma_w} \quad (\text{cm}^2/\text{s})$$

where γ_w = the unit weight of water;
α = the Mitchell diffusion coefficient, cm^2/s, which is used in Eq. (19);
$|S|$ = the absolute value of the slope of the pF-vs-gravimetric water content, w line; and
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:

$$S = -20.29 + 0.1555(LL\%) - 0.117(PI\%) + 0.0684(\%\,\text{-}\#200) \tag{22}$$

where LL = the liquid limit in percent;
PI = the plasticity index in percent; and
-#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 Fig. 1. The correction term in the relation between γ_h and γ_σ given in Eq. (12) is found by:

$$\frac{h}{\Theta\left[\frac{\partial h}{\partial \Theta}\right]} = \frac{0.4343}{Sw} \tag{24}$$

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 Fig. 8. First, point A is located at the intersection of the field capacity volumetric water content ($= 0.88\,\Theta_{sat}$) 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 $0.10\,\Theta_{sat}$ 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.

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 by Unified Soil Class (Mason et al. 1986)

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 (1980). 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.

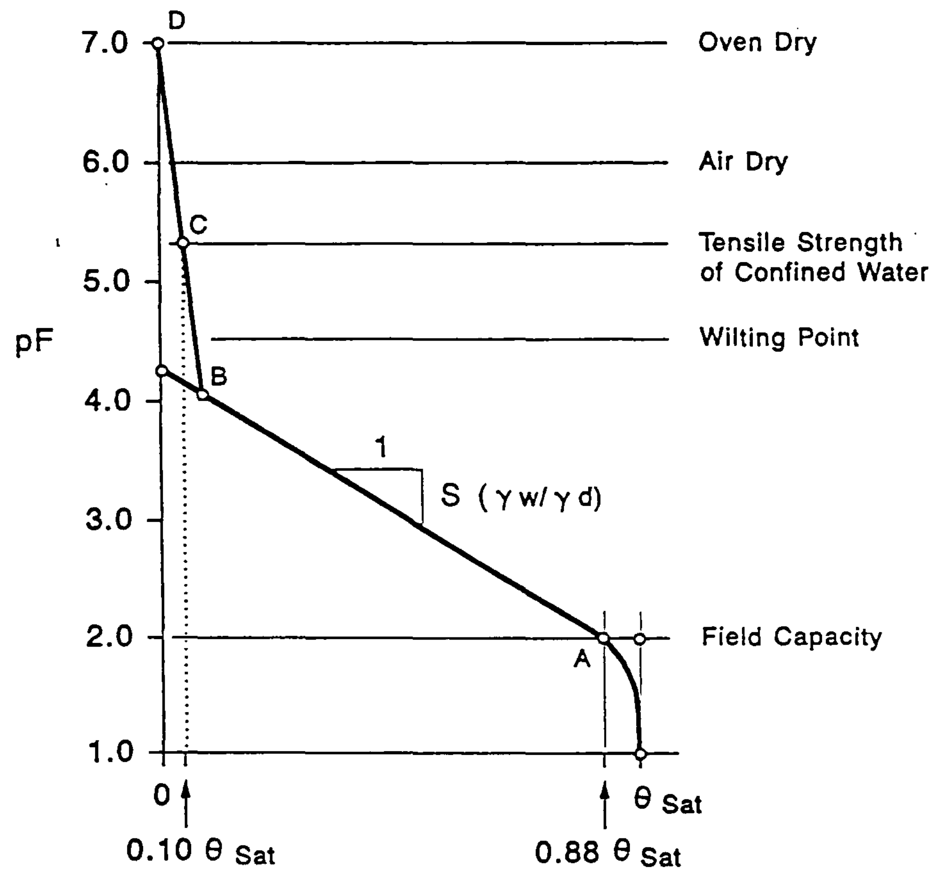

FIG. 8. Approximate Construction of a Suction (pF)-versus-Volumetric Water Content Curve

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 Figs. 9 and 10. Seven different soils were used in the study. No distance less than 2.0 feet (0.6 m) was considered to be adequate for design purposes.

In Fig. 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 with properties between No. 3 and No. 4 have edge moisture variation distances in the range presently used in the PTI manual (1980).

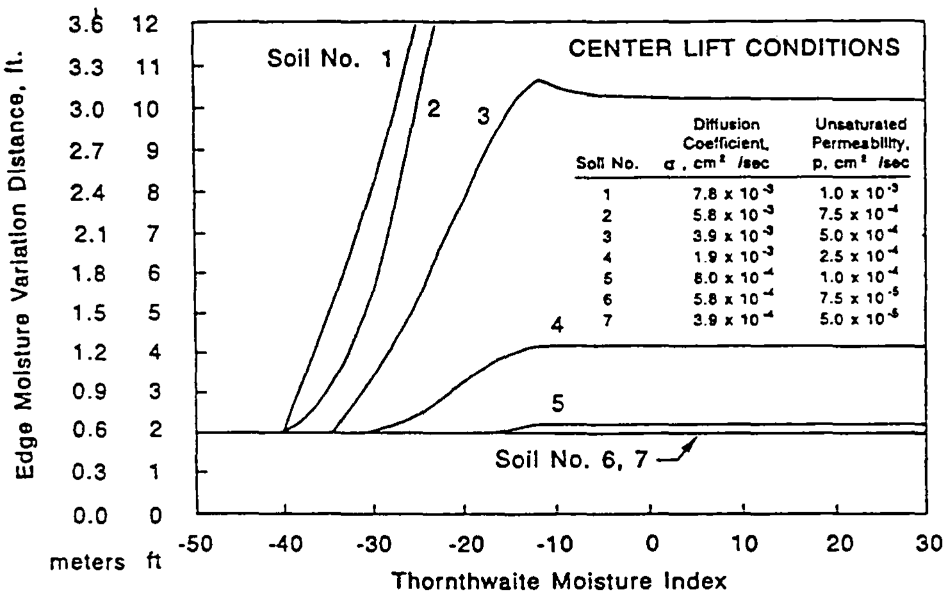

FIG. 9. Edge Moisture Variation Distances for the Center Lift Moisture Condition

In Fig. 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.

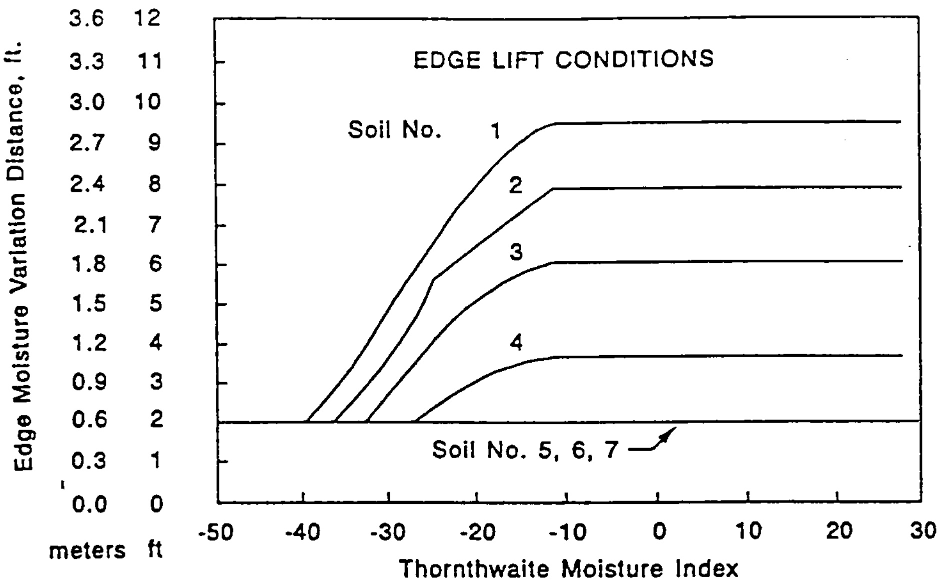

FIG. 10. Edge Moisture Variation Distances for the Edge Lift Moisture Condition

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 Figs. 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.

ACKNOWLEDGEMENTS

The author gratefully acknowledges D. A. Gay for developing the coupled unsaturated moisture flow and elasticity program used in the analysis reported in this paper and R. Jayatilaka for making the multiple runs of the program which resulted

in the family of curves in Figs. 9 and 10. The Texas Department of Transportation supported the research project that produced the data shown in Fig. 7.

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PAPER NO.3

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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 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.

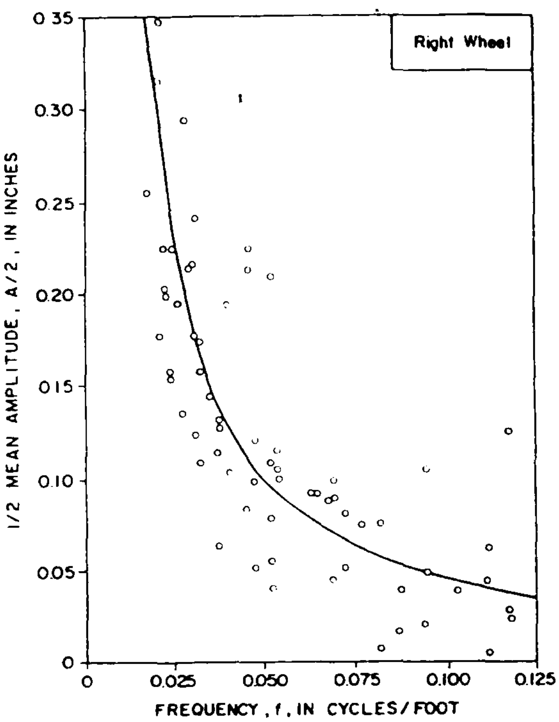

Figure 1: Amplitude versus Frequency Spectrum of a Pavement on Expansive Soil.

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.

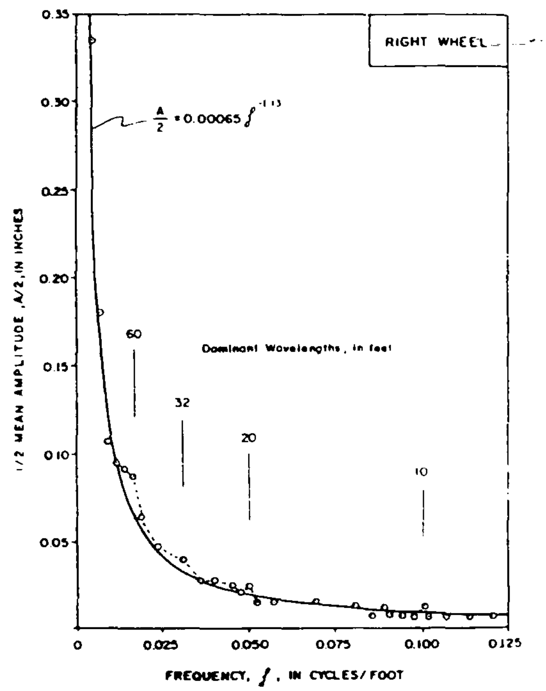

Figure 2: Fast Fourier Transform Amplitude versus Frequency Spectrum of a Pavement on Expansive Soil.

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.

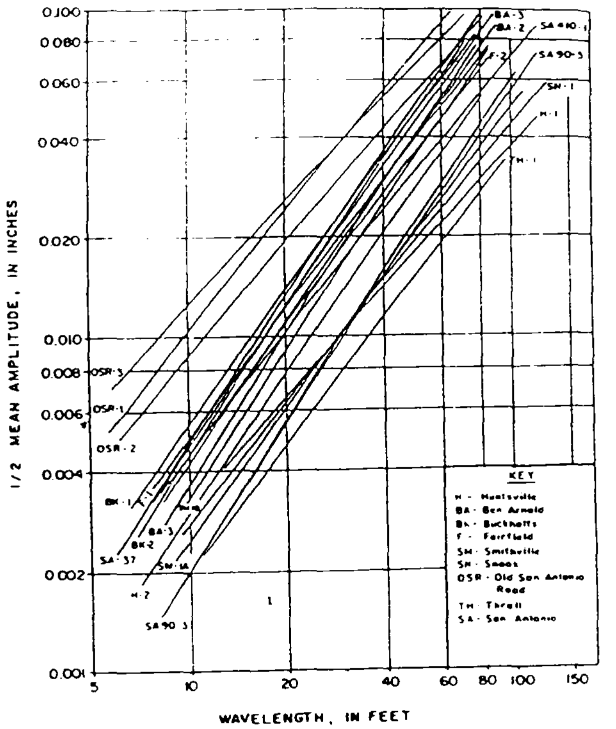

Figure 3: A Collection of Fast Fourier Transform Amplitude versus Wavelength Spectra of Pavements on Expansive Soils.

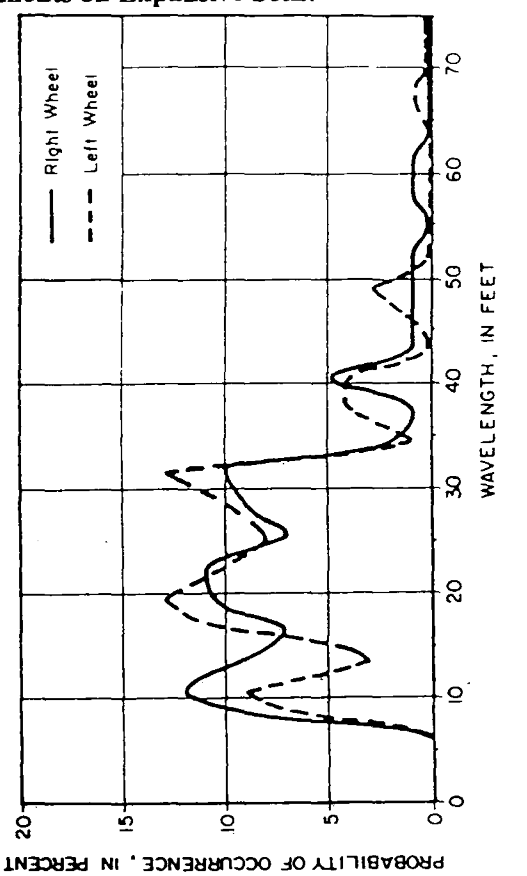

Figure 4: Probability Density of Right and Left Wheel Path Wavelengths of a Pavement on Expansive Soil

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,

$$\overline{\sigma_w} = -\theta\ h_m \qquad (2)$$

where $\overline{\sigma}_w$ = 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

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

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

$$s = c' + \left[(\sigma - u_a) + \theta\ (u_a - u_w)\right]\ \tan\phi' \qquad (4)$$

since

$$\tan\phi^b = \theta\ \tan\phi' \qquad (5)$$

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

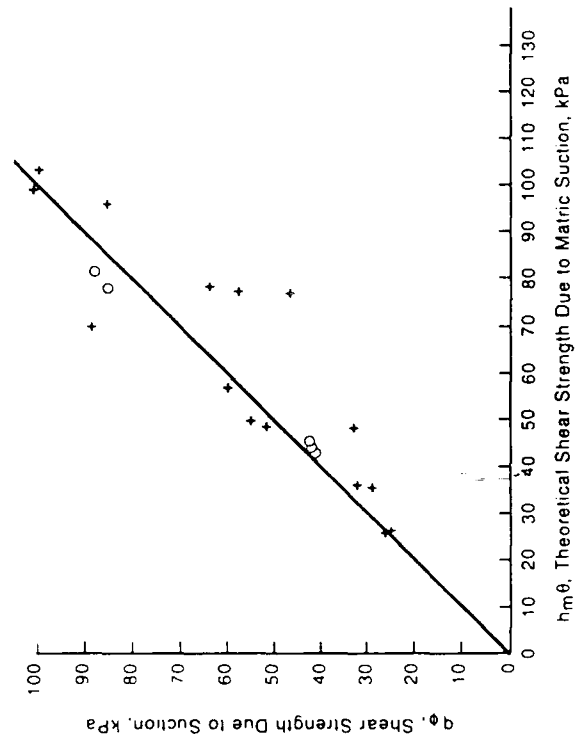

Figure 5: Measured versus Predicted Shear Strength of Soil Due to Matric Suction (Lam, 1980).

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

$$\tan\phi^b = 1.0\tan\phi' \qquad (6)$$

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,

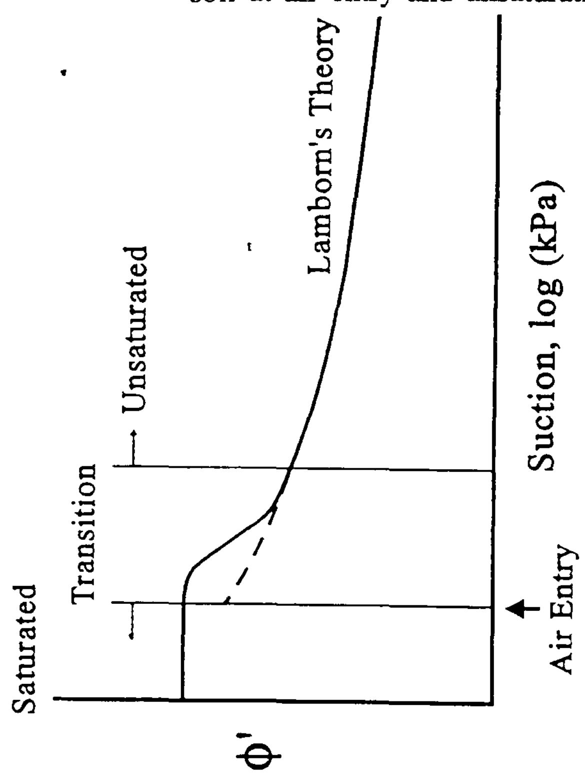

Figure 6: Transition of Friction Angle due to Matric Suction from the Saturated to Unsaturated State.

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 ϕ^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

θh_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:

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

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):

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

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):

$$I_{1u} = I_1 - 3\theta f h_m \qquad (18)$$

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

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

where

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

$\alpha = I_{1u}^{k_2} J_2^{\frac{1}{2}k_3}$

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

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

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

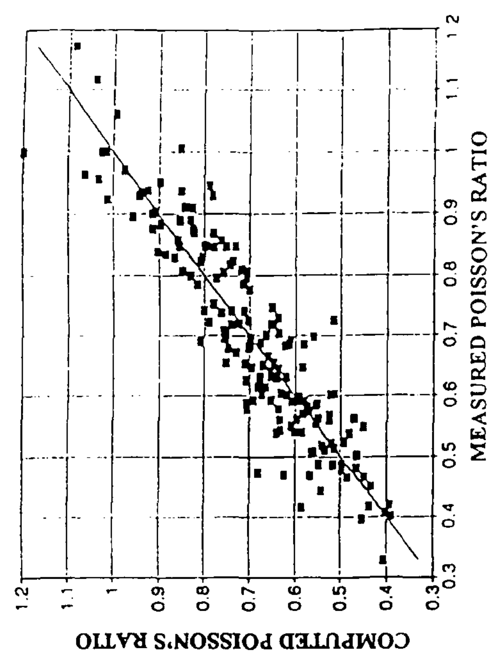

Figure 7: Measured versus Predicted Poisson's Ratio for Granular Materials.

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

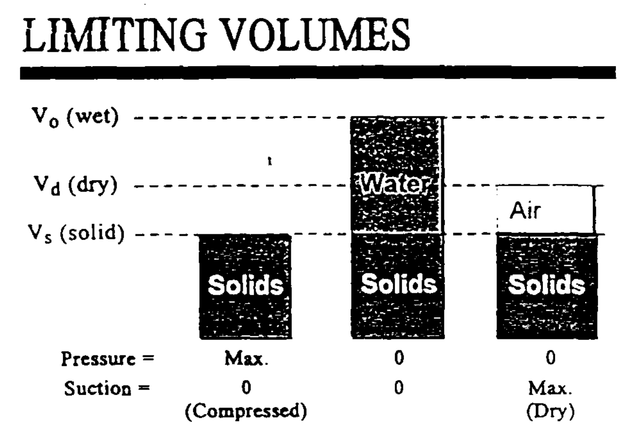

Figure 8a: Natural Limits of the Volume Change Process in Unsaturated Soils.

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

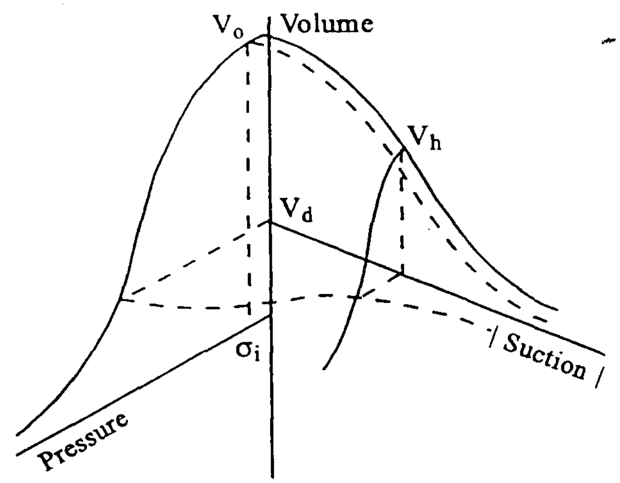

Figure 8b: The Volume - Mean Principal Stress - Suction Surface.

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

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

This produces the equation

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

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:

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

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.

$$\gamma_\sigma \cong \frac{C_s}{1 + e_o} \tag{27}$$

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

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

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

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

where $m = \dfrac{K_{of}\,\sigma_v}{G}$ (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_σ; 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

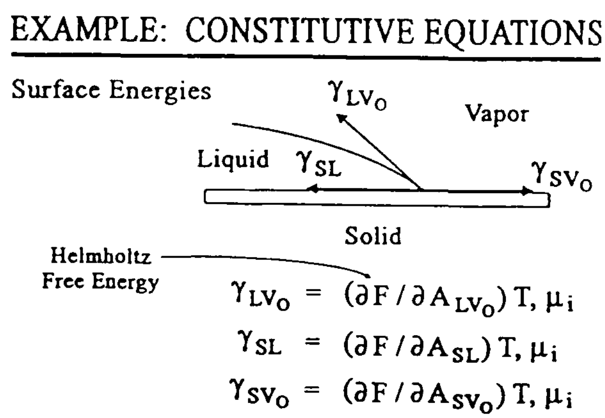

Figure 9: Vector Diagram Illustrating Young's Equation

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}	Γ^{+}	Γ^{-}
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 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.

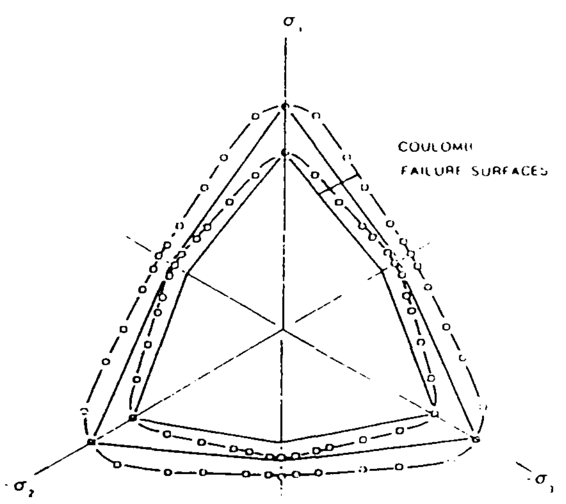

Figure 10: Mohr - Coulomb and Lade Yield Functions Compared with Measured Yield Data (Lade and Duncan, 1973)

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
$\lvert h_m \rvert$	=	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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PAPER NO. 4

VOLUME CHANGE

AND

FLOW CALCULATIONS

IN

EXPANSIVE SOILS

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INTRODUCTION

VOLFLO is a computer program which performs volume change and flow calculations for expansive soils. It was developed in the early 1980's at Texas A&M University under the guidance of Dr. Robert L. Lytton. The Post Tensioning Institute (PTI) distributes VOLFLO for use in conjunction with its PTISLAB program.

OWNERSHIP

VOLFLO is the property of and is furnished under license by Ray D. Ullrich & Associates, Inc. The program is confidential and may be used, copied and disclosed only in accordance with the terms of the license agreement.

DISCLAIMER

The method of analysis utilized in this program is based solely on the work performed at Texas A&M University. Ray D. Ullrich & Associates, Inc. makes no warranty, either expressed or implied as to the accuracy or applicability of this method.

While care has been exercised in the writing of the program and in the presentation of relevant data on its use, the writers cannot claim unequivocally that there are no errors. Therefore, the user must accept full responsibility for the use of the program. The writers do not state that the program will yield correct information and no warranty is given or implied to that effect.

It should be obvious that the execution of volume change and flow calculations in expansive soils, including the use of VOLFLO, is not an exercise that can be relegated to a non-professional. The writers assume that a qualified engineer will use this program and evaluate its results.

SYSTEM REQUIREMENTS

VOLFLO runs on IBM PCs and compatibles under any version of DOS numbered 2.0 or higher. It requires 256K of RAM and operates on either the monochrome or the color/graphics display adapter. The program runs with floppy disks or hard disks. It uses any printer that accepts the Epson/IBM command set.

MAKING A BACK-UP COPY OF VOLFLO

VOLFLO is not "copy protected." This means that you can and should make a working copy of the disk and keep the original in a safe place. Refer to your DOS manual for instructions on how to copy disks. It is illegal to make copies of VOLFLO for use by another person or for use on more than one machine.

INSTALLING ON A HARD DRIVE

To install VOLFLO on a hard drive, create a directory called PTI and copy all of the files with the extension .EXE into the directory. See your DOS manual for instructions on creating directories and copying files. You may also wish to create subdirectories for the storage of input and output files.

STARTING THE PROGRAM FROM A FLOPPY DRIVE

Boot your computer with DOS 2.0 or higher. If your computer has only one floppy disk drive, remove the DOS disk and insert the program disk. At the prompt, type VOLFLO. For a computer with two floppy drives, we recommend that after boot up you insert the program disk in drive A and a blank, formatted diskette in drive B for data file storage.

STARTING THE PROGRAM FROM A HARD DISK

Boot the computer with DOS and change the directory to PTI. At the prompt, type VOLFLO.

PROGRAM DESCRIPTION

VOLFLO calculates volume change and moisture flow rates in expansive soils for five different sets of effects occurring near a foundation system:

1. General case - no effects
2. A vertical barrier to moisture flow at the edge of the foundation
3. A horizontal barrier to moisture flow at the edge of the foundation
4. A tree near the edge of the foundation. Tree roots may or may not extend beneath the foundation.

4a. A flower bed near the foundation.

4b. Total heave or shrinkage.

5. Both trees and horizontal barrier. This case is a combination of cases 3 and 4.

The volume change can be computed for an expansive soil with a depth to constant suction down to twenty (20) feet. The soil may be composed of up to six layers within the active zone.

The constant suction value may lie between 2 pF and 5 pF.

THEORY

VOLFLO calculates the soils shrinkage and swelling using soil suction data. Only the effect of horizontal moisture flow is considered. The effect of vertical moisture flow is neglected.

For horizontal moisture flow:

$\Delta h = -(v/k)\,\Delta x$

where Δh = horizontal change in suction, cm
Δx = change in horizontal location, cm
v = velocity of moisture flow, om sec

$$k = \frac{2 \times 10^{-6}}{1 + 10^{-9}|h|^3} \qquad \frac{\textit{the permeability}}{cm/sec}$$

The shrinking or swelling of the soil is calculated at the edge of the foundation.

For swelling, the percent volume change is:

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

For shrinking, the percent volume change is

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

where h_f = suction at edge of foundation, cm
h_i = equilibrium suction at depth z, cm
σ_i = overburden correction const. g/cm^2

$\sigma_i = hco\ (\gamma_t)$

where hco = depth above which no volume change correction is made
σ_f = mean pressure at depth z, g/cm^2
$\sigma_f = z(\gamma_t)(1+2k_o/3)$

where k_o - lateral earth pressure coefficient
γ_t = unit weight of soil

$\Delta V/V$ = percent volume change in decimal form
γ_h = volume change coefficient due to shrinking or swelling
γ_h = % fine clay (decimal) γ_{100}
where γ_{100} = volume change guide number (See page 11)

The vertical volume change at depth z below the edge of the foundation is:

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

where f is the vertical volume change coefficient

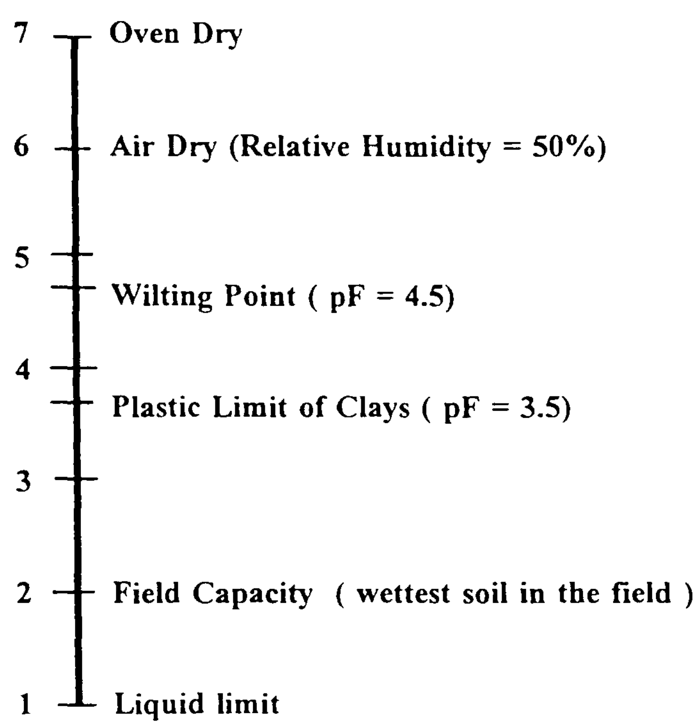
Therefore the total heave or shrinkage at depth z is:

$$y_m = \Sigma \left(\frac{\Delta H}{H} \right)_i (\Delta z)$$

where Δz is the vertical increment, cm

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

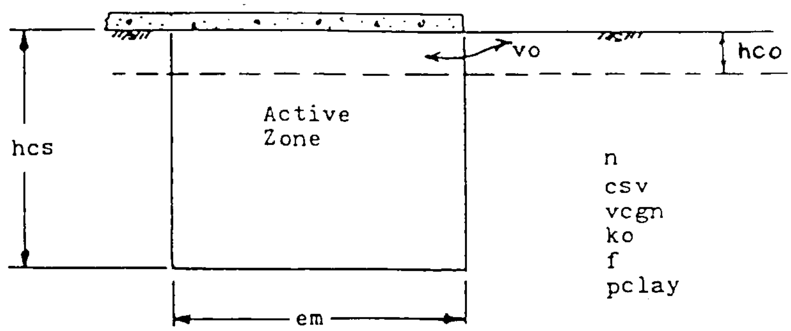
$$cm = \frac{lb}{in^2} \ x \ 70.37 \left(\frac{cm}{psi}\right)$$

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

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

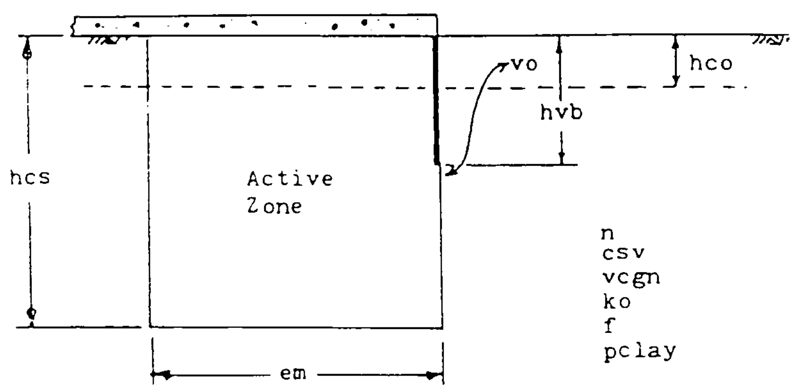
$$cm = psf \ x \ 0.4887 \left(\frac{cm}{psf}\right)$$

<u>CASE 1</u>



em	=	edge moisture variation distance in feet
hcs	=	depth to constant suction in feet
vo	=	velocity of moisture flow in inches/month
n	=	velocity distribution factor
csv	=	constant suction value at depth hcs in pF
vcgn	=	volume change guide number
ko	=	lateral earth pressure coefficient
f	=	vertical volume change coefficient
hco	=	depth in feet above which no volume change correction is made
pclay	=	percent clay in a layer

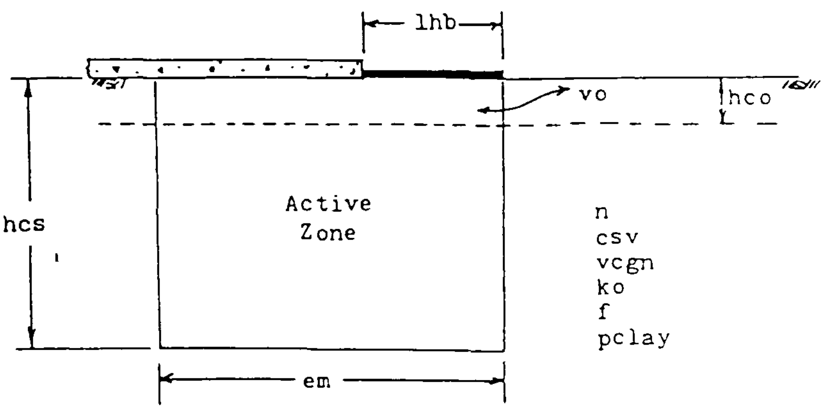
<u>CASE 2 - VERTICAL MOISTURE BARRIER</u>



em	=	edge moisture variation distance in feet
hcs	=	depth to constant suction in feet
hvb	=	depth of vertical moisture barrier in feet
vo	=	velocity of moisture flow in inches/month
n	=	velocity distribution factor
csv	=	constant suction value at depth hcs in pf

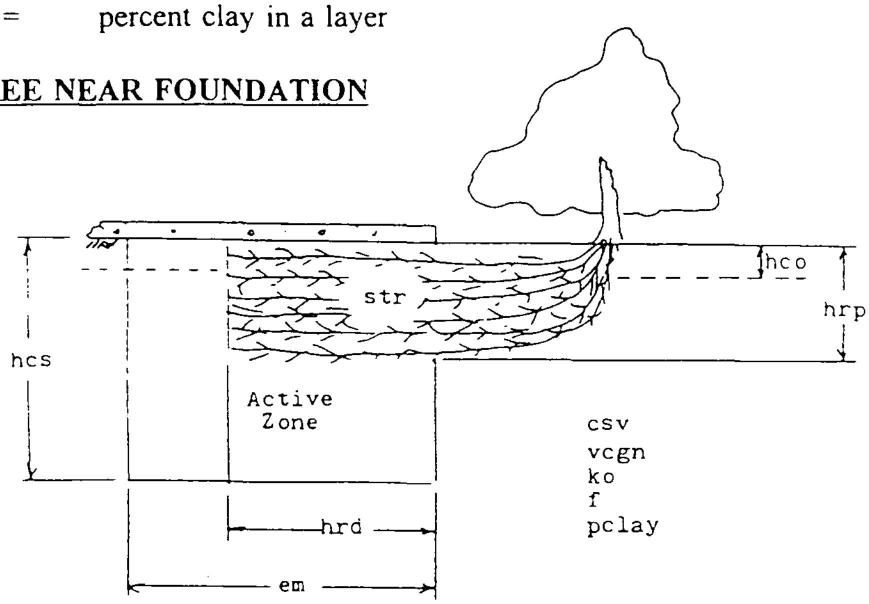
vcgn	=	volume change guide number
ko	=	lateral earth pressure coefficient
f	=	vertical volume change coefficient
hco	=	depth in feet above which no volume change correction is made
pclay	=	percent clay in a layer

CASE 3 - HORIZONTAL MOISTURE BARRIER



em	=	edge moisture variation distance in feet
hcs	=	depth to constant suction in feet
lhb	=	length of horizontal moisture barrier in feet
vo	=	velocity of moisture flow in inches/month
n	=	velocity distribution factor
csv	=	constant suction value at depth hcs in pF
vcgn	=	volume change guide number
ko	=	lateral earth pressure coefficient
f	=	vertical volume change coefficient
hco	=	depth in feet above which no volume change correction is made
pclay	=	percent clay in a layer

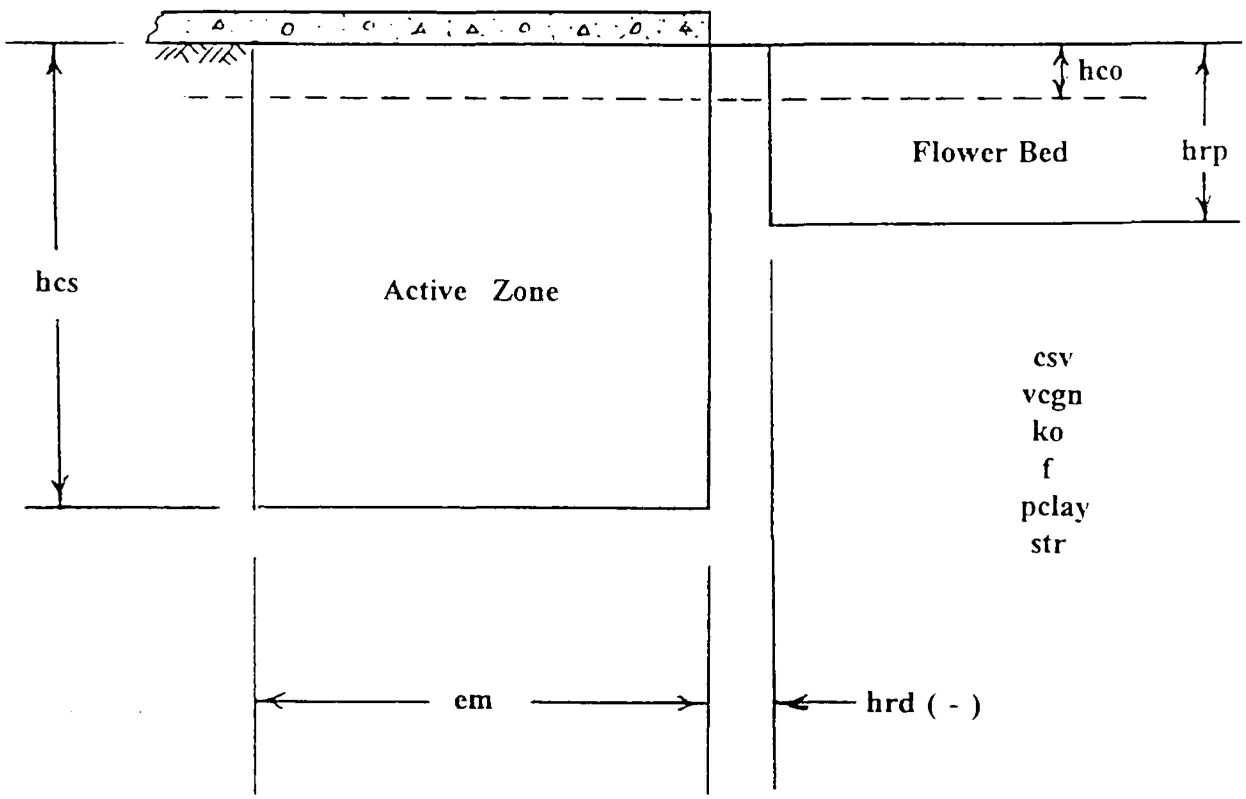
CASE 4 - TREE NEAR FOUNDATION



em	=	edge moisture variation distance in feet
hcs	=	depth to constant suction in feet
n	=	velocity distribution factor
csv	=	constant suction value at depth hcs in pF
vcgn	=	volume change guide number
ko	=	lateral earth pressure coefficient
f	=	vertical volume change coefficient
hco	=	depth in feet above which no volume change correction is made
pclay	=	percent clay in a layer
hrp	=	depth of root penetration in feet
hrd	=	horizontal distance in feet of roots from edge of foundation
str	=	suction level due to tree in pF

CASE 4A - FLOWER BED NEAR FOUNDATION

(Same geometry as a tree near a foundation - Case 4)

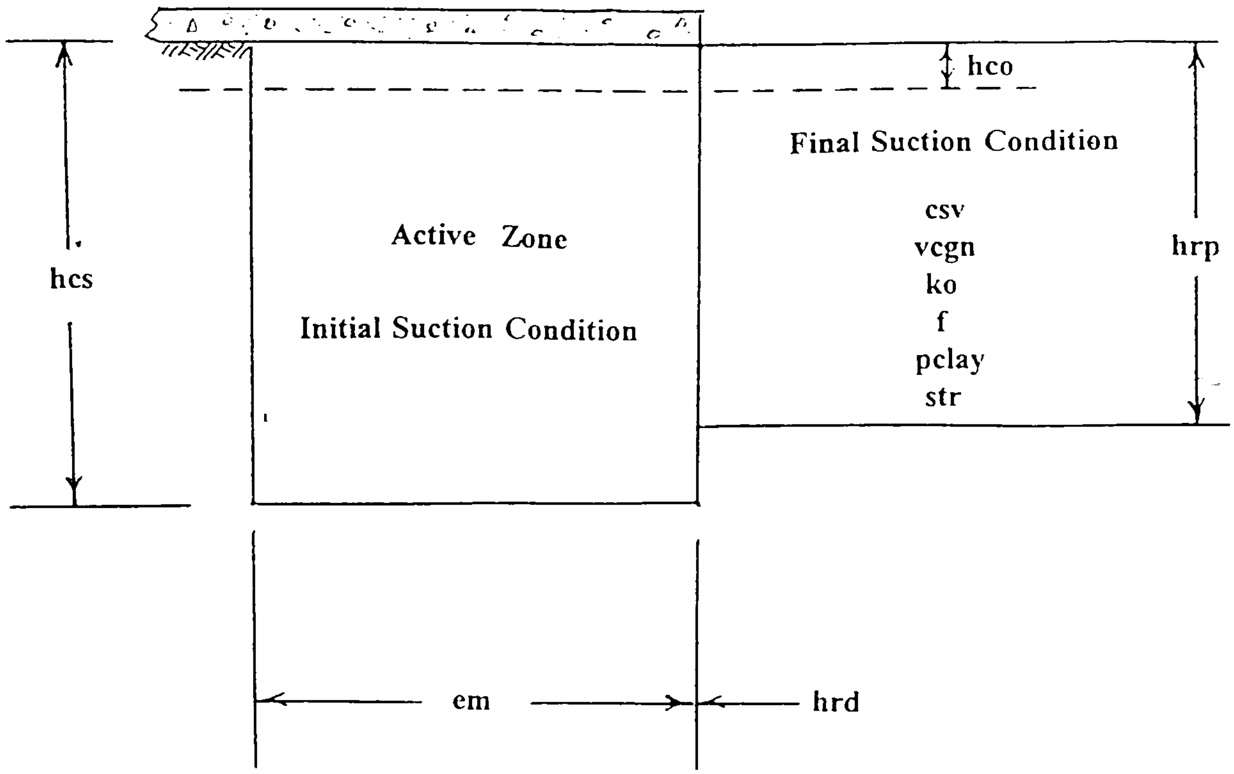


All variables the same as for a tree near foundation - Case 4- except:

hrp	=	depth of flower bed
hrd	=	horizontal distance in feet of flowerbed from edge of foundation
str	=	suction level in flower bed. Very wet is pF = 2.5. Field capacity is pF = 2.0.

CASE 4B - TOTAL HEAVE OR SHRINKAGE

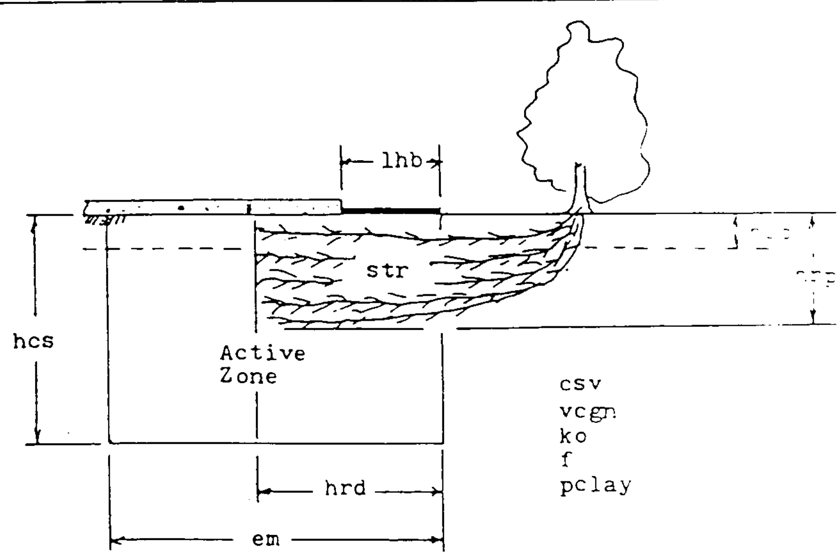
(Same geometry as a tree near a foundation - Case 4)



Same variables as for a tree near a foundation except:

csv	=	initial suction at a depth of hcs in pF
hrp	=	depth of suction change
hrd	=	o.o. Heave or shrinkage is calculated at the edge of the foundation
str	=	final suction in pF.

CASE 5 - TREE AND HORIZONTAL MOISTURE BARRIER



em = edge moisture variation distance in feet
hcs = depth to constant suction in feet
n = velocity distribution factor
csv = constant suction value at depth hcs in pF
vcgn = volume change guide number
ko = lateral earth pressure coefficient
f = vertical volume change coefficient
hco = depth in feet above which no volume change correction is made
pclay = percent clay in a layer
hrp = depth of root penetration in feet
hrd = horizontal distance in feet of roots from edge of foundation
str = suction level due to tree in pF. Wilting point is 4.5.
lhb = length of horizontal moisture barrier in feet

VELOCITY OF MOISTURE FLOW

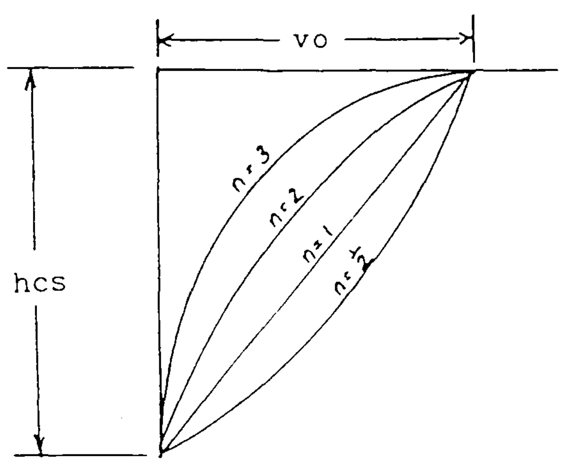
For Cases 1, 2 and 3 the estimated velocity of moisture flow, vo, must be input. If the moisture flow is outward, input a positive velocity. If the moisture flow is inward, input a negative velocity.

VELOCITY DISTRIBUTION FACTOR

The velocity of moisture flow at depth z is obtained from the relationship

$$v = vo\left[\frac{hcs - z}{hcs}\right]^n$$

The figure below shows the relationship between the velocity of moisture flow and the depth to constant suction. A value of n = 0.5 should always be used unless there is a specific reason to use another value. The value of 0.5 is more realistic and, if anything, somewhat conservative.



If the magnitude of the velocity of moisture flow is too large, the suctions nearer to the edge of the foundation will be greater than 6 pF (outflow) or less than 2 pF (inflow). A message will appear on the screen. If it is an outflow problem, the message is:

> Sorry, your estimated velocity of moisture flow causes suction value to exceed -1000000 (6 pF). Please enter a SMALLER VELOCITY OF MOISTURE FLOW, vo (in/month).

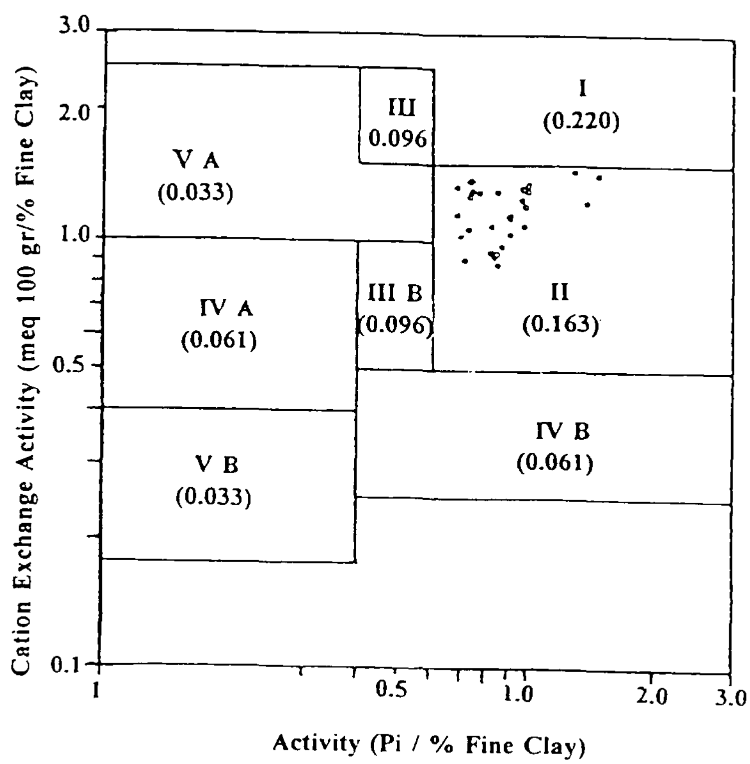
If it is an outflow problem the message is:

> Sorry, your estimated velocity of moisture flow causes suction value to drop below -100 (-2 pF). Please enter a SMALLER VELOCITY OF MOISTURE FLOW, vo (in/month).

If either of the above messages appears, input a smaller magnitude of velocity of moisture flow and press RETURN. If the message appears again, continue to input a velocity smaller than the previous one until the message no longer appears.

VOLUME CHANGE GUIDE NUMBER

The volume change guide number can be obtained from the chart below:



PI (%) = liquid limit - plastic limit

$$\% \text{ Fine Clay} = \frac{\% \text{ Passing 2 micron size}}{\% \text{ Passing \#200 size}}$$

$$\text{Cation Exchange Capacity} \approx (PL\%)^{1.17}$$

$$\text{Activity Ratio } A_c = \frac{PI (\%)}{\% \text{ Fine Clay}}$$

$$\text{Cation Exchange Activity, } CEA_c = \frac{\text{Cation Exchange Capacity}}{\% \text{ Fine Clay}}$$

LATERAL EARTH PRESSURE COEFFICIENT, ko

The recommended lateral earth pressure coefficients are:

ko	=	0	many cracks in the soil
ko	=	⅓	soil moisture decreasing
ko	=	⅔	soil moisture increasing
ko	=	1	cracks are closed tightly

VERTICAL VOLUME CHANGE COEFFICIENT, f

The recommended vertical volume change coefficients are:

f = 0.5	soil moisture decreasing
f = 0.8	soil moisture increasing

HORIZONTAL DISTANCE OF ROOTS FROM EDGE OF SLAB

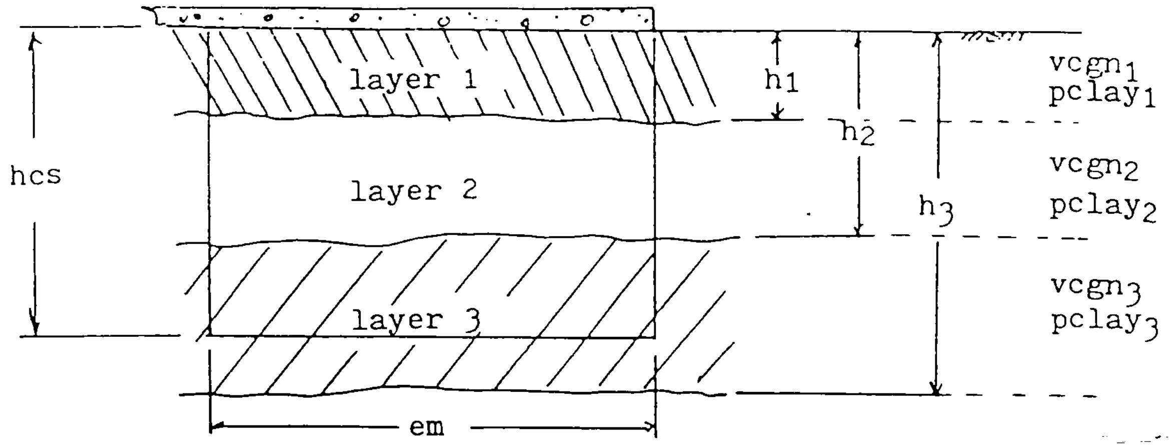
If the roots extend under the foundation, the value is positive. If the roots extend just to the edge of the foundation, the value is zero. If the roots do not extend to the foundation, the value is negative.

For Case 5 (trees and a horizontal moisture barrier) the distance is measured from the outside edge of the moisture barrier EXCEPT for the case where the roots extend under the barrier but not under the foundation itself. In this case, the distance is measured from the edge of the foundation and is positive.

SOIL LAYERS

A maximum of six soil layers can be considered. Depth to the bottom of each layer is measured from the ground line. Note that the depth to the bottom of the last layer must be greater than the depth to constant suction.

SOIL LAYERS (a maximum of six layers)



NOTES ON PROGRAM OPERATION

Once you initialize the program you will see a title screen with the program name and version number. After pressing any key to continue, you will see the Main Menu. The program automatically returns to this menu upon completion of any function.

Press "1" to input data for a new problem. VOLFLO is user-friendly in several respects:

- Entry screens are presented as logical groups by input type
- The cursor automatically jumps to the first input prompt. Enter the value desired and press the ENTER key.
- Each prompt includes the proper units.
- If you make an entry error prior to pressing ENTER, just backspace, correct and press ENTER.
- If you don't realize that you've made an error until after you've pressed ENTER, don't worry. You will have as many chances as you need to correct any entry. Proceed with entering data. After the last input item on the screen, a message will appear at the bottom of the screen asking if all of the values are correct. A "Y" response will take you to the next input screen while a "N" will reposition the cursor at the first input. Just press ENTER to accept inputs which are correct and type over incorrect ones.

After the last data entry screen, a save screen will prompt you for the name of the file to store your input data. Respond with the full path name. For a single drive computer, an appropriate response might be PROB1.IN. For a dual drive machine, the response might be B:PROB1.IN. With a hard disk you might type C:\PTI\VOLFLO\PROB1.IN. Note that file names are limited to eight characters and extensions to three characters.

Even if you've already saved a file to disk, it's not too late to change the data. After saving, the program returns to the Main Menu. By pressing "4" you can review and/or edit all of the values. As before, press ENTER to accept a value. Type over a value and press ENTER for a value you wish to change.

After you are satisfied with the input, type "3" to perform the calculations. Upon completion the program will prompt for a file name for storage of the output data. Again, provide the full pathname and extension as outlined above. This feature is useful in that it insures that archive copies of your output files are available for future reference.

You can obtain hard-copy results on your Epson/IBM compatible dot matrix printer by pressing "5" at the Main Menu.

Press "6" to exit to DOS.

EXAMPLE PROBLEMS INCLUDED

The program disk contains input and output files for 18 example problems. To run a sample problem, press "2" at the Main Menu. At the FILE NAME: prompt enter "A:SAMPLE1.IN". The Main Menu will reappear. If you wish to review the input data, press "2". Press "3" to perform calculations. A new A:SAMPLE1.OUT file will over-write the existing one and the program will return to the Main Menu.

As an alternative, just press "5" and obtain hard copy from the existing A:SAMPLE1.OUT file.

NOTE ON ENTERING REMARKS

The remarks prompt will not accept commas. If you inadvertently enter a comma the message "Redo from start?" will appear on the screen. Re-type your remarks without the commas. Otherwise you must restart the program.

PAPER NO. 5

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 x 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 x 10^6 dyne/cm^2
1 cm H_20	=	9.806 x 10^{-4} Bars
	=	9.678 x 10^{-4} Atm
	=	1.422 x 10^{-2} psi
	=	9.806 x 10^{-2} kPa
	=	9.806 x 10^2 dyne/cm^2
1 psi	=	6.895 x 10^{-2} Bar
	=	6.805 x 10^{-2} Atm
	=	70.314 cm H_20
	=	6.895 kPa
	=	6.895 x 10^4 dyne/cm^2
1 kPa	=	1.000 x 10^{-2} Bars
	=	9.869 x 10^{-3} Atm
	=	0.145 psi
	=	10.198 cm H_20
	=	1.000 x 10^{-4} dyne/cm^2

PAPER NO. 6

TITLE: The Prediction of Total Heave Using Soil Suction Profiles, Atterberg Limits, Hydrometer, and Filter Paper Suction Measurements.

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4. Name of Method
None

5. Technical Literature

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Uniform Building Code
PTI Manual

1.0 INTRODUCTION

The prediction of total heave in expansive soil is of most use when applied to the design of structures resting on or in the expansive soil. These structures include slabs, drilled shafts, canal linings, pipelines, retaining walls and basement walls, and other such structures. Of interest to the designer is to have an accurate idea of the maximum movement or differential movement that must be withstood by the design.

Total heave is a transient phenomenon. It is the movement of the surface of an expansive soil from one condition to another. It is necessary, therefore, to identify both an initial and a final condition in order to predict a total heave. Differential movement is the difference in two such predictions which occur simultaneously. For the most part, differential movement is of most interest to designers although vertical foundation elements must be designed taking the total heave with depth into account.

Expansive soils respond to changes in mechanical stress (or total stress) and to moisture stress with changes in volume and shape. If one of these is held constant, the soil responds to changes in only one of the types of stress. The coefficient of volume change which is the ratio of the volume strain to the logarithm of the stress ratio (final stress divided by the initial stress) is different for the two types of stress applied.

Laboratory tests in which both types of stress change require an intimate knowledge of the compressibility, confining pressure, and suction-water content relations of the soil being tested in order to be able to interpret the test. An example of this is the swell pressure test in any of the modes it is run. During this test there is both a change of suction, Δh, a change of vertical pressure, $\Delta\sigma_1$, and a change of confining pressure, $K_o\Delta\sigma_1$, where K_o is the coefficient of lateral pressure. It can be shown that if the net volume change is zero, the relation between Δh and $\Delta\sigma_1$, is as follows:

$$\Delta\sigma_1 = \Delta h \frac{3K}{(1+2K_o)} \bullet \frac{n[1-n(\frac{\partial S}{\partial\theta})]}{\theta\,(\frac{\partial h}{\partial\theta})} \tag{1}$$

where:

K = the bulk modulus of the soil.
n, S = the porosity and degree of saturation, respectively.
θ = the volumetric water content.
$\frac{\partial h}{\partial\theta}$ = the slope of the suction-vs-volumetric water content curve.
Δh = the change of suction that results in the measured swell pressure, $\Delta\sigma$.

The "swell pressure" measured depends upon all of these characteristics. If any of these varies from one test to the next on the same soil, it is impossible to obtain consistent results with the test.

A desirable alternative is to impose a volume change on the soil due to the change of only one of the stresses and measure the resulting volume change coefficient. The volume change coefficients are γ_h and γ_σ.

a. Suction Test (Zero Mechanical or Total Stress change).

$$\gamma_h = \frac{\Delta V/V}{\log_{10}(\frac{h_f}{h_i})} \tag{2}$$

b. Compression Test (Zero Suction Change).

$$\gamma_\sigma = \frac{\Delta V/V}{\log_{10}(\frac{\sigma_f}{\sigma_i})} \tag{3}$$

In which:

σ_i, σ_f = the initial and final values of the applied mean principal stresses.

h_i, h_f = the initial and final values of the applied suctions.

γ_σ = the volume compression index.

γ_h = the suction compression index.

The former is related to the commonly used compression index (C_c) by:

$$\gamma_\sigma = \frac{C_c}{1+e_0} \tag{4}$$

e_o = the initial void ratio.

There is a difficulty with the compression test and that is that as the volume changes, so does the volumetric water content, by an amount, $\Delta\theta$. The suction must change by an amount, $\frac{\partial h}{\partial\theta}\cdot\Delta\theta$, and there is a resulting additional volume change that occurs concurrently with the compression. Without knowing the θ and $\frac{\partial h}{\partial\theta}$ of the soil during the test as well as the confining pressures, it is impossible to determine the correct value of γ_σ.

The only test in which a correct value can be determined is the total stress free suction test which produces γ_h. There is a relation between γ_σ and γ_h which is:

$$\gamma_\sigma = \gamma_h \frac{1}{\left[1+\dfrac{h_i}{\theta_i\left(\dfrac{\partial h}{\partial\theta}\right)}\right]} \tag{5}$$

where: h_i, θ_i = the initial suction and volumetric water content.

$\frac{\partial h}{\partial\theta}$ = the slope of the suction-vs-volumetric water content curve.

For this reason, the prediction of heave which is based upon the suction compression index (due to suction change) is one that is more fundamental than any other. Measurement of γ_σ, the volume compression index, in a compression ring inherently provides results that require additional tests in order to determine fundamental material properties.

The heave prediction method is based upon the following set of equations:

$$\left(\frac{\Delta v}{v}\right) = -\gamma_\pi \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_\pi \log_{10}\left(\frac{\pi_f}{\pi_i}\right) \tag{6}$$

where: h_i, h_f = the initial and final matrix suction values.
σ_i, σ_f = the initial and final mean principal stress values.
π_i, π_f = the initial and final values of osmotic suction values.
γ_h, γ_σ, γ_π = the compression indexes for matrix suction, mechanical (or total) stress, and osmotic suction.
$\frac{\Delta v}{v}$ = the volume strain at any depth.

The mean principal stress values require a knowledge of the lateral earth pressure coefficient. The mean principal stress is given by:

$$\sigma = \sigma_z\left(\frac{1+2Ko}{3}\right) \tag{7}$$

K_o = the lateral earth pressure coefficient which varies between o when the soil mass is cracked and ranges upward to as high as the passive earth pressure coefficient when the cracks are closed and the suction (matrix or osmotic) is becoming less negative.

The lateral earth pressure coefficient is approximately equal to its elastic value when the suction is at a value that is in equilibrium with the ambient moisture conditions. This value is controlled either by the climatic evapo-transpirative moisture balance or by a high water table.

The "initial mean principal stress", σ_i, is that stress level below which no overburden correction must be applied, and may be considered to be a material property. It has been found to correspond to the mean principal stress at a depth of 40 cm.

The vertical strain, ΔH/H, at any depth is a fraction of the volume strain, Δv/v. The fraction, f, is termed the "crack fabric factor" and the vertical strain is given by:

$$\frac{\Delta H}{H} = f\left(\frac{\Delta V}{V}\right) \tag{8}$$

The lateral strain is given by:

$$= \left(\frac{1-f}{2}\right)\frac{\Delta V}{V} \tag{9}$$

It is notable that Dr. Eulalio Juarez-Badillo (*) has found the following expression for K_o, the lateral earth pressure coefficient when the soil is at its equilibrium suction condition:

$$K_o = \frac{\dfrac{1}{\sin\phi'} + \dfrac{\mu}{\gamma_\sigma} - 1}{\dfrac{1}{\sin\phi'} + \dfrac{\mu}{\gamma_\sigma} + 1} \tag{10}$$

where: μ = the Poisson's ratio of the soil.
γ_σ = the total stress compression index.
ϕ' = the effective friction angle of the soil.

The vertical heave, Δ_1 is the sum of the strains, ΔH/H, multiplied by the increment of depth, ΔZ, to which they apply.

$$\Delta = \sum_{i=1}^{n}\left(\frac{\Delta H}{H}\right)_i \cdot \Delta Z_i \tag{11}$$

It is found in computing the volume strains and vertical strains that a depth is reached below which the volume change term for the mean principal stress ratio exceeds in magnitude and is opposite in sign to the volume change term for the matrix suction ratio. At that depth, the strain energy released (or taken on) by the water phase is equal to the strain energy that can be stored in (or released from) the soil structure. Below that depth, there is no more volume change or vertical movement, even though there may be changes in suction and in mean principal stress. The depth

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*Personal communication

determined in this way is the depth of the active zone. As can be seen, the depth of the active zone is dictated by several factors including the lateral earth pressure coefficient, K_o; the crack fabric factor, f, and the initial and final values of suction with depth. Generally, the smaller K_o is, and the larger the suction ratio, the deeper is the active zone.

Estimation of the mean principal stress and suction with depth is an essential part of determining the total heave. The final mean principal stress may be estimated from the overburden pressure and any imposed foundation pressure. The initial and final suction values that are used depend upon the process for which a prediction of the total heave is desired.

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 suction levels due to drying). Steady state conditions are given by Darcy's law:

$$v = -k \left(\frac{\partial H}{\partial Z}\right) \tag{12}$$

The total head, H, is made up of the total suction, h, and the elevation head:

$$H = h + Z \tag{13}$$

The gradient of total head is:

$$\frac{\partial H}{\partial Z} = \frac{\partial h}{\partial Z} + 1 \tag{14}$$

Solving for the change of suction as a function of the change of elevation gives:

$$\partial h = -\partial Z \left(1 + \frac{v}{k}\right) \tag{15}$$

Use of Gardner's equation for the unsaturated permeability gives:

$$\Delta h = -\Delta Z \; [\, 1 + \frac{v}{k_o} \, (1 + a \, |h|^n) \,] \tag{16}$$

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 for the unsaturated permeability gives:

$$\Delta h = -\Delta Z \; [\, 1 + \frac{v}{k_o} \, (\frac{h}{h_o})] \tag{17}$$

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.

The velocity of water entering or leaving the soil may be estimated from Thornthwaite Moisture Index moisture balance computations.

The suction profiles for two transient states can be predicted approximately using:

$$U(Z,t) = U_e + U_o \; \exp(-\sqrt{\frac{n\pi}{\alpha}}Z) \; \cos(2\pi nt - \sqrt{\frac{n\pi}{\alpha}}Z) \tag{18}$$

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 x 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 1.

Table 1. 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 (14) shows that the equilibrium suction profile is for a vertical velocity of zero and that it has a slope of 1 cm more negative suction for every 1 cm higher in elevation.

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 tabulated below.

Table 2. Equilibrium Suction Values, U_e

	Mitchell Unsaturated Permeability, cm^2/sec		
TMI	5×10^{-5}	2.5×10^{-4}	1.0×10^{-3}
-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

The Mitchell unsaturated permeability, p, is estimated by:

$$p = \frac{\alpha\gamma_d}{|S|\gamma_w} \left(\frac{cm^2}{sec}\right)$$

where:

γ_d = the dry unit weight of the soil.
γ_w = the unit weight of water.
α = the Mitchell diffusion coefficient, cm^2/sec, which is used in Equation (18).
$|S|$ = the absolute value of the slope of the pF-vs-gravimetric water content, w line.

It is noted that the fundamental definition of P is :

$$p = \frac{ko|h_o|}{0.4343} \tag{19}$$

where:

$|h_o|$ = 200 cm for clays.

The value of α can be estimated from:

$$\alpha = 0.0029 - 0.000162(S) - 0.0122(\gamma_h) \tag{20}$$

The value of S is negative and can be estimated from:

$$S = -20.29 + 0.1555\,(LL\%) - 0.117\,(PI\%) + 0.0684\,(\% - \#200) \tag{21}$$

where:

LL = the liquid limit in percent.
PI = the plasticity index in percent.
-#200 = the percent of the soil passing the #200 sieve.

The suction compression index, γ_h, is estimated using the chart in Figure 1. The activity ratio and cation exchange activity ratio are determined for the soil. The box where these two ratios intersect on the chart gives the volume change guide number, γ_o. The table gives the volume change guide number that corresponds to the different regions on the chart. The suction compression index, γ_h, is related to the guide number by:

$$\gamma_h = \gamma_o \times \frac{(\% - 2\mu)}{(\% - \#200)} \tag{22}$$

for soils which pass entirely through the #200 sieve. For soils which have particles larger than the #200 sieve, no such correlations have been developed.

The activity ratio is:

$$AC = \frac{\dfrac{PI\ (\%)}{(\% - 2\mu)}}{(\% - \#200)} \tag{23}$$

The cation exchange activity is:

$$CEAC = \frac{\dfrac{CEC\ \dfrac{milliequivalents}{100\ gms\ of\ soil}}{(\% - 2\mu)}}{(\% - \#200)} \tag{24}$$

The cation exchange capacity of the clay may be measured directly using simple equipment such as a spectrophotometer, or may be estimated as closely as is needed by the following empirical relationship:

$$CEC \cong (PL\%)^{1.17} \tag{25}$$

where:

PL = the plastic limit in percent.

Heave (or shrinkage) from a present condition in the soil as uses 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 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.

This lengthy introduction was necessary to provide a theoretical and practical background to the method for computing heave addressed here. In the following sections, the laboratory and field test procedures and the analysis will refer to the concepts, equations, and empirical relations.

The tables of U_e and U_o and the empirical relations for S, α, and p were developed in a research project for the Texas Department of Transportation, and represent data from several soils sampled around Texas in several different climatic zones found in the state. Laboratory tests, field observations at over a dozen sites, and over six hundred runs with a calibrated two-dimensional finite element coupled transient moisture flow-and-elasticity computer programs were made in matching the field observations and arriving at these relationships.

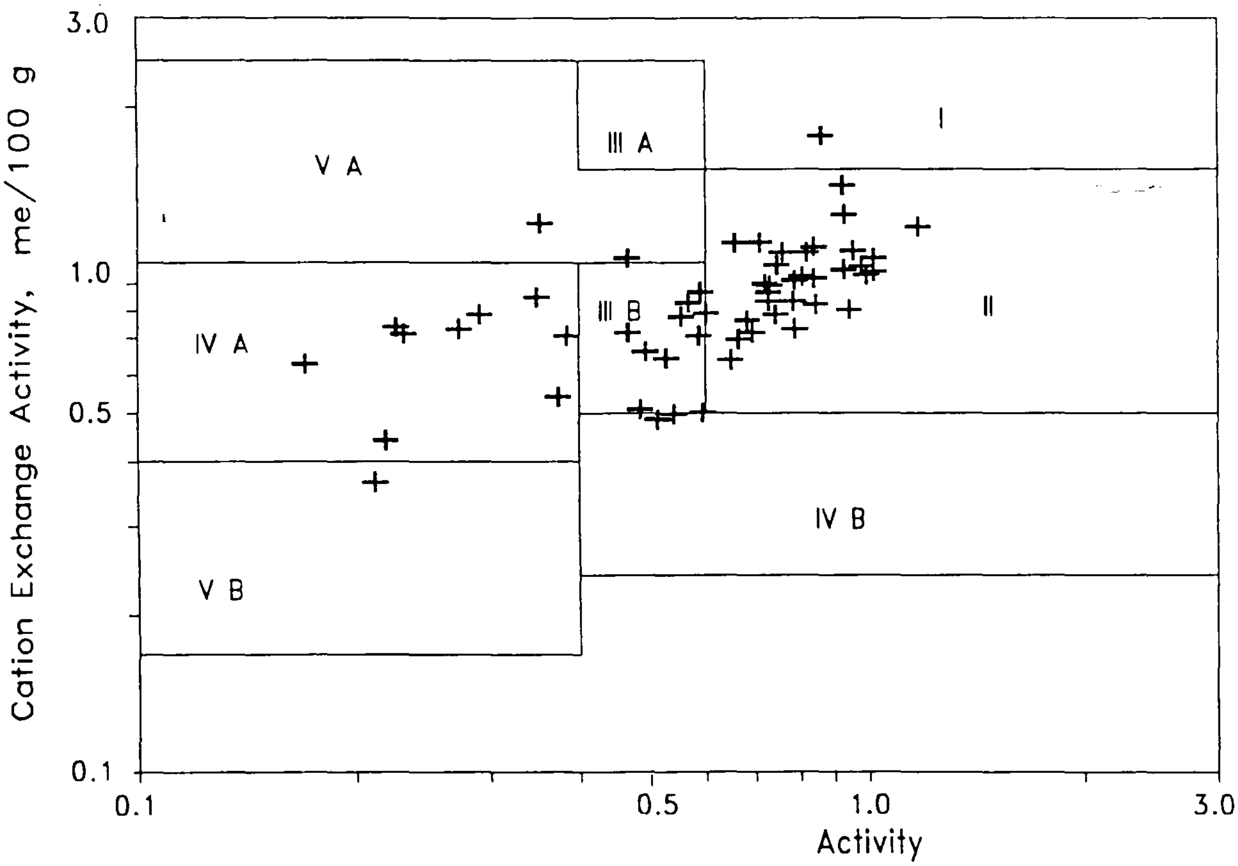

Figure 1. Chart for the Prediction of Suction Compression Index.

Table 3. Volume Change Guide Numbers

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

It is not intended to describe the computer program as the heave prediction method proposed here. Instead, it is the simplified method that makes use of all of the empirical relationships and the simplest of laboratory tests that are used in those relationships. It would be incorrect, however, to assume that these results are not supported by a sound theoretical foundation. This lengthy introduction was written to outline the theoretical background which underlies this method.

2.0 LABORATORY TEST PROCEDURES

No descriptions will be given of laboratory tests which are well-known standards. Those which are not will be described in appendices attached to this document.

2.1 Description of Laboratory Equipment

The laboratory testing equipment needed for this method are the following:

a. Atterberg limits equipment.
b. Hydrometer equipment.
c. Specific gravity of soil equipment (for use with the hydrometer).
d. Oven (to determine water content).
e. Filter paper.
f. Balance (capable of weighing $\pm$ 0.0001 gm).

g. Tares (for soil and filter paper water content).
h. Vacuum desiccator equipment (for osmotic suction measurements).
i. Calipers on density rings (for dry unit weight).
j. Spectrophotometer and centrifuge for cation exchange capacity (optional).

2.2 <u>Preparation of the Specimen</u>

There is no need to describe the preparation of specimens to measure water contents, dry unit weight, and hydrometer tests. Samples placed in vacuum desiccators should be placed on glass rather than metal since the latter causes an error in the measured suction. The description of samples tested with filter paper will be described in Appendix A.

2.3 <u>Laboratory Testing Procedure</u>

All laboratory testing required by this method are simple, standard tests. Unless the suction is controlled by a high water table or osmotic suction is present, or there is a desire to predict total heave from some current measured condition there is no need to measure suction to predict the heave. Under normal circumstances, it can be predicted with sufficient accuracy.

In order to estimate the volume change properties, diffusion coefficient, unsaturated permeability, and slope of the suction-vs-water content curve of the soil in place, it is necessary to determine the properties as shown in the following table, Table 4.

Table 4. Tests Needed to Calculate Soil Properties

Property	LL	PI	PL	(%-#200)	(%-2_μ)	w*	γ_d*
	Laboratory Tests Needed						
γ_h		x	x	x	x		
$\gamma\sigma$		x	x	x	x		
S	x	x		x			
α	x	x	x	x	x		
p	x	x	x	x	x	x	x
$\frac{\partial h}{\partial \Theta}$	x	x		x			x
Θ						x	x
Uo	x	x	x	x	x		x
Ue	x	x	x	x	x		x
h_i, h_f	x	x	x	x	x		x

*Note: w = Gravimetric water content
γ_d = Dry unit weight of the soil

It is noted that the relation between γ_h and γ_σ requires a knowledge of the initial suction, h_i; the initial volumetric water content, Θ_i; and the slope of the suction-vs-volumetric water content curve, $\partial h/\partial\Theta$. However, this relation can be simplified considerably. It is found that the slope is determined by:

$$\left(\frac{\partial h}{\partial \theta}\right)_i = \frac{1}{0.4343}\,\frac{S\,\gamma_w}{\gamma_d}\,h_i \tag{26}$$

and the term which relates γ_h to γ_σ is given by:

$$\frac{h_i}{\theta_i\left(\frac{\partial h}{\partial \theta}\right)_i} = \frac{0.4343}{S\,w_i} \tag{27}$$

This term should be a negative number.

As can be seen from the table, all of the predictions can be made once the

Atterberg limits, the percents passing the #200 sieve and 2 micron size, the water content and the dry unit weight of the soil are known.

2.4 Data Collection and Reduction

No special forms, data collection, or data reduction are needed other than to follow the formulas for γ_h, γ_σ, S, α, and p given in the introductory section. The following equations must be used in sequence to determine the material properties.

1. Suction compression index, γ_h
   a. Equation 24 Ac
   b. Equation 26 CEC
   c. Equation 25 CEAc
   d. Figure 1, Table 3 γ_o
   e. Equation 22 or 23 γ_h

2. Mean Principal Stress Compression Index, $\gamma\sigma$
   a. Equation 28 $h/\theta \left(\frac{\partial h}{\partial \theta}\right) = \frac{0.4343}{S\, w_i}$
   b. Equation 5 γ_σ

3. Unsaturated Permeability, p
   a. Equation 21. S
   b. Equation 20. α
   c. Equation 19. p

4. Wet and Dry Suction Profiles
   a. Table 1, Table 2. Ue, Uo
   b. Equation 18. U(z,t) or alternatively
   c. Equation 16 or 17

5. Volume Change with Depth
   a. Equation 7. σ
   b. Equation 6. $\Delta v/v$
   c. Equation 8. $\Delta H/H$ (vertical strain)
   d. Equation 11. Δ (vertical heave)

The use of these tables and equations in sequence produces the predicted heave.

2.5 Presentation of Test Results

The computed values of γ_h, γ_σ, p, and initial and final suction values should be tabulated with depth for each soil stratum. The volume change components due to suction and total stress, the net volume change between the two, the vertical strain, the incremental heave (or shrinkage), and the total heave (or shrinkage) with depth should also be tabulated for each depth increment.

3.0 FIELD TEST PROCEDURE

There is no need for field tests with this procedure although there is a need to take samples from each soil stratum and to run the tests necessary on each of the samples to determine the soil properties and the suction, volume change and heave profiles.

It is also necessary to observe the samples carefully to determine the presence of root fibers, cracks and their orientation, discoloration, and precipitated salts. Calcareous salts indicate zones of evaporation and gypsum crystals indicate the presence of sulfates which may, in the presence of water and calcium, form expansive crystals of etringite and thaumasite.

Root fibers and crack fabric are good indicators of the depth to which water penetrates each wet season, and can be used as a measure of the seasonally moisture active zone. The volumetrically active zone will necessarily be shallower that this for the reasons given in the introduction.

These observations are very valuable in collaborating the computed results of the suction and heave profiles and contribute to reasonable values of the assumed quantities of K_o and f, the lateral earth pressure coefficient and the crack fabric factor.

These observations should be recorded in each boring log made at each site. As a general rule, borings should be made to a depth equal to at least twice the depth of the estimated moisture active zone. When the foundation element to be designed is a drilled pier, the borings should be taken to the above depth or to 50 percent deeper than the expected depth of the pier, whichever is deeper.

4.0 ANALYSIS TO COMPUTE TOTAL HEAVE

4.1 Theory and Assumptions Related to the Computations

The theory was presented in the Introduction section. The only assumptions that need to be made to estimate the heave (or shrinkage) profile and the total heave (or shrinkage) are the lateral earth pressure coefficient, K_o, and the crack fabric factor, f. Values of K_o which have proven to give good results are the following:

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.

Values of f, the crack fabric factor, which have been back-calculated from field observations are as follows:

f = 0.5 when the soil is drying out.
f = 0.8 when the soil is wetting up.

The remainder of the theory is explained in the Introduction section.

It should be recognized that the process of soil heaving is one in which there is an energy balance. The energy to lift the weight of soil above a point comes from the release of the Gibbs free energy of the water in tension. The released energy is stored in the soil mass resulting in an increase in volume, where that is possible, or an increase of mean principal stress (principally lateral stress) where it is not. No volume change is possible where the released soil water energy is exceeded by the ability of the soil mass to store the released energy as recoverable strain energy by an increase of total stress. This fact is accounted for in the computations by treating the volume change component due to total stress as an "overburden correction factor". It is always opposite in sign to the volume change component due to change of suction. Because of the energy balance between the two, it can never exceed in magnitude the volume change component due to a change of suction. It is this fact which allows this method to compute the depth below which no volume change (or upward movement) takes place.

To complete this discussion, it should be added that shrinkage also obeys the energy balance. In this case, an increase in suction increases the energy stored in the soil water. The increase is made up to the extent possible by a decrease of stored strain energy and potential energy in the soil mass. In the computations, the shrinkage volume component due to the increase in suction is countered by an increase of the volume component due to a reduction in total stress. This return of energy to the soil water cannot exceed what it can store and so, once more, the volume change component due to total stress cannot exceed in magnitude that due to a change of suction. This allows the depth to which shrinkage displacements can occur to be computed.

The computational sequence is outlined in Section 2.4 on Laboratory Data Collection and Reduction. It makes use of the equations set forth in the Introduction section. The total heave (or shrinkage) is found by adding together the increments of vertical movement as given in Equation 29.

$$\Delta = \sum_{i=i}^{n} f_i(\frac{\Delta v}{v})_i \, \Delta Z_i \tag{28}$$

where:

n = the number of depth increments.

f_i = the ith crack fabric factor.

$(\frac{\Delta V}{V})_i$ = the volume strain in the ith depth increment.

ΔZ_i = the height of the ith depth increment.

The total stresses used in computing the volume strains are the vertical overburden pressure and the lateral earth pressures which are related to it by the lateral earth pressure coefficient, K_o. When surcharge or foundation pressure are to be added, any closed form or numerical means of computing the vertical and horizontal pressures may be used and added to the overburden pressures.

4.3 Sample Calculations

Sample calculations are given in Appendix B. They are intended to be simple examples and so do not include calculations of initial and final suction values. Instead, they include the calculation of the volume strain components, their net value, the vertical strain, the vertical movement increments, and the total heave or shrinkage. Several examples are given to show the effect of cracks on heave and depth of vertical movement.

4.4 Presentation of Analytical Results

The calculated results for the example problems are also presented in Appendix B.

5.0 SUMMARY COMMENTS

The total heave (and shrinkage) calculations presented here are supported by a sound theoretical development, a wide variety of field observations, and empirical relations which permit all of the material properties to be calculated from the results of simple laboratory tests. These tests are the Atterberg limits, hydrometer test, water content, dry density, and percent passing the #200 sieve, under normal circumstances. When high water tables or significant osmotic suctions are present, filter paper, a precise balance, and vacuum desiccators are needed in addition.

Very few assumptions are needed except the K_o and f-values. The fact that these are sensitive variables in predicting heave (and shrinkage) profiles with depth, means that they can be back-calculated with confidence from field observations of vertical movements. They do not need to be measured directly in order to confirm the assumed values, although direct measurement would also be of interest.

Because the method is based upon theory, it is capable of being used to make other calculations and, in fact, has been used for that purpose. Another use includes the computation of lateral earth pressures with the two-dimensional finite element program. This latter use has shown that lateral pressures can become so large as to generate passive earth pressures. Still another use is the computation of the depth of cracking due to the extraction of water by roots.

When searching for an appropriate technology for use in developing countries, this method of predicting heave strongly commends itself because of the simplicity of the testing and equipment that are needed.

PAPER NO.7

Appendix A

Measurement of Suction with

Filter Paper

Standard Test Method for Measurement of Soil Potential (Suction) Using Filter Paper[1]

This standard is issued under the fixed designation D 5298; 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 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 $\approx$ 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

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[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.

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[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:

$$h = \frac{RT}{v} \cdot \ln R_h \quad (1)$$

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/m³, 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

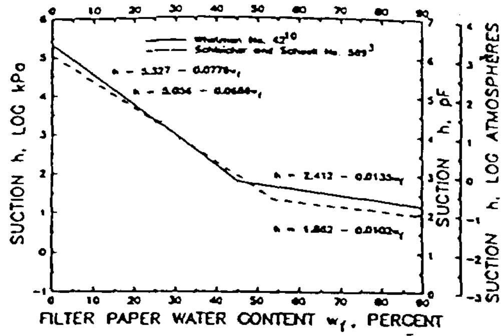


FIG. 1 Calibration Suction-Water Content Curves for Wetting of Filter Paper (3) (Coefficient of Determination r > 0.99)

NOTE 2—When the soil is not sufficiently moist, adequate physical contact between the filter paper and soil may not always be possible. This can cause an inaccurate measure of matric suction. Matric suction may be inferred by subtracting the osmotic suction from the total suction. The osmotic suction may be determined by measuring the electrical conductivity (see Test Method D 1125) of pore-water extracted from the soil using a pore fluid squeezer (6) or using Test Method D 4542; a calibration curve (7) may be used to relate the electrical conductivity to the osmotic suction.

8.3 *Filter Paper Placement*—Place an intact soil specimen or fragments of a soil sample, 115 to 230 g mass, in the specimen container. The soil specimen should nearly fill the specimen container to reduce equilibration time and to minimize suction changes in the specimen.

8.3.1 *Measurement of Total Suction*—Remove two filter papers from the desiccator and immediately place over the specimen, but isolate from the specimen by inserting screen wire, O-rings, or other inert item with minimal surface area between the filter papers and the soil, see Fig. 2(*a*). A filter paper edge should be bent up or offset slightly to hasten later removal of the filter paper from these large containers with tweezers, see 8.6.

8.3.2 *Measurement of Matric Suction*—Place three stacked filter papers in contact with the soil specimen, see Fig. 2(*b*). The outer filter papers prevent soil contamination of the center filter paper used for analysis of the matric suction. The outer filter papers should be slightly larger in 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 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

$$M_1 = M_f + M_w + T_c \quad (2)$$

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:* (5)

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:

$$M_2 = M_f + T_h \quad (3)$$

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 desiccan 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:

$$M_f = M_2 - T_h \quad (4)$$

$$M_w = M_1 - M_2 + T_h - T_c \quad (5)$$

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:

$$w_f = \frac{M_w}{M_f} \cdot 100 \quad (6)$$

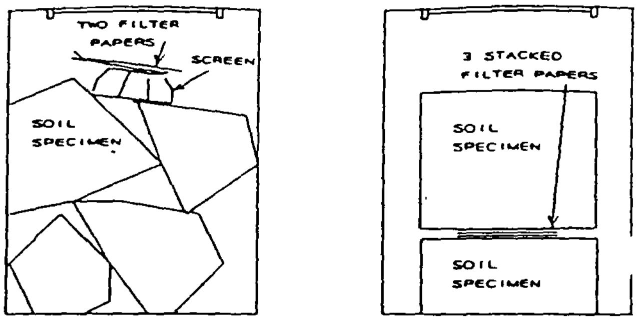
where:

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:

$$h = m \cdot w_f + b \quad (7)$$

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.

BORING NO.: ________   DATE TESTED: ________
DATE SAMPLED: ________   TESTED BY: ________
SAMPLE NO.: ________

[illegible]							
[illegible]							
[illegible]		Top / Bottom	Top / Bottom	Top / Bottom	Top / Bottom	Top / Bottom	Top / Bottom
[illegible]	T_c						
[illegible]	M_1						
[illegible]	M_2						
[illegible]	T_h						
[illegible]	M_f						
[illegible]	M_w						
[illegible]	w_f						
[illegible]	h						

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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Appendix B

Sample Calculations of Heave

and Shrinkage

(originally presented at an ASCE-sponsored seminar at the University of Houston, Houston, Texas on June 21, 1985)

PAPER NO. 8

NOTES ON VOLUME CHANGE AND FLOW CALCULATIONS IN EXPANSIVE SOILS

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JUNE 21, 1985

UNIVERSITY OF HOUSTON

Typical Suction Levels

Air Dry	-	6.0 pF	(Rel. Hum. = 50%)
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	
Soil at "Field Capacity"	-	2.0 pF	← Wet Limit for Clays = 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 = o, $\Delta h/\Delta z = -1$.

Permeability

$$k = \frac{2 \times 10^{-6}\ \mathrm{cm/sec}}{1 + 10^{-9}\ |h|^3} \ *$$

$|h|$ = absolute value of suction

* varies with soil type. – Gardner formulation. In cracked soil, Mitchell formulation is better.

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

$40^{cm} \times \gamma_t \ (^{gm}/cm^3)$

σ_f = mean pressure in (g/cm^2) 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%

2. % Fine Clay

3. Cation Exchange Capacity

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

2. $\% \text{ Fine Clay} = \dfrac{\% \text{ Passing } (-2\mu) \text{ size}}{\% \text{ Passing } (\#200) \text{ size}}$

3. Cation Exchange Capacity $\cong$ $(PL\%)^{1.17}$ meq/100 gm or $\cong (LL\%)^{0.912}$

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

5. Cation Exchange Activity, CEAC = $\dfrac{\text{Cation Exchange Capacity}}{\text{\% Fine Clay}}$

6. Volume Change Guide Number (From Chart), γ_{100}

7. Volume Change Coefficient, γ_h

$$= \text{\% Fine Clay (decimal)} \times \gamma_{100}$$

Example Problem - Equilibrium Suction Profile

No vertical flow: $v = o$

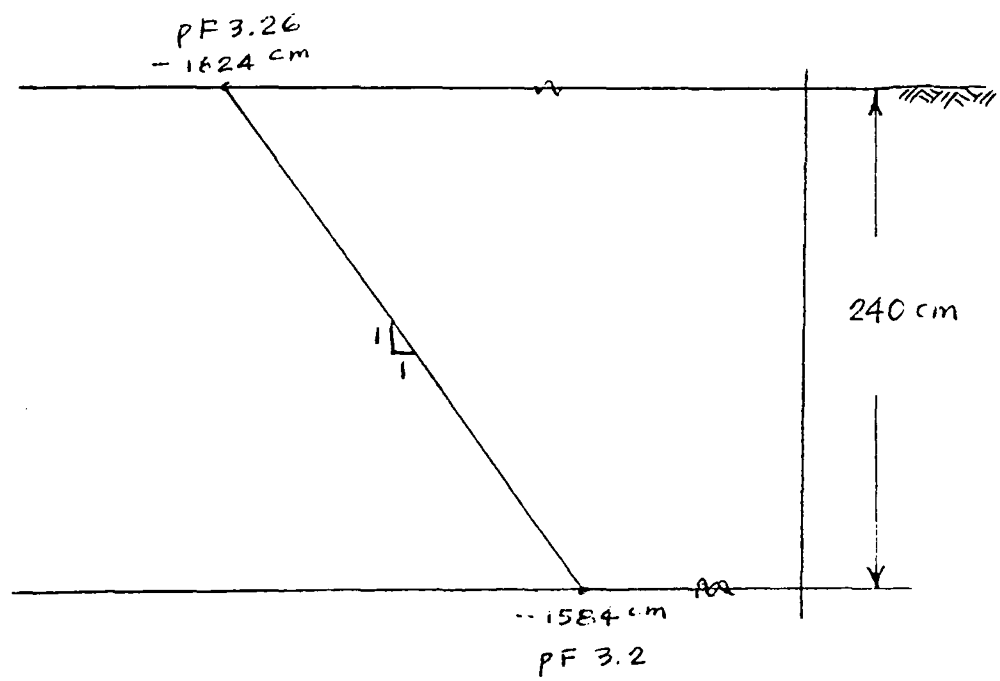
$$\frac{\Delta h}{\Delta z} = -1$$

Suction at 8 ft: pF 3.2 = -1584 cm.

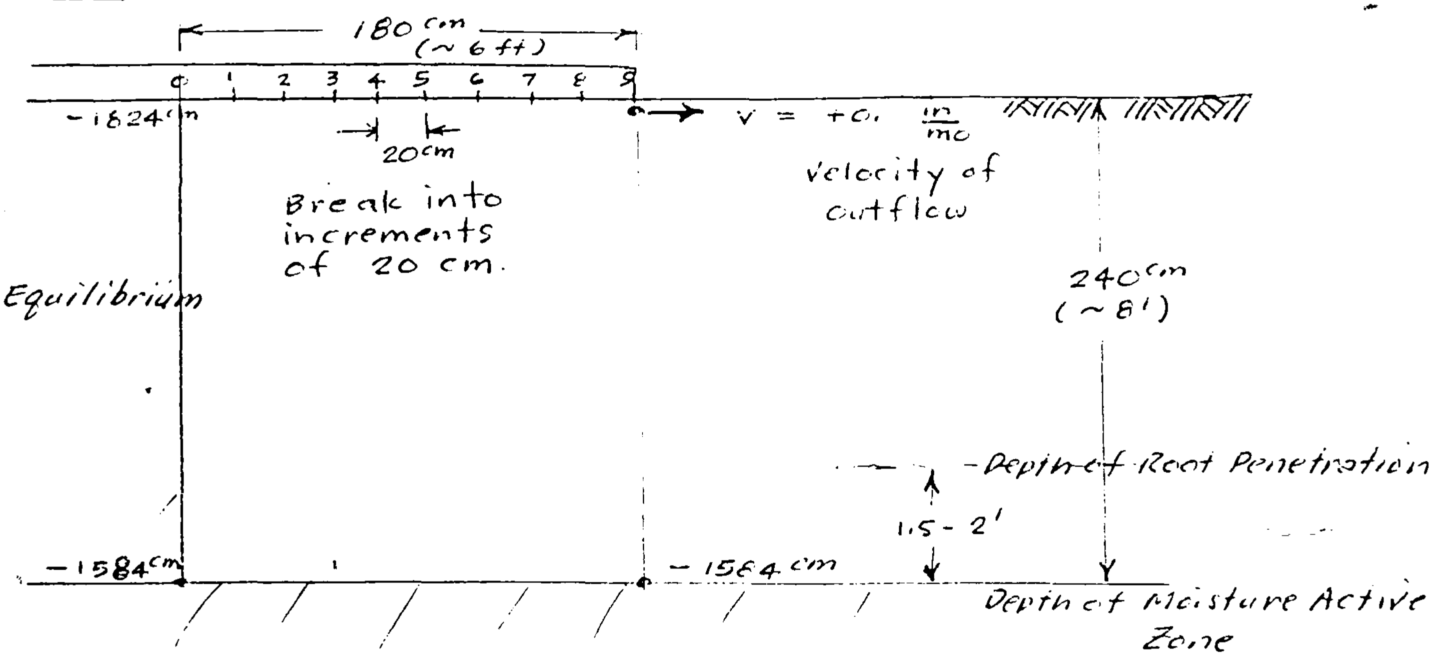
Suction at Surface: $-1584 \text{ cm} + \left(\dfrac{\Delta h}{\Delta z}\right) \times 8' \times 30 \dfrac{\text{cm}}{\text{ft}}$

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

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



Example Problem - Horizontal Flow



1. Horizontal outflow

$$\text{velocity} = +0.7 \frac{in}{mo} \cong 0.7 \times 10^{-6}\ cm/sec$$

$$\Delta h = -\frac{v}{\bar{k}}(\Delta x) = -\frac{0.7 \times 10^{-6}\ [1 + 10^{-9}|h|^3]}{2.0 \times 10^{-6}}\ (20^{cm})$$

At Station 0:

suction at station

$$\Delta h = -\frac{0.7 \times (10^{-6})}{2 \times 10^{-6}} \times (20cm) \times [1 + 10^{-9}\ (1824)^3]$$

$$\Delta h = -7\ [1 + 6.0684 \times 10^9 \times 10^{-9}] = -49.5^{cm}$$

say -50^{cm}

At Station 1

$$h_1 = -1824 - 50 = -1874^{cm}$$

Example Problem - Horizontal Flow (continued)

At Station 1 (continued)

suction at station 1

$$\Delta h = -7 \times [1 + 10^{-9}\ (1874)^3]$$

$$\Delta h = -7 \times [1 + 6.5813 \times 10^{-9} \times 10^9] = -53.1^{cm}$$

$\underline{\Delta h} \cong \underline{-53}$ ^{cm}

The calculations proceed from station to station until Station 9 is reached. The results of the calculation are tabulated below.

Station	h suction, cm	Δh, cm change of suction
0	- 1824	- 50
1	- 1874	- 53
2	- 1927	- 57
3	- 1984	- 62
4	- 2046	- 67
5	- 2113	- 73
6	- 2186	- 80
7	- 2266	- 88
8	- 2354	- 98
9	- 2452	

Example Problem - Horizontal Flow (continued)

2. Horizontal Inflow

$$\text{velocity} = -\,0.7\ \frac{\text{in}}{\text{mo}} \cong -\,0.7 \times 10^{-6}\ {}^{\text{cm}}/\text{sec}$$

$$\Delta h = -\,\frac{v}{k}\,(\Delta x) = +\,\frac{0.7 \times 10^{6}}{2.0 \times 10^{6}}\,(20^{cm})\ [1 + 10^{-9}|h|^{3}]$$

The same kind of calulations are made as in the case of horizontal outflow with the following result:

Station	h, cm suction	Δh, cm Change of Suction
0	- 1824	+ 50
1	- 1774	+ 46
2	- 1728	+ 43
3	- 1685	+ 41
4	- 1644	+ 38
5	- 1606	+ 36
6	- 1570	+ 34
7	- 1536	+ 32
8	- 1504	+ 31
9	- 1473	

Example Problem - Volume Change

$LL = 76\%$

$PL = 22\% \quad = \quad CEC = 37.2$ meq/100gm

$PI = 54$

$\%\ Clay = 60\%$

$AC = 54/60 = 0.90$

$CEAC = 37.2/60 = 0.62$

$\gamma_h|100\% = 0.163$

$\gamma_h = 0.163 \times 0.60 = 0.098$

Assume $\gamma_o = 0.098$

Dry pF $= 4.20$ (-15984cm)

Wet pF $= 2.16$ (-144cm)

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

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

$f = 0.5$ (Drying)

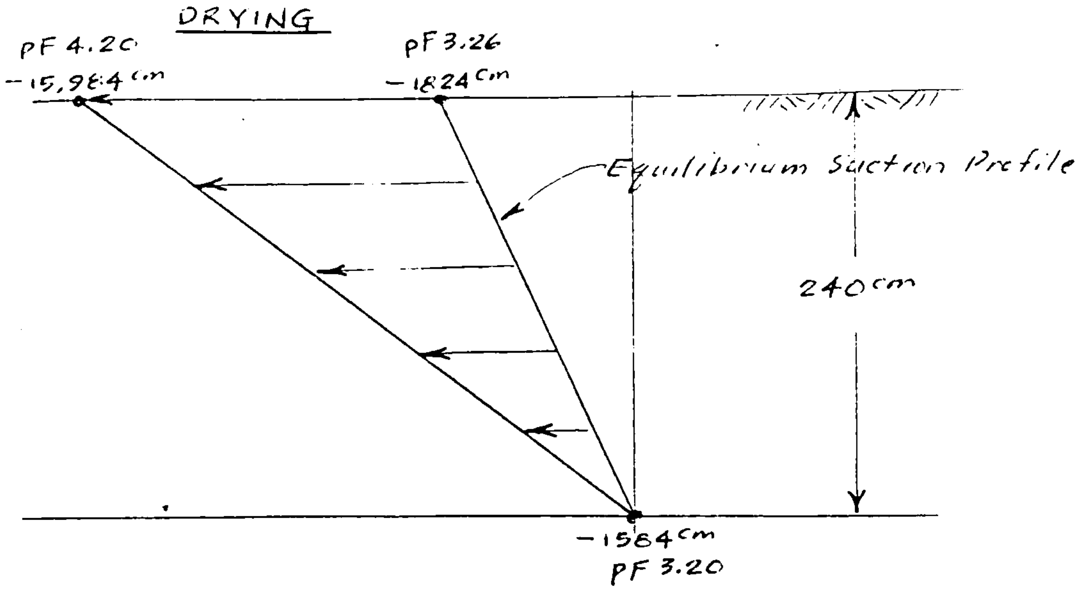
$f = 0.8$ (Wetting)

$$\sigma_i = 40^{cm} \times \gamma_t$$

$$\sigma_f = 2 \times \gamma_t \times \left(\frac{1 + 2Ko}{3}\right)$$

$Ko = 1$ (Assume)

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



Drying
$f = 0.5$

$\gamma_h' = \gamma_\sigma' = 0.098$

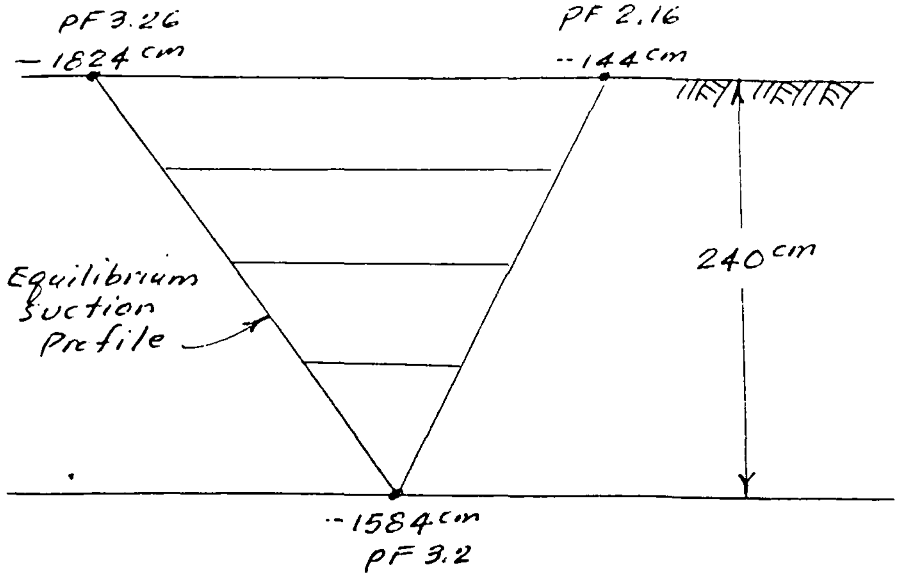
	Depth, cm	h_i, cm	h_f, cm	$-\gamma_h \log_{10}(\frac{hf}{hi})$	$+\gamma\sigma \log_{10}(\frac{\sigma f}{\sigma_i})$	$(\frac{\Delta v}{v})$	$(\frac{\Delta H}{H})$
0′	0	-1824	-15984	-0.0924	-	0.0924	0.0462
	20	-1804	-14784	-0.0895	-	0.0895	0.0448
	40	-1784	-13584	-0.0864		0.0864	0.0432
2′	60	-1764	-12384	-0.0829	0.0173	0.0656	0.0328
	80	-1744	-11184	-0.0791	0.0295	0.0496	0.0248
	100	-1724	-9984	-0.0748	0.0390	0.0358	0.0179
4′	120	-1704	-8784	-0.0698	0.0468	0.0230	0.0115
	140	-1684	-7584	-0.0640	0.0533	0.0107	0.0054
	160	-1664	-6384	-0.0572	0.0590	-	-
6′	180	-1644	-5184	-0.0489	0.0640	-	-
	200	-1624	-3984	-0.0382	0.0684	-	-
	220	-1604	-2784	-0.0235	0.0726	-	-
8′	240	-1584	-1584	0.0000	0.0000	-	-

Assume

Ko = 1

Drying (continued)

Depth, cm	$(\frac{\Delta H}{H})$	H, cm	ΔH, cm	y_m
0	0.0462	10	0.462	4,069 cm. = 1.60 in.
20	0.0448	20	0.895	
40	0.0432	20	0.864	
60	0.0328	20	0.656	
80	0.0248	20	0.496	
100	0.0179	20	0.358	
120	0.0115	20	0.230	
140	0.0054	20	0.107	
160	-		-	
180	-		-	
200	-		-	
220	-		-	
240	-		-	



Wetting
$f = 0.6$

	Depth, cm	h_i, cm	h_f, cm	$-\gamma_h \log_{10}(\frac{hf}{hi})$	$+\gamma\sigma \log_{10}(\frac{\sigma_f}{\sigma_i})$	$(\frac{\Delta V}{V})$	$(\frac{\Delta H}{H})$
0′	0	-1824	-144	-0.1081	-	0.1081	0.0865
	20	-1804	-264	+0.0818	-	0.0818	0.0654
	40	-1784	-384	-0.0654		0.0654	0.0523
2′	60	-1764	-504	-0.0533	0.0173	0.0368	0.0294
	80	-1744	-624	-0.0437	0.0295	0.0142	0.0114
	100	-1724	-744	-0.0358	0.0390	-	-
4′	120	-1704	-864	-0.0289	0.0468	-	-
	140	-1684	-984	-0.0229	0.0533	-	-
	160	-1664	-1104	-0.0175	0.0590	-	-
6′	180	-1644	-1224	-0.0126	0.0640	-	-
	200	-1624	-1344	-0.0081	0.0684	-	-
	220	-1604	-1464	-0.0039	0.0726	-	-
8′	240	-1584	-1584	0.0000	-	-	-

Assume

Ko = 1

Wetting (continued)

Depth, cm	$(\frac{\Delta H}{H})$	H, cm	ΔH, cm	y_m, cm
0	0.0865	10	0.865	4.035 = 1.59 in.
20	0.0654	20	1.308	
40	0.0523	20	1.046	
60	0.0294	20	0.588	
80	0.0114	20	0.228	
100	-	-	-	
120	-	-	-	
140	-	-	-	
160	-	-	-	
180	-	-	-	
200	-	-	-	
220	-	-	-	
240	-	-	-	

Wetting With Open Cracks

$\gamma_h = \gamma\sigma = 0.098$

wetting

f = 0.8

	Depth, cm	$-\gamma_h \log_{10}(\frac{h_f}{h_i})$	$+\gamma\sigma \log_{10}(\frac{\sigma_f}{\sigma_i})$	$(\frac{\Delta v}{v})$	$(\frac{\Delta H}{H})$	H, cm	ΔH	y_m, cm
0′	0	+0.1081	-	0.1081	0.0865	10	0.865	6.149
	20	+0.0818	-	0.0818	0.0654	20	1.308	=2.42
	40	+ .0654	-	0.0651	0.0523	20	1.046	
2′	60	+ .0533	-	0.0533	0.0426	20	.852	
	80	+ .0437	-	0.0437	0.0350	20	.700	
	100	+ .0358	-	0.0358	0.0286	20	.572	
4′	120	+ .0289	-	0.0289	0.0231	20	.462	
	140	+ .0229	-0.0066	0.0163	0.0130	20	.260	
	160	+ .0175	-0.0122	0.0053	0.0042	20	.084	
6′	180	+ .0126	- .0173	-	-			
	200	+ .0081	- .0217	-	-			
	220	+ .0039	- .0258	-	-			
8′	240	+ .0000	- .0295	-	-			

Assume

$K_o = 0$

$$\sigma_f = \frac{Z}{3} \times \gamma_t$$

$$\sigma_i = 40 \times \gamma_t$$

$$\frac{\sigma_f}{\sigma_i} = \frac{Z}{120}$$

PAPER NO. 9

Appendix C

Measurement of the Suction Compression Index

(From Appendix D, Reference 5, as
modified from References 1, 12 and 11)

CLOD TEST PROCEDURE

Measurement Procedure--Soil samples weighing 120 ± 20 g are separated from undisturbed samples and placed in 30 cm^3 (8 ounce) moisture cans, as soon after sampling as practical. This can easily be accomplished in the field if samples are extruded from the samplers. Other tests may be performed after samples are returned to the laboratory in order to vary the moisture condition to develop data for a wide range of moisture, if desired. Samples are normally wetted to three moisture contents wetter than natural and three drier by assuming a value of in situ moisture and adjusting sample moisture based on weight.

Suction measurements are made following procedures given above for filter paper suction measurements. After equilibration the filter paper and soil sample are separated. Filter paper is treated as described above, the soil sample is treated as described by the following.

Samples are weighed (W_1), followed by preparation of the sample for bulk density measurements. A wire, tag, and possibly a hair net are attached to provide a means of handling the sample. Hair nets are used only for those samples that fall apart without support. In tests involving moisture adjustments, these tare items are added before the moisture altered. A second weight is measured (W_2).

The next step involves coating the sample with Saran™ resin. The solutions used are 1:7 or 1:4 (only for coarse soils), Saran™: methyl ethyl ketone. This procedure should be performed in a well-ventilated area, preferably under a fume hood. The sequence for applying the coating is as follows:

1. For 1:4 solution: dip in liquid, dry 5 minutes; dip in again, dry 55 minutes.

2. For 1:7 solution: dip in liquid, dry 5 minutes; dip again, dry eight minutes; dip again, dry 55 minutes.

These procedures for coating are based on the Soil Conservation Service method.

Immediately weigh the sample in air and water at the end of the drying period. These weighings are designated W_3 and W_4 respectively. In the normal NMERI method, W_4 is the buoyant force exerted on the sample, which is measured directly, rather than the submerged weight of the sample.

Samples are then air dried to an approximately constant weight. They are weighed in air and water again, yielding values designated as W_5 and W_6. The samples are then placed in a cool oven which is started and raised to 105°C, and dried for 48 hours. This procedure is used to prevent the coating from separating from the sample due to thermal shock.

Samples are removed from the oven and cooled until they can be easily handled. They are weighed in air and water again (W_7 and W_8). All weights measured with the sample in water are buoyant force on the sample (W_4, W_6, W_8).

Summary of Measurements--

W_1 = Weight of wet sample = $W_s + W_w$

W_2 = Weight of wet sample plus tare = $W_s + W_w + T$

W_3 = Weight of wet sample, tare, coating = $W_s + W_w + T + W_r$

W_4 = Buoyant force on submerged sample

W_5 = Weight of air-dried sample, coating, tare = $W_s + (W_w)_a + T + W_r$

W_6 = Buoyant force on submerged air-dried sample
W_7 = Oven dry weight = $W_s + T + 0.85(W_r)$
W_8 = Buoyant force on sample
T_4, T_6, T_8 = Water temperatures at which the buoyant force measurements (W_4, W_6, W_8) are made

where

W_s = weight of solids
W_w = weight of water
T = tare weight (wire, tag, hair net)
W_r = weight of Saran™ resin coating (when oven dried the resin loses 15 percent of its weight).
γ_r = density of Saran™ resin = 1.2 g/cc

Computations--

1. Weight of solids:

$$(W_s) = W_7 - 0.85(W_3 - W_2) - (W_2 - W_1)$$

2. Water content (gravimetric):

$$(w) = \frac{W_1 - W_s}{W_s}$$

3. Dry bulk density of moist sample:

$$[Dbm] = \left[\frac{W_s}{\frac{W_4}{(\gamma_w)_4} - \frac{W_3 - W_2}{1.3}}\right]$$

where

$(\gamma_w)_4$ = water density at T_4

4. Void ratio of moist sample:

$$e = \frac{G_s \gamma_w}{Dbm} - 1$$

where

G_s = specific gravity of solid particles

5. Degree of saturation of moist sample:

$$S = \frac{1}{e}\left[\frac{(1 + w)(Dbm)(1 + e)}{(\gamma_w)_4}\right] - G_s$$

6. Water content (gravimetric) after air drying:

$$(w)_a = \left[\frac{W_5 - W_s - (W_2 - W_1) - (W_3 - W_2)}{W_s}\right]$$

7. Dry bulk density after air drying:

$$Dba = \left[\frac{W_s}{\dfrac{W_6}{(\gamma_w)_6} - \dfrac{W_3 - W_2}{1.3}}\right]$$

8. Void ratio after air drying:

$$(e)_a = \frac{G_s \gamma_w}{Dba} - 1$$

9. Degree of saturation of air dried sample:

$$(S)_a = \frac{wGs}{e_a}$$

10. Dry bulk density of the oven dried sample:

$$(Dbd) = \left[\frac{W_s}{\dfrac{W_8}{(\gamma_w)_8} - \dfrac{W_3 - W_2}{1.3} \quad 0.85}\right]$$

where

$(\gamma_w)_8$ = density of water at T_8

11. Suction compression index (C_h) =

$$\left[\frac{Dbd}{Dbm} - 1\right] \frac{Dbm}{Dbd} \left[\frac{1}{\log \dfrac{h_f}{h_i}}\right]$$

or

$$C_h = \frac{\Delta V/V_m}{\Delta \log h}$$

where

h_i = measured initial suction

h_f = final assumed to be 5.5 pF or 31.0 MPa or use a plot of $\Delta V/V_m$ versus suction to obtain h_f.

Example (weights in grams)--

h_i = 3.850 pF = 694 kPa

W_1 = 108.11

W_2 = 109.16

W_3 = 113.07

W_4 = 62.63

W_5 = 93.00

W_6 = 52.66

W_7 = 89.33

W_8 = 50.01

$T_4 = T_6 = T_8 = 71°F$

γ_w = 0.9772

G_s = 2.77

W_s = (89.83) - 0.85(113.07 - 109.16) - (109.16 - 108.11) = 85.46 g

$$w = \frac{108.11 - 85.46}{85.46} = 0.265 \text{ or } 26.5 \text{ percent}$$

$$Dbm = \frac{85.46}{\dfrac{62.63}{0.9772} - \dfrac{(113.07 - 109.16)}{1.3}} = 1.399 \text{ g/cm}^3$$

$$e = \frac{2.77 \cdot (1.0)}{1.399} - 1 = 0.980$$

$$S = \frac{1}{0.980}\left[\frac{(1.265)(1.399)(1.980)}{1.0}\right] - 2.77 = 0.805 \text{ or } 80.5 \text{ percent}$$

$$w_a = \frac{93.00 - 85.46 - (113.07 - 109.16) - (109.16 - 108.11)}{85.46} = 0.03 \text{ or}$$

3 percent

$$Dba = \frac{85.46}{\frac{52.66}{0.9772} - \frac{(113.07 - 109.16)}{1.3}} = 1.680 \ g/cm^3$$

$$e_a = \frac{(2.77)1.0}{1.680} - 1 = 0.649$$

$$s_a = \frac{0.03\ (2.77)}{0.649} = 0.128 \text{ or } 12.8 \text{ percent}$$

$$Dbd = \frac{85.46}{\frac{50.1}{0.9772} - \frac{[(113.01) - 109.16]}{1.3} 0.85} = 1.758 \ g/cm^3$$

$$C_h = \left[\frac{1.758}{1.399} - 1\right] \frac{1.399}{1.758} \left[\frac{1}{\log \frac{31.0}{0.694}}\right] = 0.124$$

PAPER NO. 10

Gamma100

A Re-examination of

Predictive Methods for Estimating Soil Swelling Properties

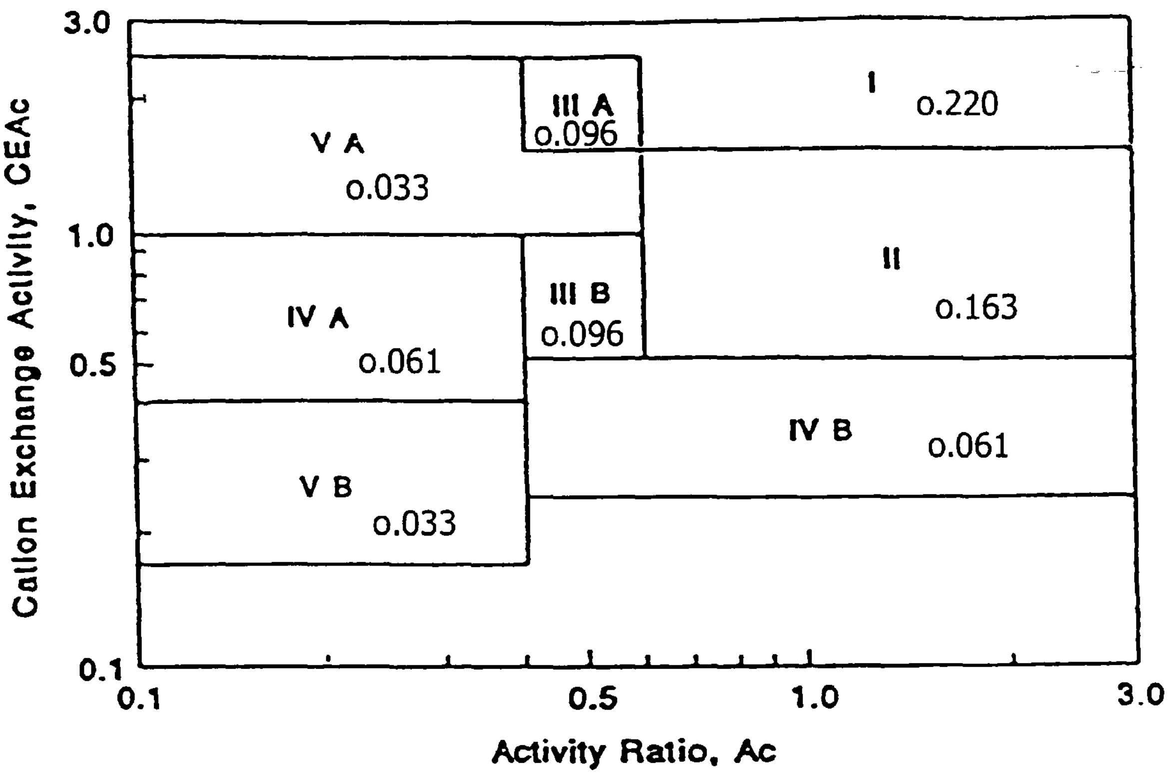
Preliminary Final Charts

Andrew P. Covar

Oct 1999

Background

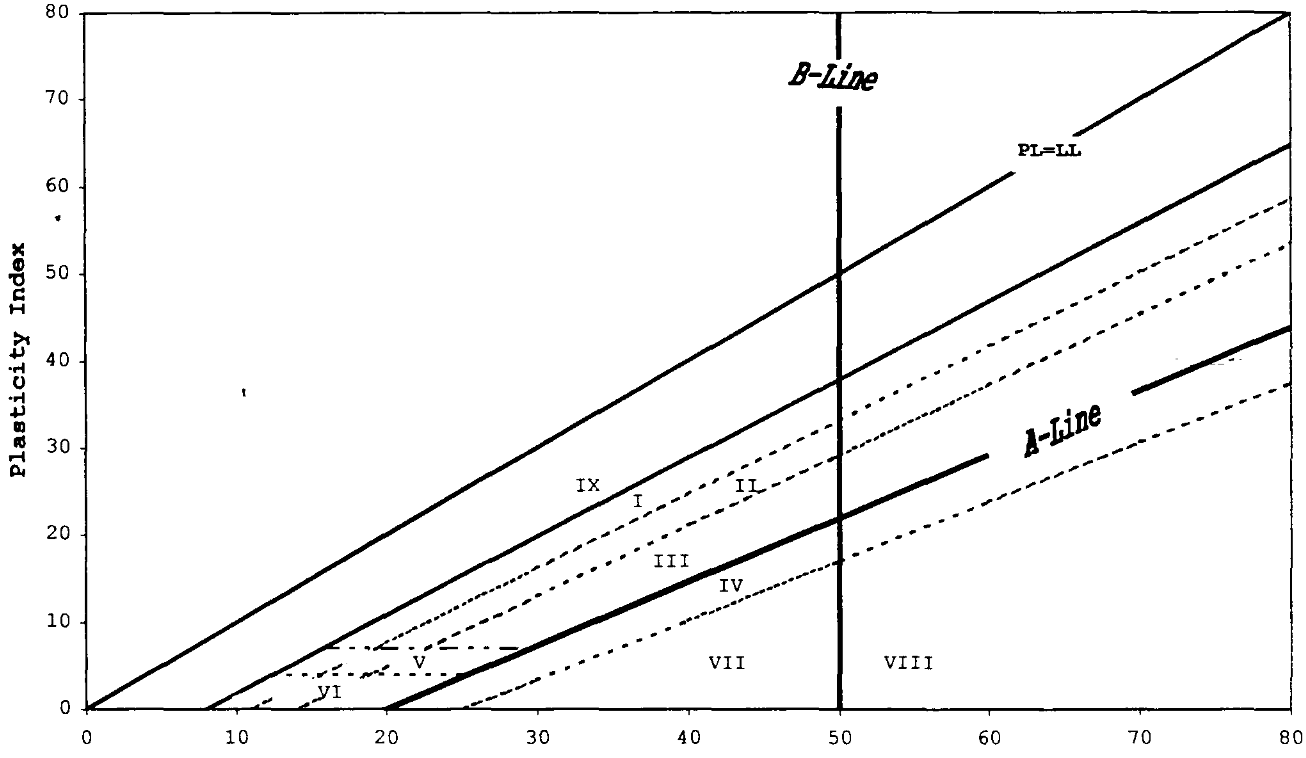
Chart 1, shown below, has been used for many years as an aid to the engineer in predicting the potential for soils to shrink and swell under certain conditions. The chart was produced with a good sampling of data available at the time. Since the early 1980's the availability of soil engineering data has vastly increased. This project makes use of a portion of that data.



Method

Using data from the Natural Resources Conservation Service a record of approximately 131,000 soil samples was examined and screened down to approximately 6500 individual soil samples each of which had been tested for the needed engineering properties. These data were then distributed among 9 different groups according the method described by Casagrande (Chart 2).

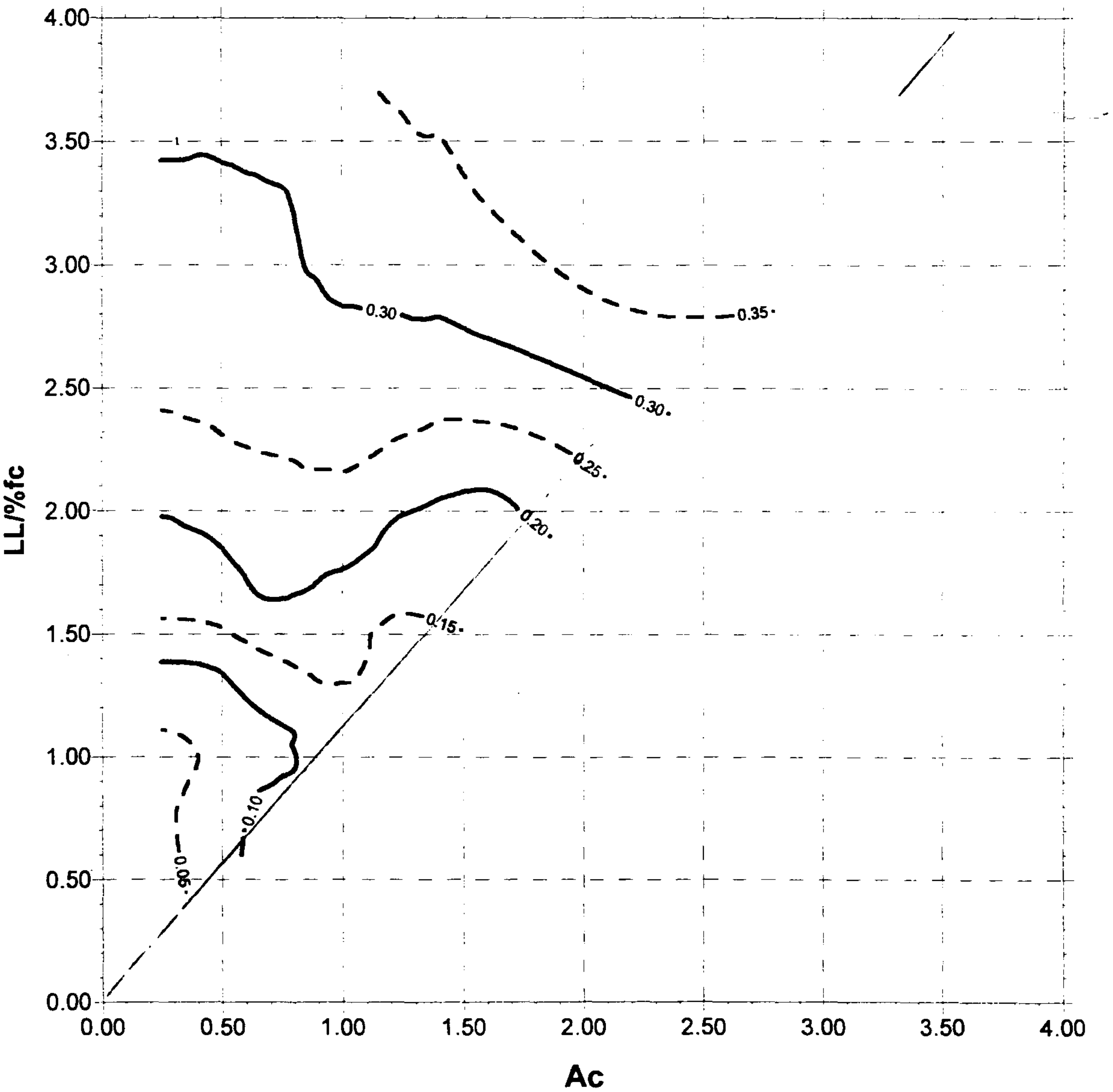
Contour plots were constructed for each of the 9 groups. These are shown on the following pages. These charts should be considered preliminary at this time pending additional technical review.



Mineral Classification Scheme

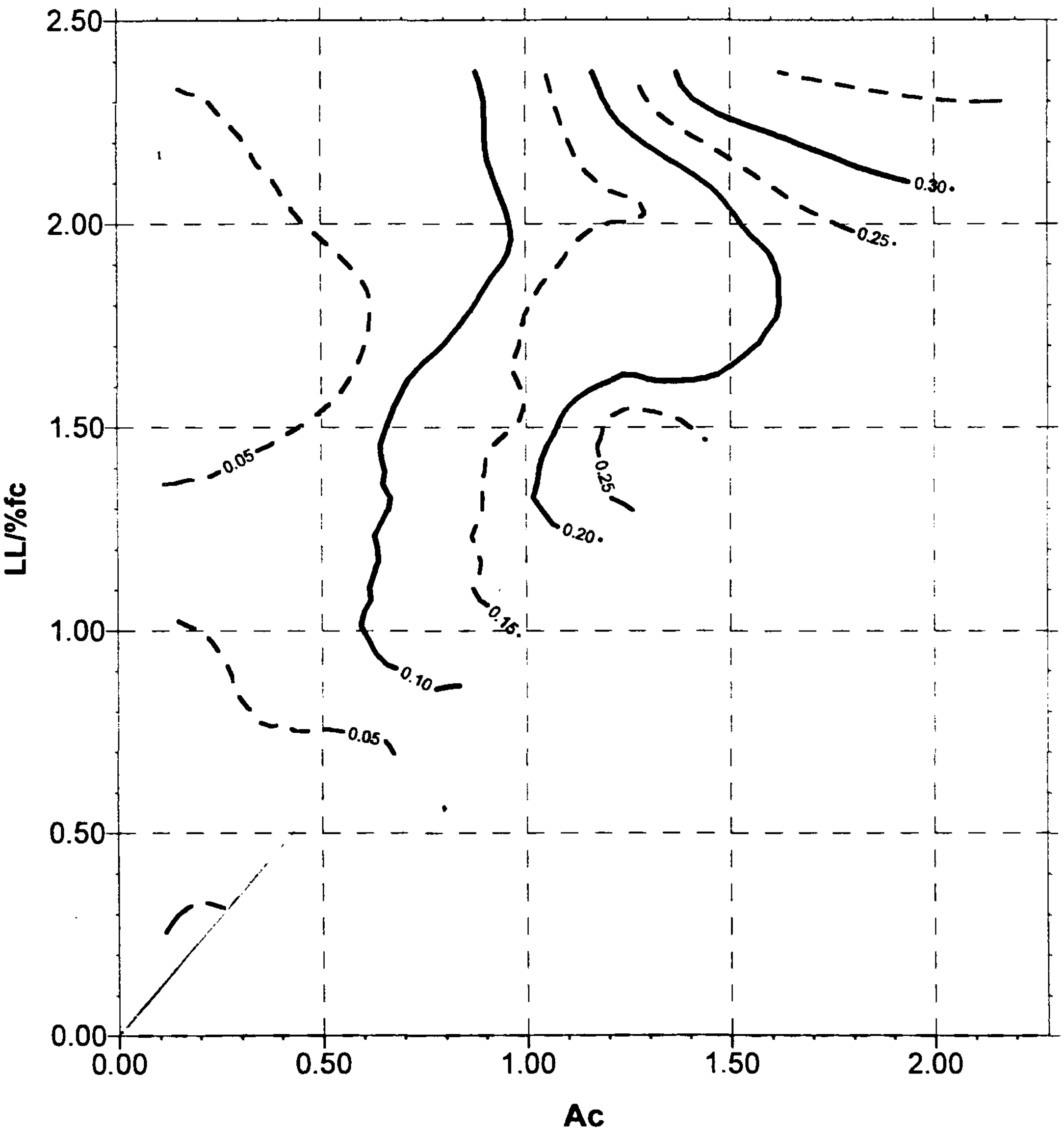
Each of the 9 charts that follow show predicted Gamma100 values for soils classified from the chart above. Constant value Gamma100 values are plotted on axes as follows, Ac = PI/fine clay, LL/%fc = Liquid Limit/%fine clay.

Zone I

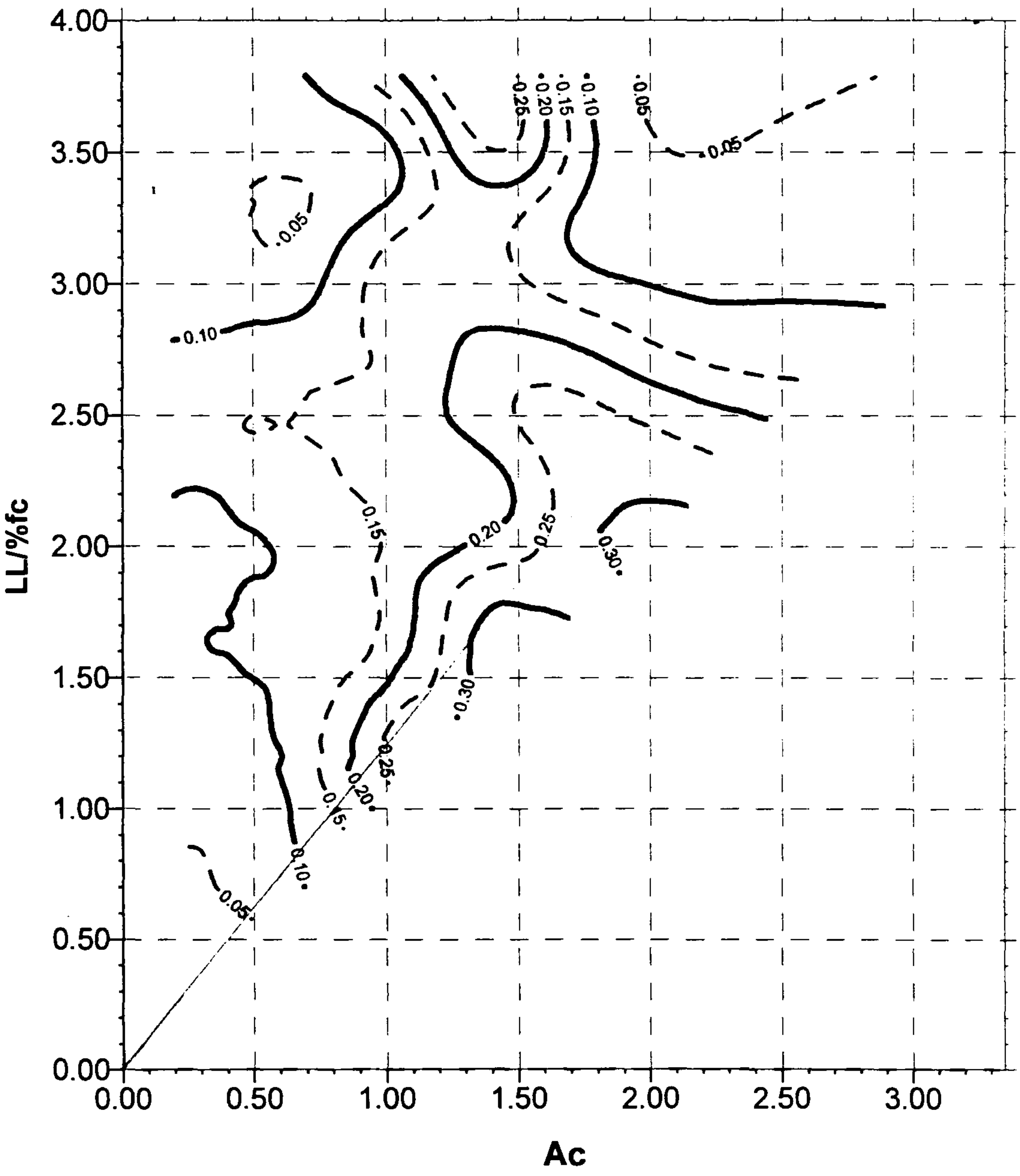


4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
LL/%fc
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
Ac
0.35
0.30
0.30
0.25
0.20
0.15
0.10
0.05

Zone II

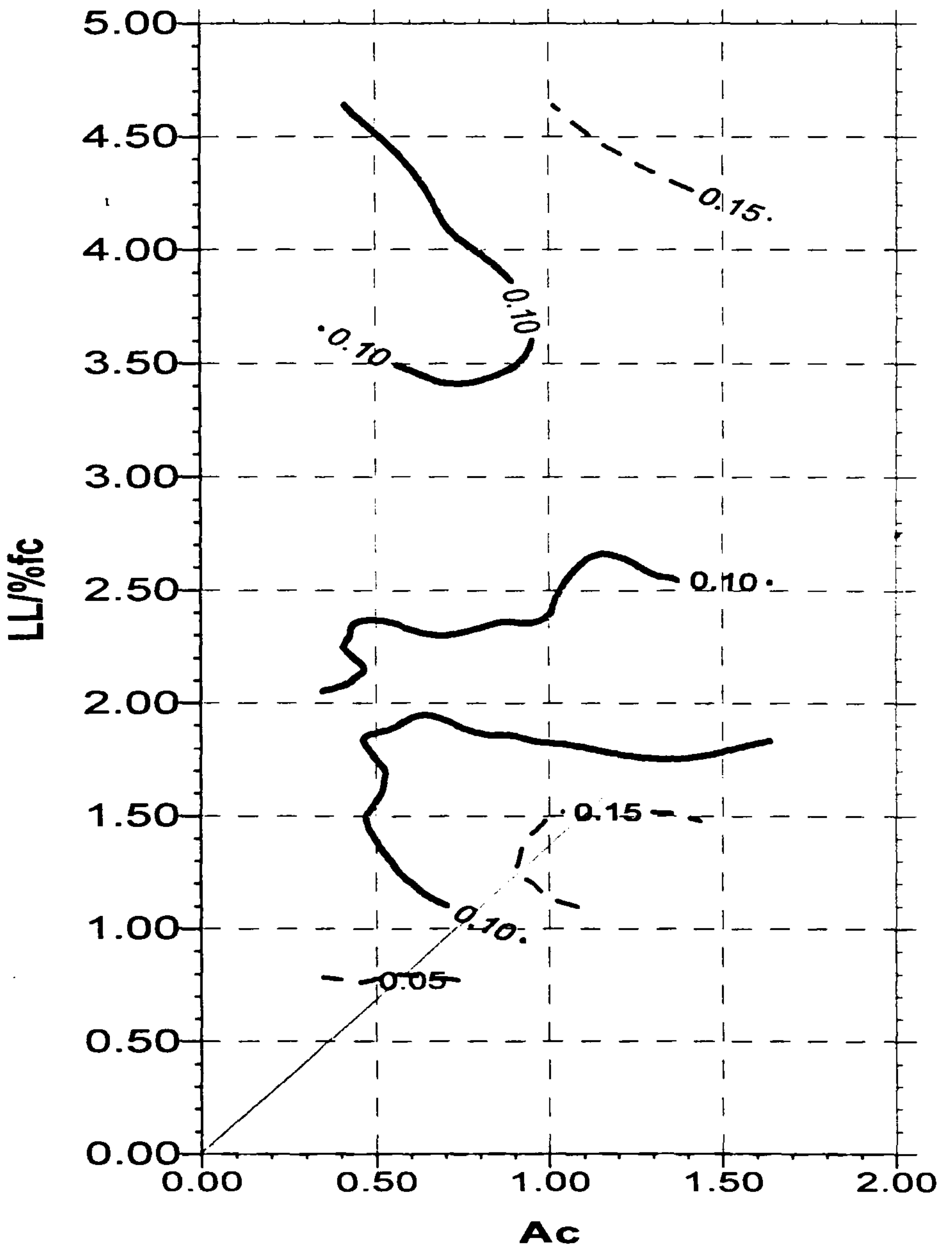

2.50
2.00
1.50
1.00
0.50
0.00
LL/%fc
0.00
0.50
1.00
1.50
2.00
Ac
0.05
0.10
0.15
0.20
0.25
0.30

Zone III

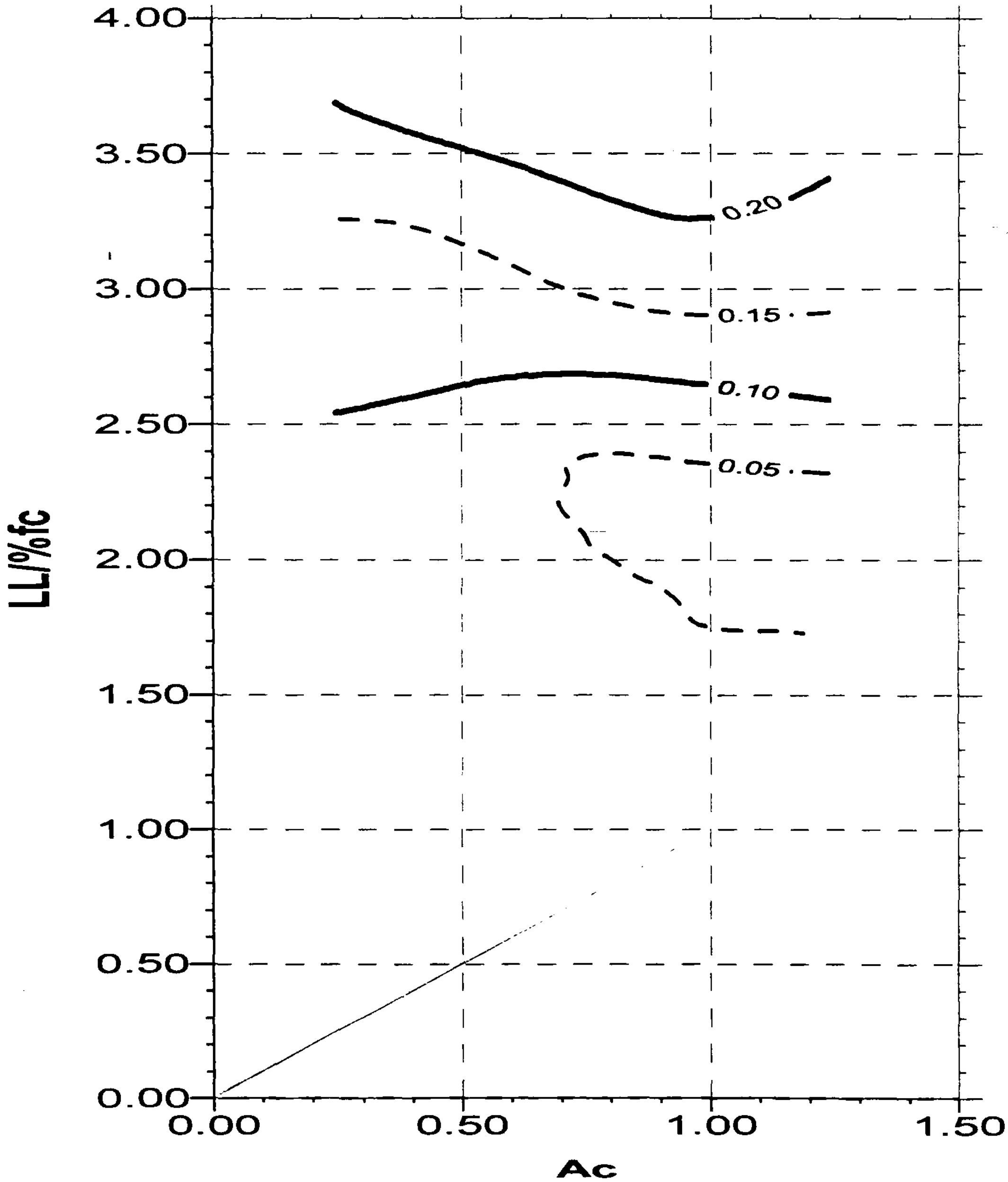


LL/%fc
Ac
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
0.00
0.50
1.00
1.50
2.00
2.50
3.00
0.05
0.10
0.15
0.20
0.25
0.30

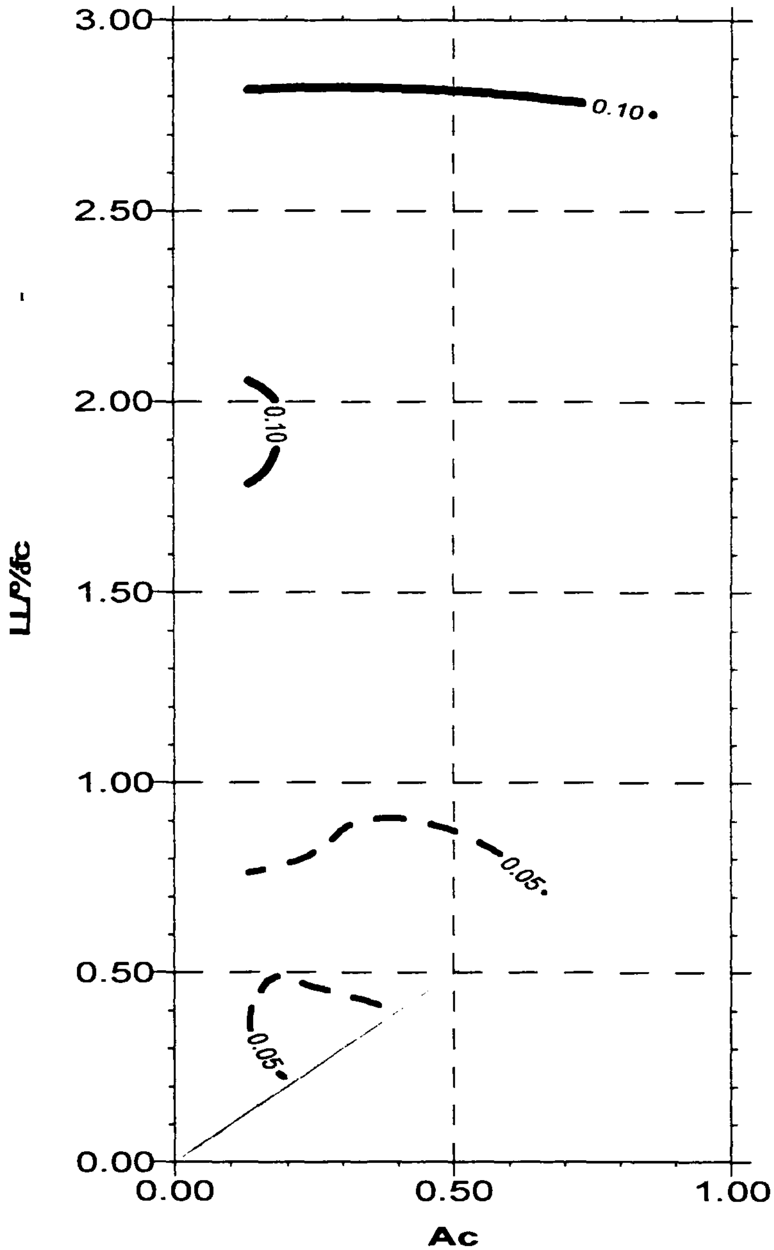
Zone IV


5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
LL/%fc
0.00
0.50
1.00
1.50
2.00
Ac
0.15
0.10
0.10
0.10
0.15
0.10
0.05

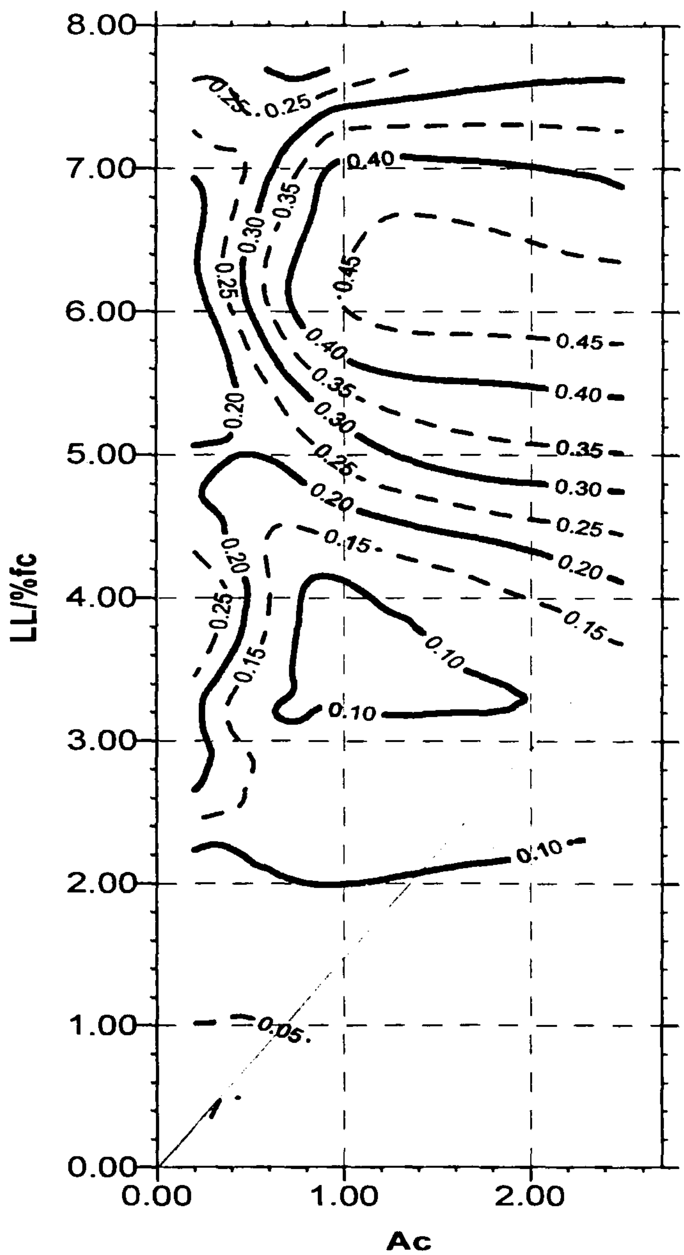
Zone V


4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
LL/%fc
0.00
0.50
1.00
1.50
Ac
0.20
0.15
0.10
0.05

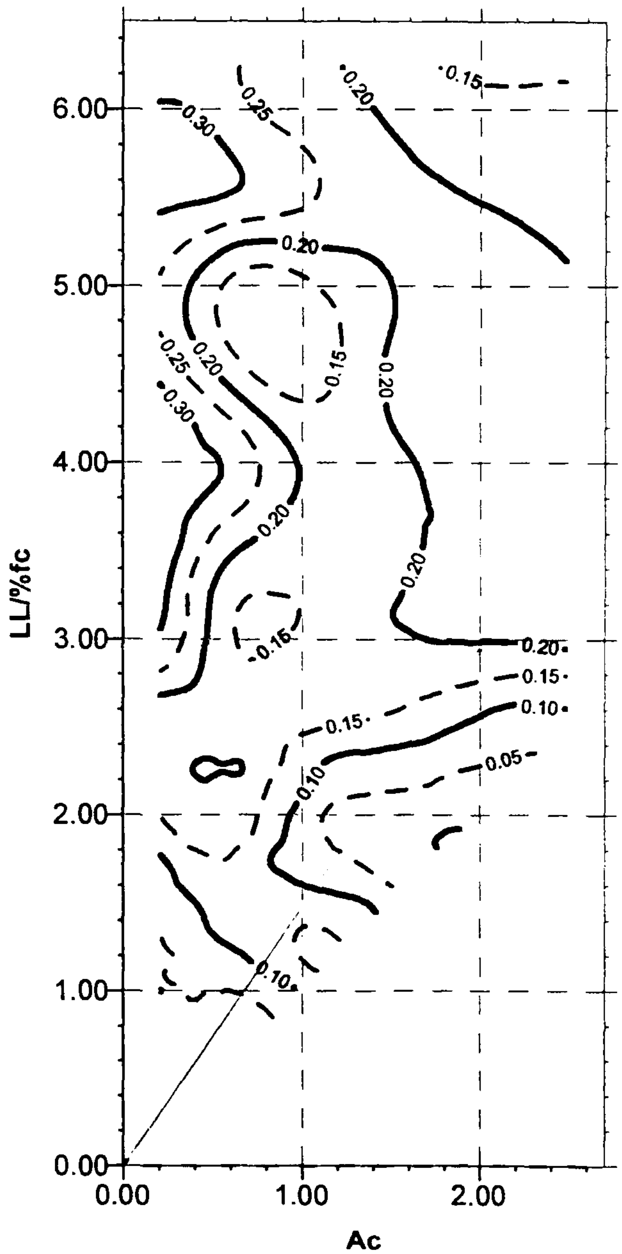
Zone VI



Zone VII

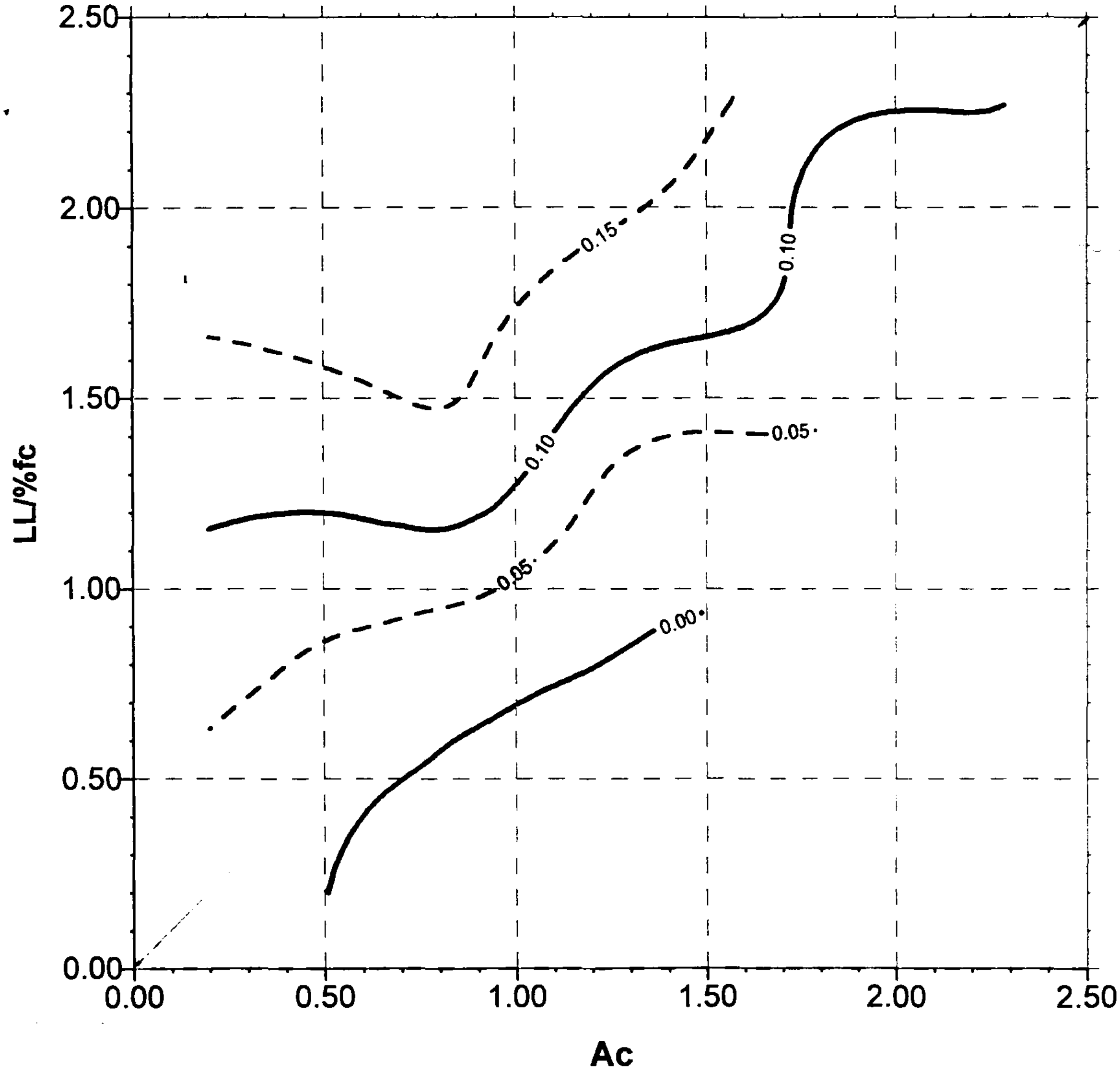

8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
LL/%fc
0.00
1.00
2.00
Ac
0.25
0.30
0.35
0.40
0.45
0.20
0.15
0.10
0.05

Zone VIII



0.15
0.25
0.20
0.30
0.20
0.15
0.20
0.25
0.20
0.30
0.20
0.20
0.15
0.20
0.15
0.10
0.15
0.10
0.05
0.10
0.00
1.00
2.00
3.00
4.00
5.00
6.00
LL/%fc
Ac

Zone IX


2.50
2.00
1.50
1.00
0.50
0.00
LL/%fc
0.00
0.50
1.00
1.50
2.00
2.50
Ac
0.15
0.10
0.10
0.05
0.05
0.00

PAPER NO. 11

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 above 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 moisture active zone. The soil zone in which movement occurs is always more shallow than this and is called the movement active zone. 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 monitoring 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

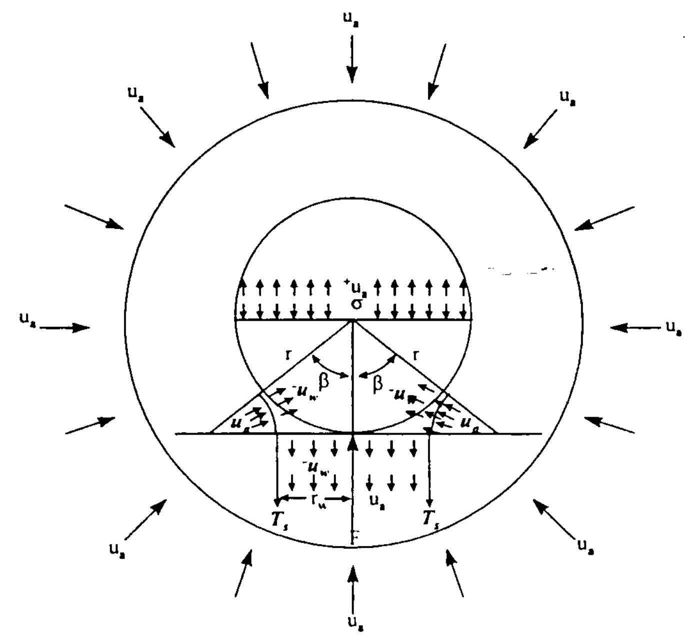


Figure 1. Free Body Diagram

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:

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

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

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

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

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

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:

$$\Delta\sigma = \theta f\,|\Delta h| \quad (5)$$

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.

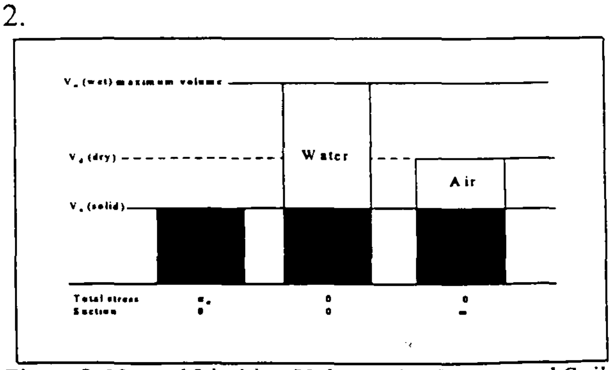


Figure 2. Natural Limiting Volumes in Unsaturated Soils and Corresponding Stress States.

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

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

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

$K_o = ½$ 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} mm²/sec and 7.8 x 10^{-1} mm²/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.

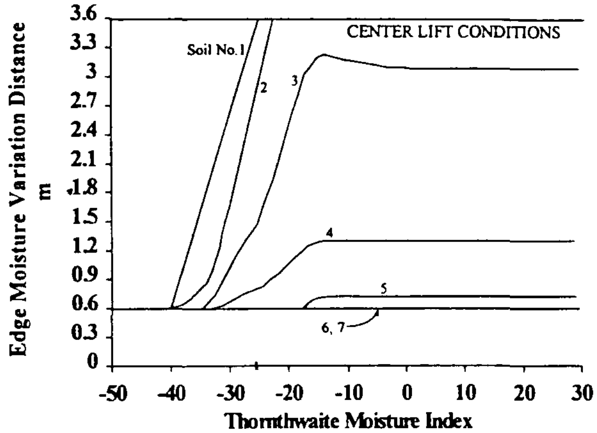


Figure 3. Edge Drying

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}

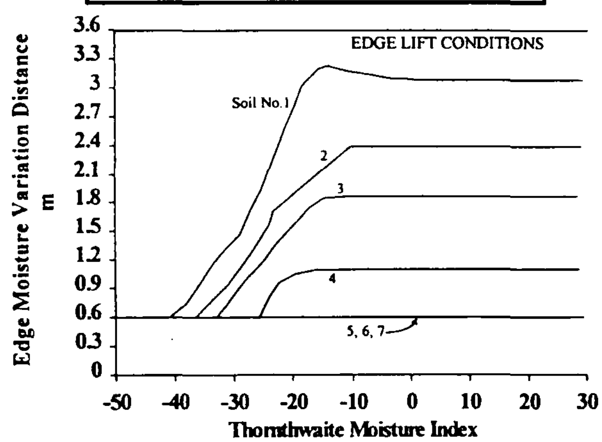


Figure 4. Edge Wetting.

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 on the log mm scale.

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

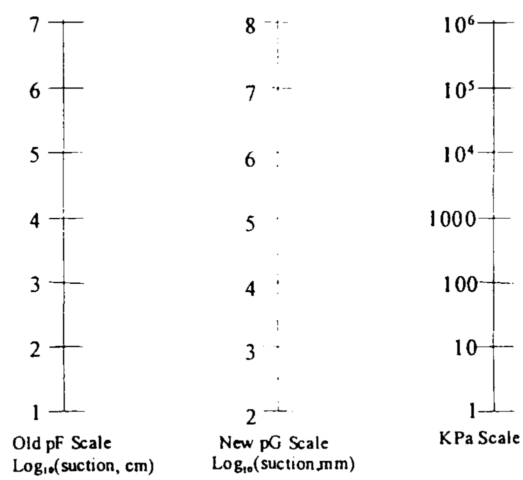
$$e_m = 10^{-3}\left(\sqrt{\frac{\alpha}{n\pi}}\right)\ln\left(\frac{2u_o}{0.2}\right) \tag{12}$$

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

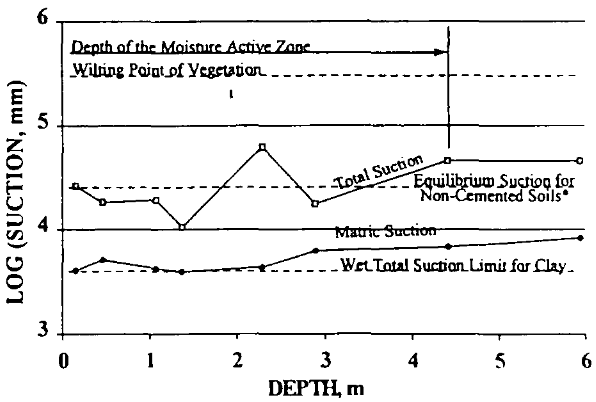
Several clues are available in the suction profile to indicate the depth of the moisture active zone, as follows:

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.

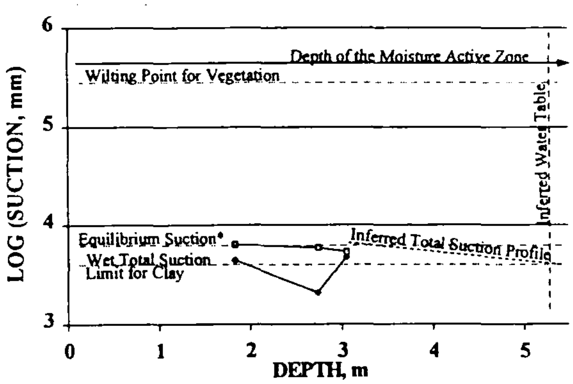
The first criterion is illustrated in Figure 4



• 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.

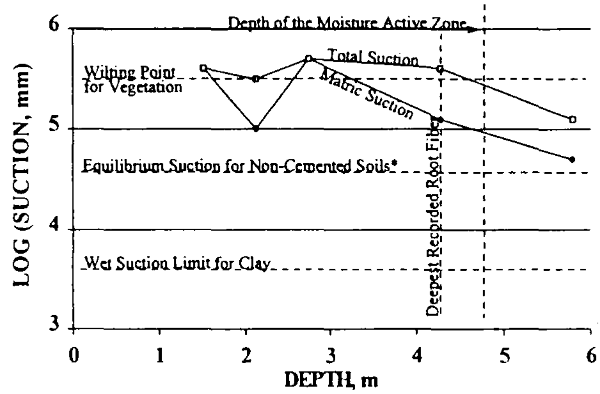
The second criterion is illustrated in Figure 5, which is a set of suction measurements made in and around a swamp in Louisiana. The location of the water table was inferred by projecting the total suction in mm downward on a 1:1 slope until it reached the wet limit of suction in clay of 3162 mm (pG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

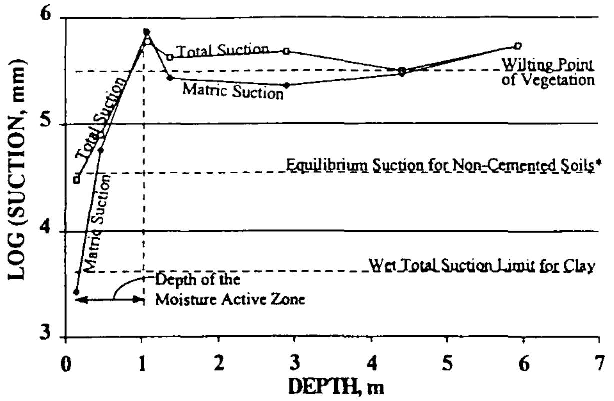
The third criterion is illustrated in Figure 6 which was measured in the root zone of a large oak tree in Texas during a hot, dry summer. The deepest recorded root fiber was at 4.3 m. The total suction, which had been at or slightly above the wilting point down to that point, began below that point to 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

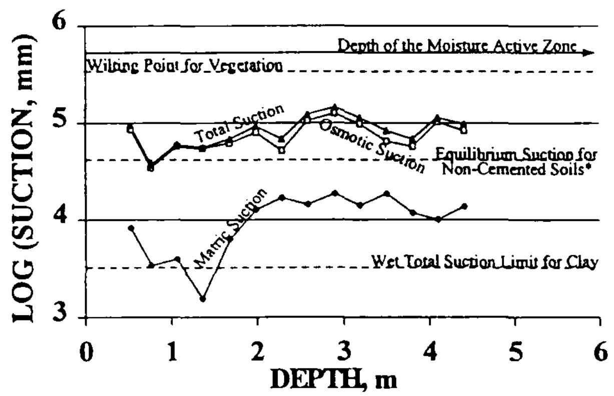
The fourth criterion is illustrated in Figure 7, a suction profile showing a cemented soil which roots cannot penetrate below a depth of 0.8 m. The inference that it is a cemented soil comes both from the boring log comments on the soil being "very stiff" and from the high level of matric suction, nearly 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.

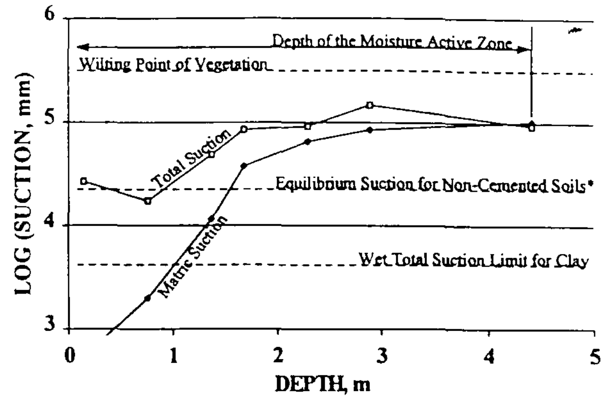
Figure 8 shows a profile of a soil with a high osmotic suction level but one that is not high enough to prevent the penetration of roots. The soil had high concentrations of soluble sulfates and underlay a pavement that had experienced repeated episodes of repeated distress. Consequently, although the 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, $|h_{dry}|$

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

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 x 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

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

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

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

whereu(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

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

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:

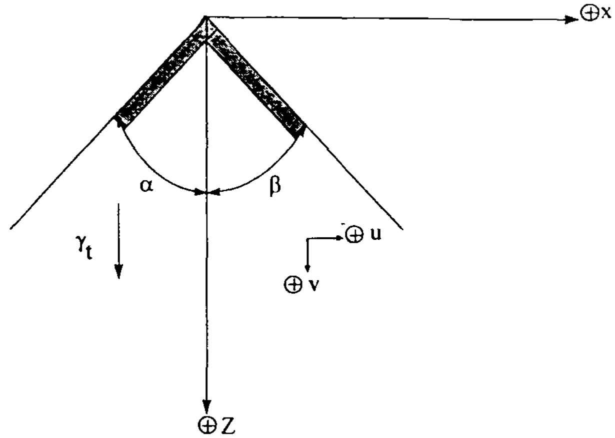


Figure 10. Sign Conventions for Love's Solution for an Elastic Wedge

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

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

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

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

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.

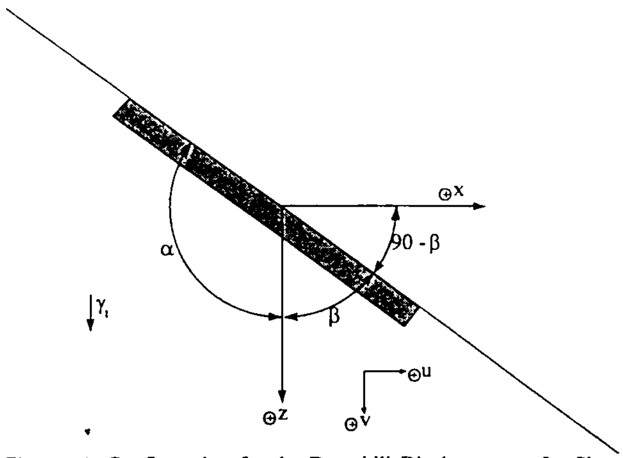


Figure 11. Configuration for the Downhill Displacement of a Slope with a Slope Angle of (90-β)°.

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

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

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

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

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

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

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

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

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.

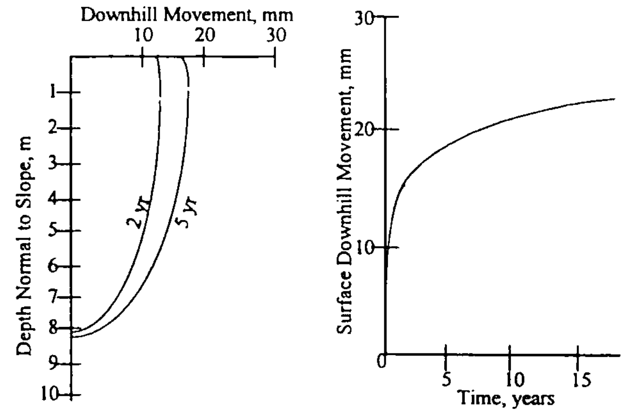


Figure 12. Typical Computed Downhill Creep Movements. Soil Properties used in making these computation were measured in the laboratory and are as follows: E_α=37 kPa; E_1=3.8 x 10^5 kPa-(s)^m; m=0.24; ν=0.4; γ_t=18 kN/m^3

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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PAPER NO. 12

Engineering Structures in Expansive Soils

Robert L. Lytton

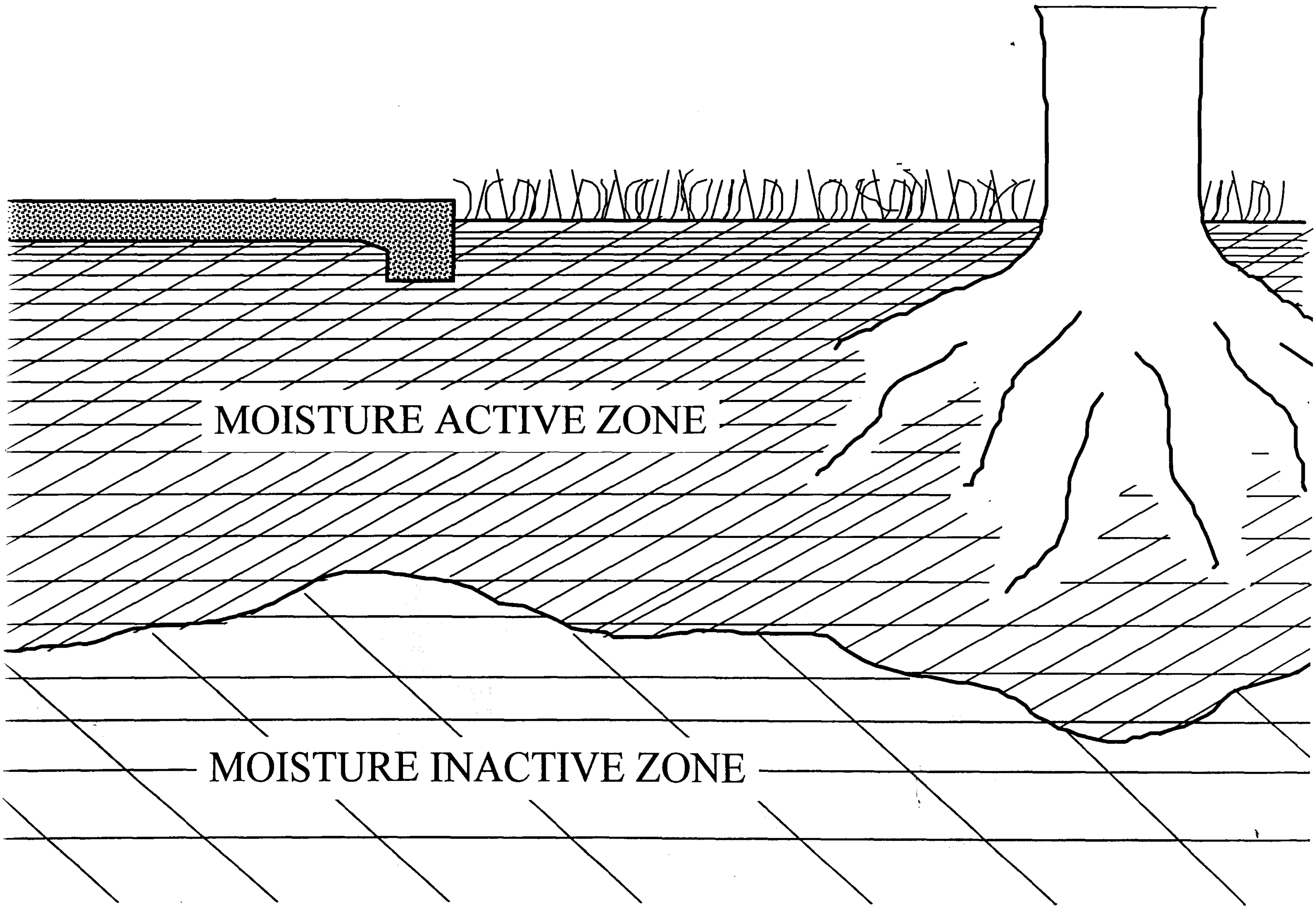
NSAT'-97

Third Brazilian Symposium on Unsaturated Soils

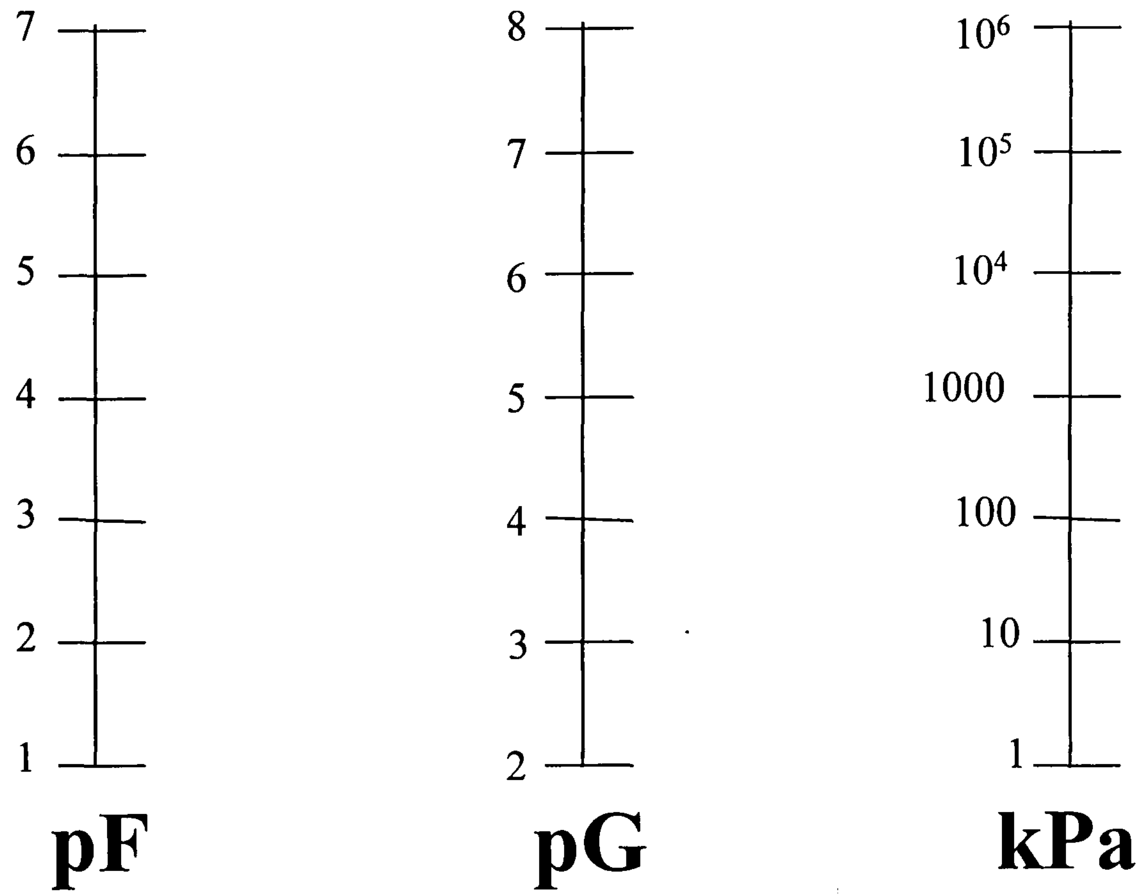
22-26 April 1997

Problem Site Conditions

- ★ Vegetation
- ★ Drainage
- ★ Slopes

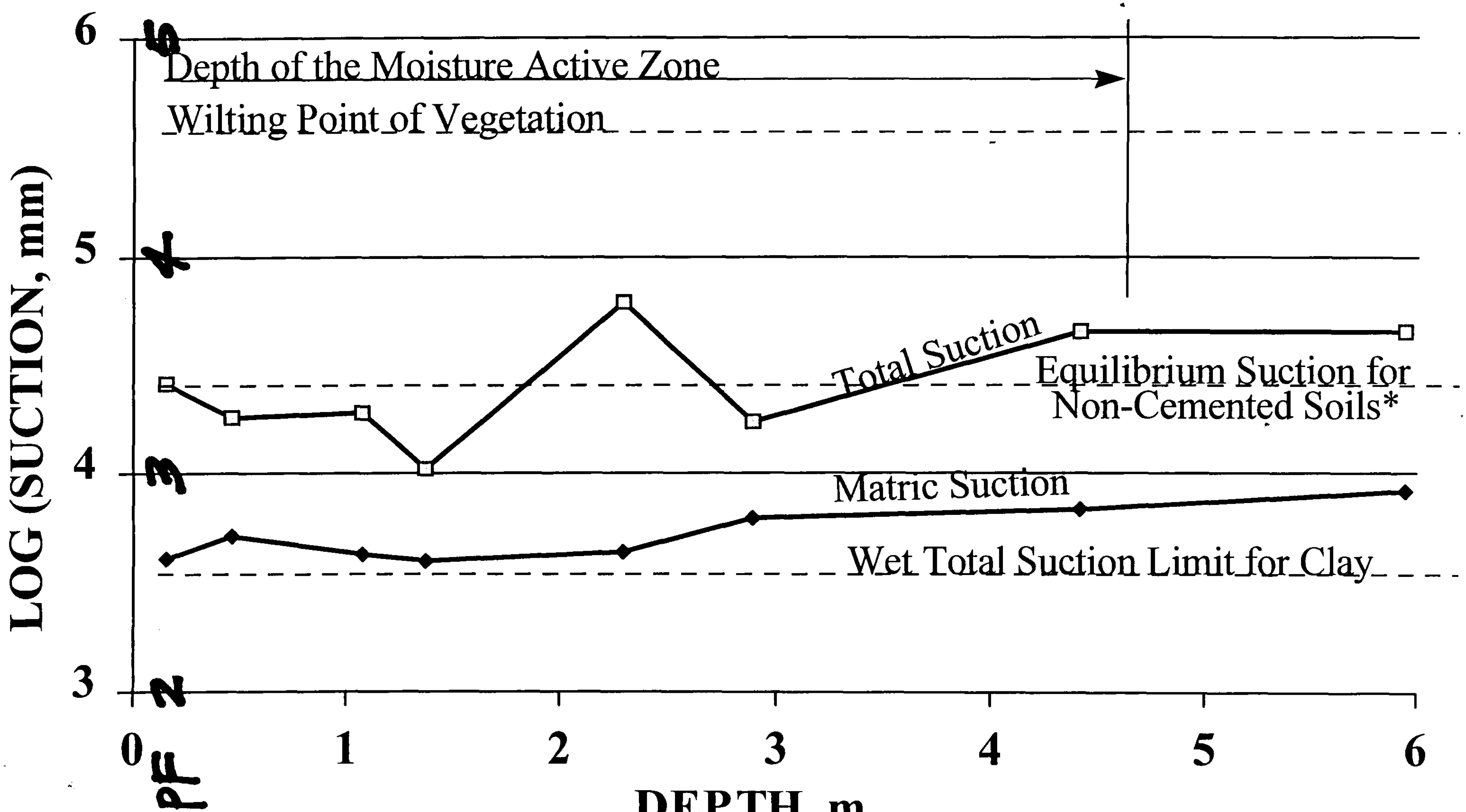

MOISTURE ACTIVE ZONE
MOISTURE INACTIVE ZONE

Log Suction Scales

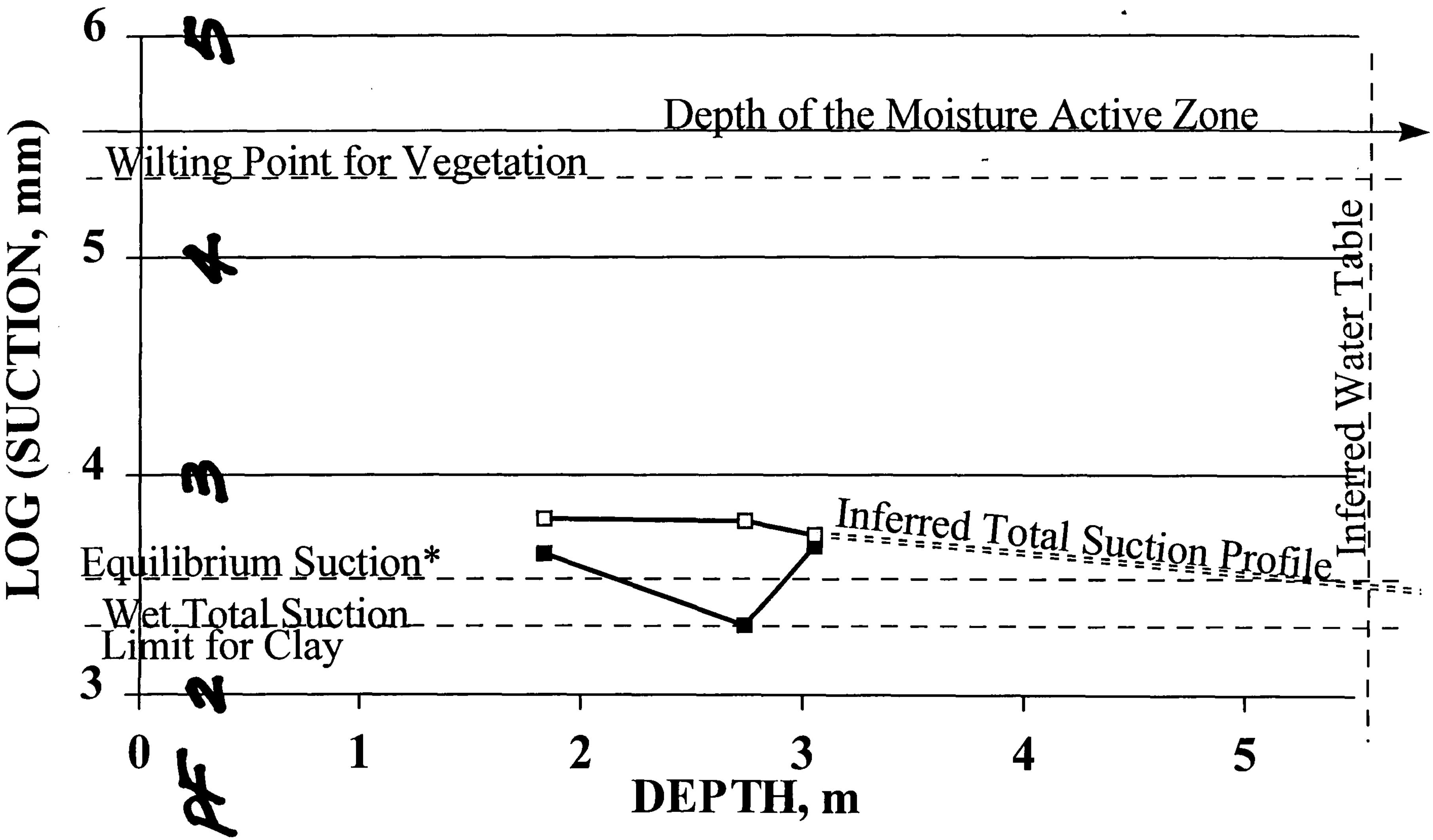


log10(| suction | , mm)

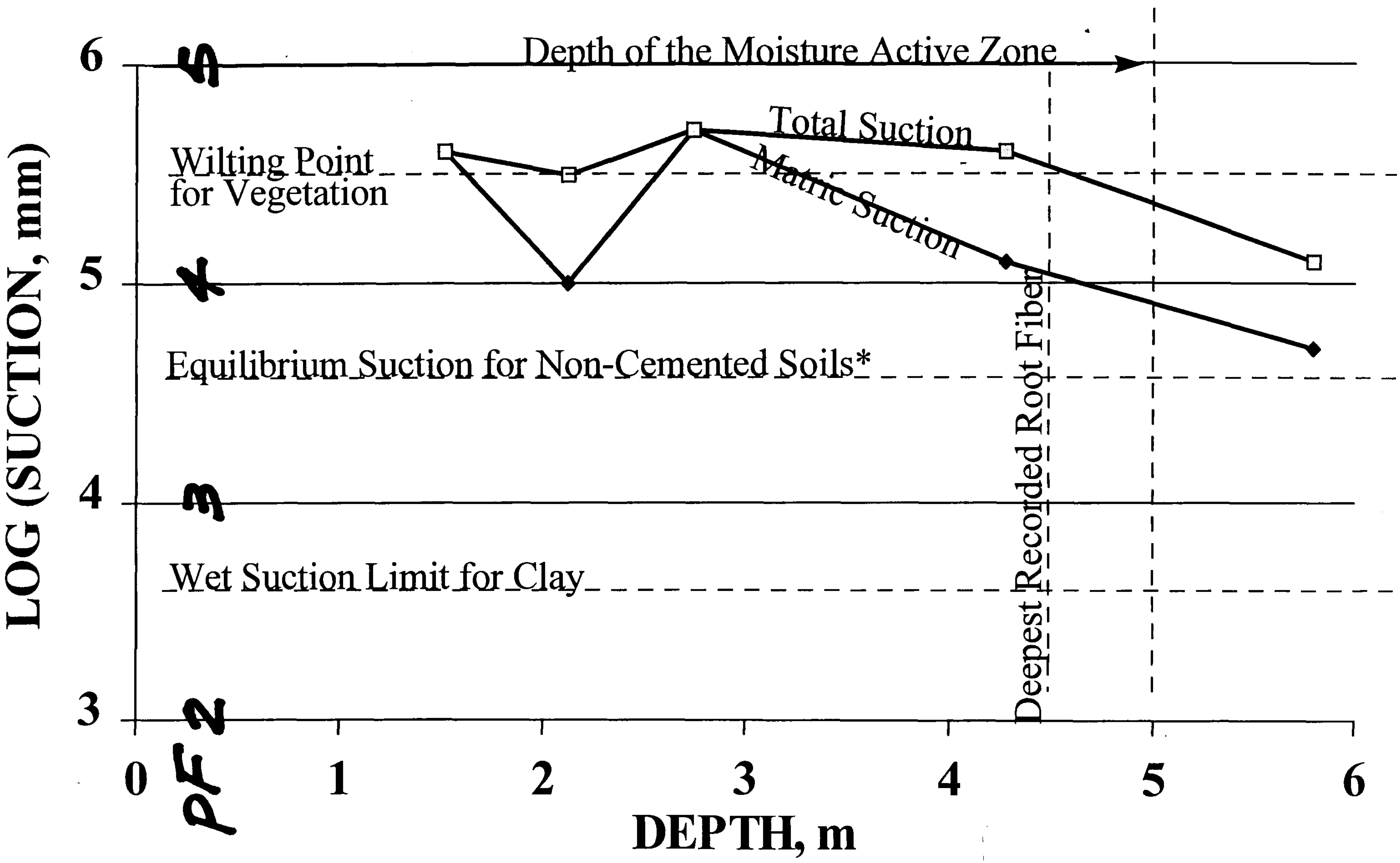
Moisture Condition	$\log_{10}(\lvert suction \rvert, mm)$	pF
Field Capacity	3.0	2.0
Clay Wet Limit	3.5	2.5
Wilting Point	5.5	4.5
Air Dry	7.0	6.0



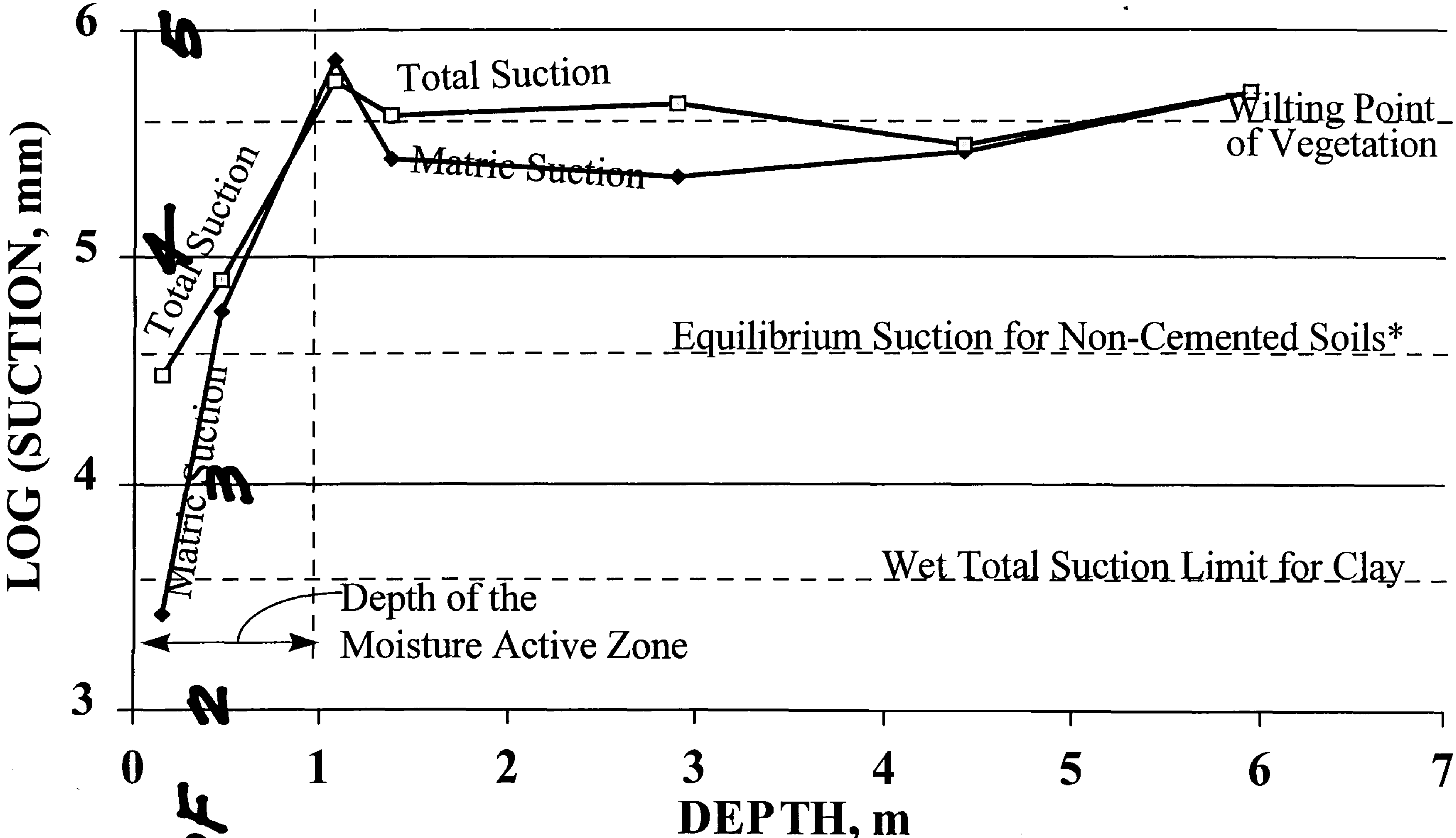
* From Epirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)



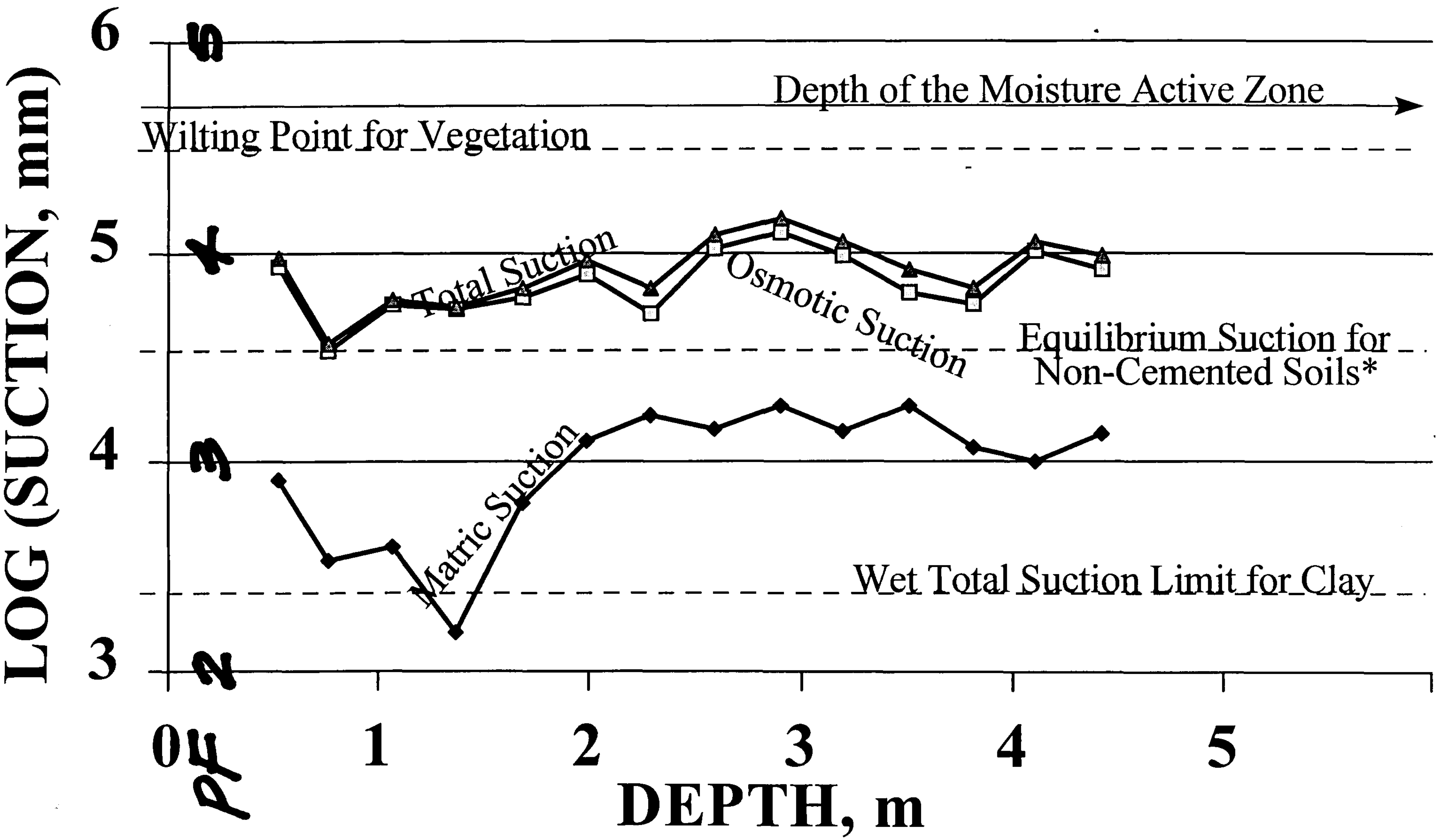
* From Empirical Thornthwaite Moisture Index Relation with equilibrium suction (Russam and Coleman, 1961)



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



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



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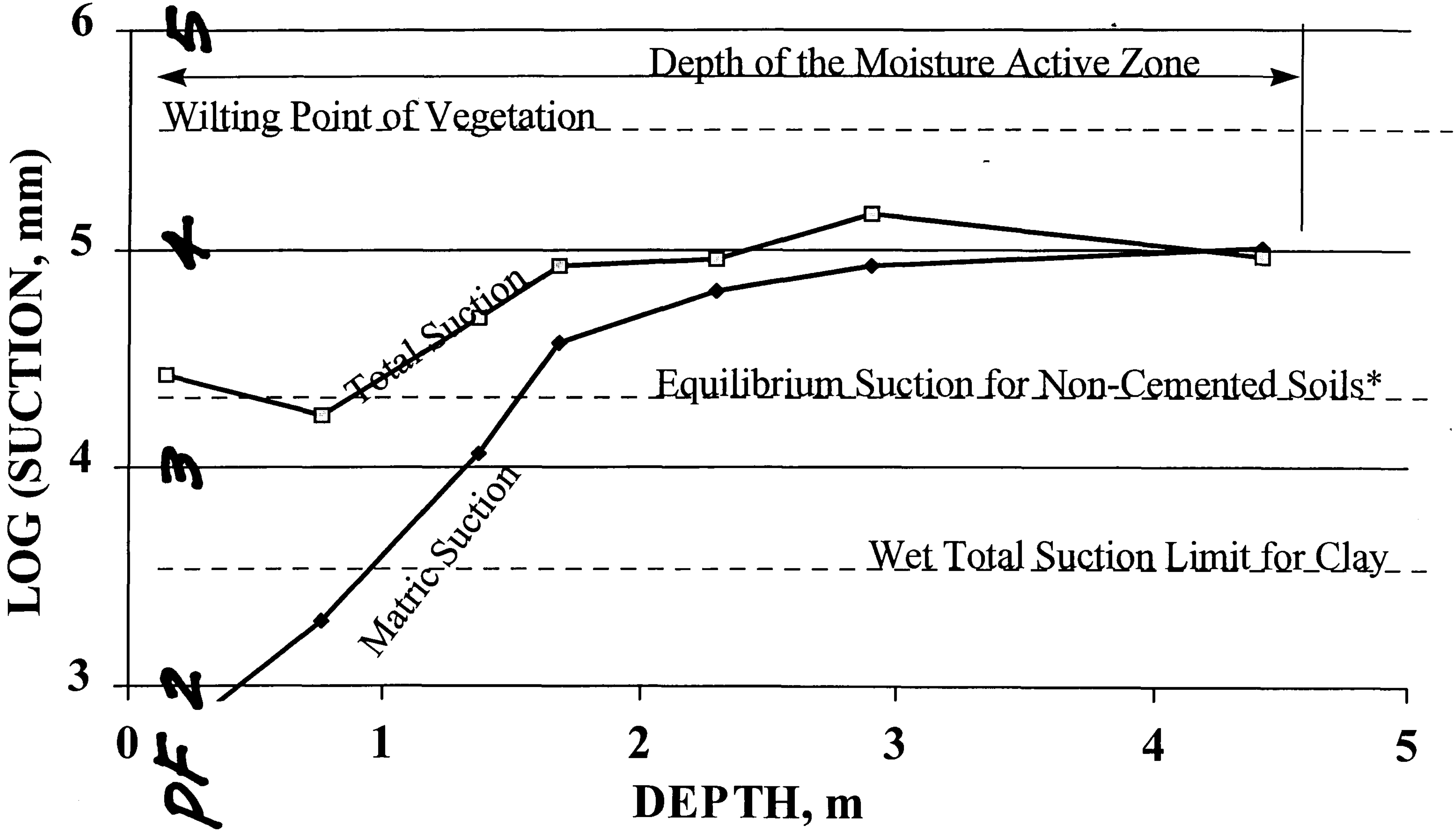

* From Empirical Relation of Thornthwaite Moisture Index with equilibrium suction (Russam and Coleman, 1961)

PAPER NO. 13

TRANSPORTATION RESEARCH

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No. 1615

Soils, Geology, and Foundations

Applications of Emerging Technologies in Transportation

TRANSPORTATION RESEARCH BOARD

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Variation of Soil Suction with Depth in Dallas and Fort Worth, Texas

JOHN T. BRYANT

The variation of soil suction and the estimate of constant soil suction with depth is used to design many slab-on-grade foundations and pavement moisture barriers. The Post-Tensioning Institute's design procedures for slab-on-grade foundations and design of vertical pavement moisture barriers use the constant suction at depth to predict differential soil movements that influence shear, deflection, and moment magnitudes and the effective barrier depth. Constant soil suction estimates can be correlated to the climate or long-term weather conditions at any given site by using the Thornthwaite moisture index (TMI), which estimates the amount of net moisture surplus or deficit from precipitation and evapotranspiration of moisture from the ground surface. On the basis of the empirical curves, the constant value of total soil suction for the Dallas–Fort Worth, Texas (DFW) area is about 246 kPa based on an average TMI of 0. Analysis of more than 1,200 total soil suction laboratory tests performed on developed and undeveloped lots indicates that the measured average total soil suction value in the upper 6 m is closer to 979 kPa for the DFW area ranging between 55 kPa and 11,246 kPa during the 1995–1997 period. Some hypothesized reasons for the difference between the empirical and measured equilibrium (constant) soil suction values are amounts of clay, clay origin, variable plasticity indexes, soluble salt content, and equilibration curve differences.

The variation of soil suction and the estimate of the constant soil suction with depth is used in the design of many slab-on-grade foundations. By using the Post-Tensioning Institute's (PTI's) design method, the design of posttensioned slab-on-grade foundations uses the value of constant suction at depth to aid in the prediction of differential soil movements, which influence the shear, deflection, and moment magnitudes affecting these foundation systems (1). Further, the design of vertical moisture barriers for pavement structures and foundations uses the constant soil suction values and the depth to constant soil suction to estimate the effective depth of the barrier.

The estimate of the constant soil suction value can be made on the basis of the climate or long-term weather conditions at any given site. This climatic variable is called the Thornthwaite moisture index (TMI) (2) and is used to determine the amount of net moisture surplus or deficit as a result of precipitation and evapotranspiration of moisture from the ground surface. The TMI is a characteristic of a site's climatic influences over a distinct period. A better estimate of the average value of the characteristic constant soil suction is obtained by using longer periods of climatic data for a given site.

The predicted values of soil suction based on the TMI are correlated to published curves (1,3). These curves predict slightly different constant soil suction values for respective TMI values, as shown in Figure 1. For the Dallas–Fort Worth, Texas (DFW) area the constant value of total soil suction is on the order of 246 kPa based on an average TMI of 0.

Bryant Consultants, Inc., 4394 Westgrove, Dallas, TX 75248.

TOTAL SOIL SUCTION

Soil suction, also known as potential, is a measure of the free energy content of soil water. The theory of soil suction and the related equipment and methods used in its measurement are well-documented. The measurement of suction can be obtained by direct or indirect method. The filter paper method is an indirect measurement in which the filter paper serves as a passive sensor. The basic principle of this method is that a filter paper, after an equilibrium period, exchanges moisture with the soil at a specific soil suction. This occurs because the relative humidity inside the soil specimen container is controlled by the soil water content and suction. Following the equilibrium period, the filter paper will have absorbed moisture equivalent to the relative humidity in the container, and the corresponding suction in the filter paper will be the same as that in the soil specimen.

LABORATORY TESTING

Measurements of the total soil suction used in this research were performed on undisturbed soil samples taken in the field at depths ranging from 0.3 m to more than 12 m below the ground surface using nominal 76-mm-diameter seamless tube samplers. The samples were packaged in the field and were wrapped in foil and placed in a plastic bag to prevent desiccation. Transportation of the soil samples to the laboratory typically occurred within several hours of the sampling.

Laboratory testing of the soil samples for total soil suction were performed in accordance with ASTM D5298-93. The total soil suction test involved placing the soil samples into sealed containers with calibrated filter papers and allowing approximately 7 days for the relative humidity within the container to come into equilibrium with the pore water vapor pressure inside the soil interstices.

Deviations from the ASTM D5298-93 apparatus requirements were as follows:

1. Whatman No. 42 ashless 55-mm filter paper was used. No special pretreatment of the filter paper was applied.
2. A 348-mL polyethylene specimen container was used instead of a metal or glass container. The container had a clamp seal.
3. Two wraps of electrical tape, approximately 6 mm wide, were used instead of the flexible plastic electrical tape to further seal the outside lid–container connection.
4. Rubber O-rings were used instead of a screen wire or brass discs to separate the filter papers during equilibration.

All weighing and transfer of the filter papers from the specimen container into the metal weighing container was performed by a trained laboratory geotechnician or by the author. The filter paper moisture contents were converted to suction values by using the Whatman No. 42 calibration curve given in ASTM D5298-93.

GEOLOGIC CONDITIONS OF AREA

The DFW area lies within the Upper Cretaceous and Lower Cretaceous sedimentary rock and Quaternary aged alluvial deposits. The sedimentary rock strata dip gradually toward the south and southeast and increase in age from east to west. The sedimentary strata present from approximately the east boundary to the west boundary of the area consist of the following, in order: Ozan–Lower Taylor marl; Austin chalk limestone; Eagle Ford shale; Woodbine formation including sands, clays shales, and sandstones; Main Street/Paw Paw limestone; and Paluxy formation consisting predominantly of sands and sandstone strata. The interbedded sedimentary rock formations typically are dissected by the Trinity River and its tributaries, which have deposited Quaternary aged sands, silts, sandy clays, and gravel along the present and ancient channels and flood plains of these rivers and creeks. Samples from all of these formations are combined in the analysis of the total soil suction variation across the DFW area.

ANALYSIS AND DISCUSSION OF RESULTS

Figure 1 provides a graphical comparison of the Russam-Coleman and PTI soil suction curves as functions of the TMI. The soil suction values are reported in pF units (logarithm to the base 10 of the negative pore pressure in centimeters of water). The curves are of similar shape, although the Russam-Coleman curve overpredicts the PTI curve for dry climates with TMI values less than −10. Conversely, the Russam-Coleman curve underpredicts the PTI curve for wet climates with TMI values greater than 0.

The DFW area is approximately bounded by the TMI values of −10 to 10 with an average value on the order of 0. The Russam-Coleman and the PTI curves indicate that the equilibrium suction for soils in the DFW area are on the order of 246 kPa. However, the curves shown in Figure 1 appear to underpredict the actual measured values of the total soil suction values for the DFW area measured between 1995 and 1997.

Figures 2, 3, and 4 show the variation of the total soil suction with depth across the DFW area during 1995 through 1997. Soil samples were taken in both developed and undeveloped areas so that a range of the total soil suction values preconstruction and postconstruction could be estimated. Figures 2, 3, and 4 represent 1,225 separate independent laboratory measurements of total soil suction using the filter paper method. Figure 2 presents the suction profile measured in 1995, and Figures 3 and 4 present the suction profiles measured in 1996 and 1997, respectively. Table 1 presents a statistical summary of the soil suction data collected for this study. The results of these soil suction tests fall between 10 kPa (2 pF) and 97 948 kPa (6 pF), which are considered to be extreme values for soil suction at the field capacity and an extreme controlling dry suction.

Review of Figures 2, 3, and 4 reveals that the total soil suction is most variable at the surface of the ground, becoming slightly less variable with depth. Table 1 indicates that the average total soil suction values are on the order of 979 kPa (4 pF) for the DFW area over the last 2.5 years. This number is substantially higher than the value of 246 kPa (3.3 to 3.4 pF) predicted by Russam-Coleman or PTI from Figure 1, and it underpredicts the actual measured total soil suction value for the DFW area between 1995 and 1997. The skew shown in Figure 4 in the 1997 results most probably is due to the

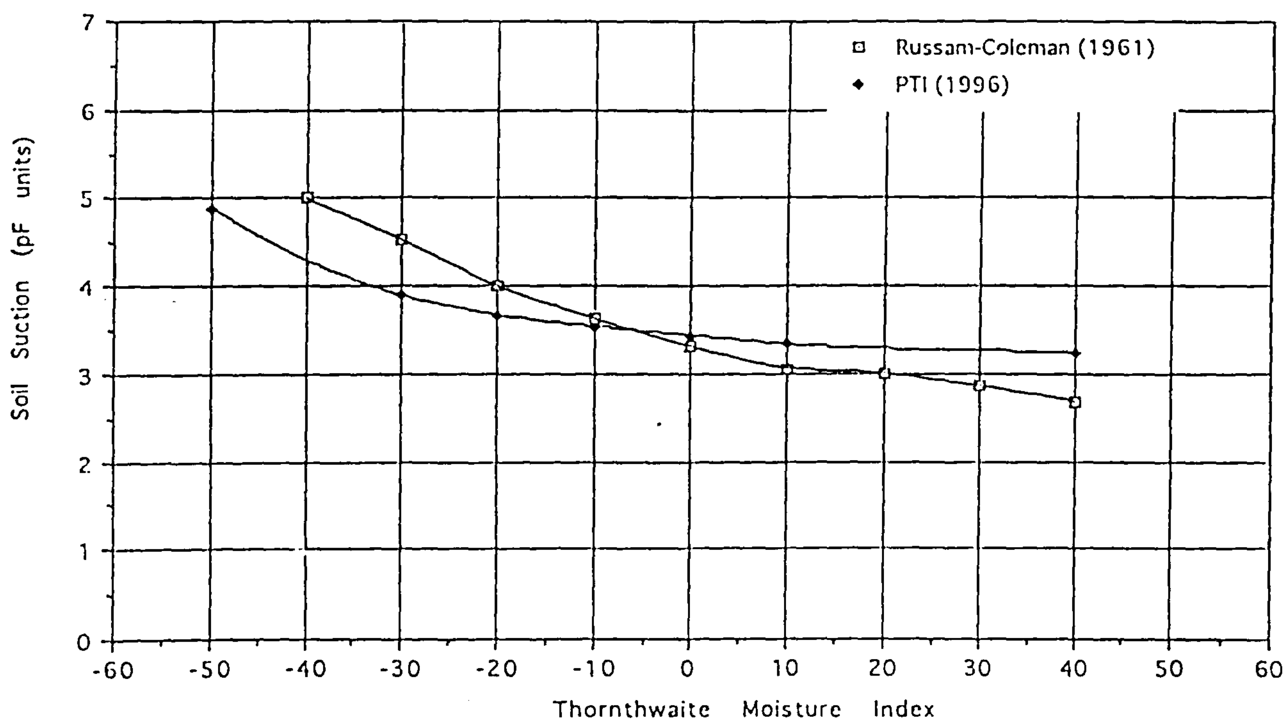

FIGURE 1 Comparison of Post-Tensioning Institute and Russam-Coleman soil suction variation with Thornthwaite moisture index.

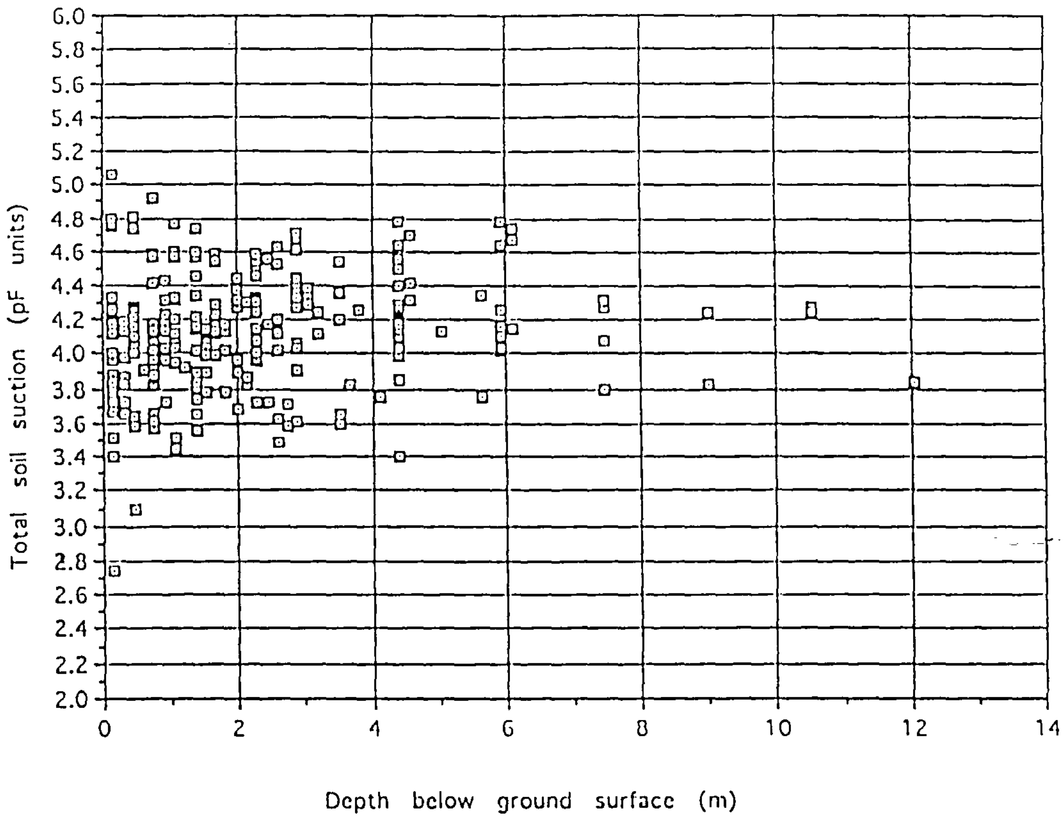

FIGURE 2 Total soil suction profile, Dallas–Fort Worth, 1995.

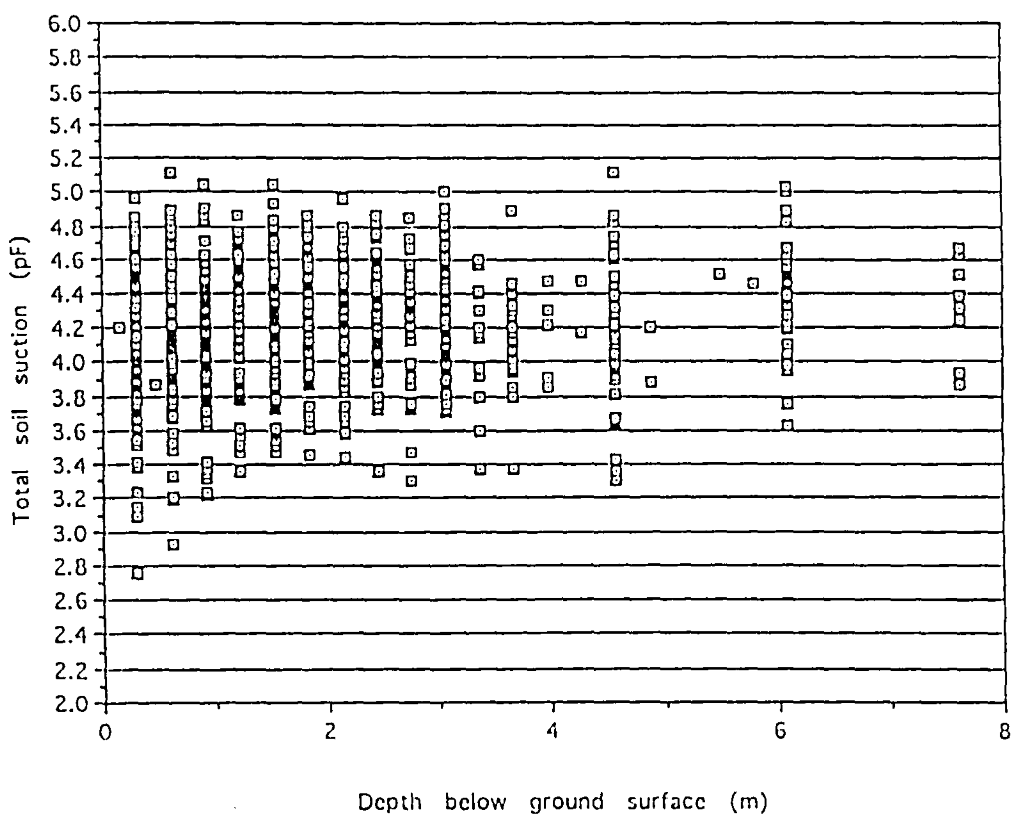

FIGURE 3 Total soil suction profile, Dallas–Fort Worth, 1996.

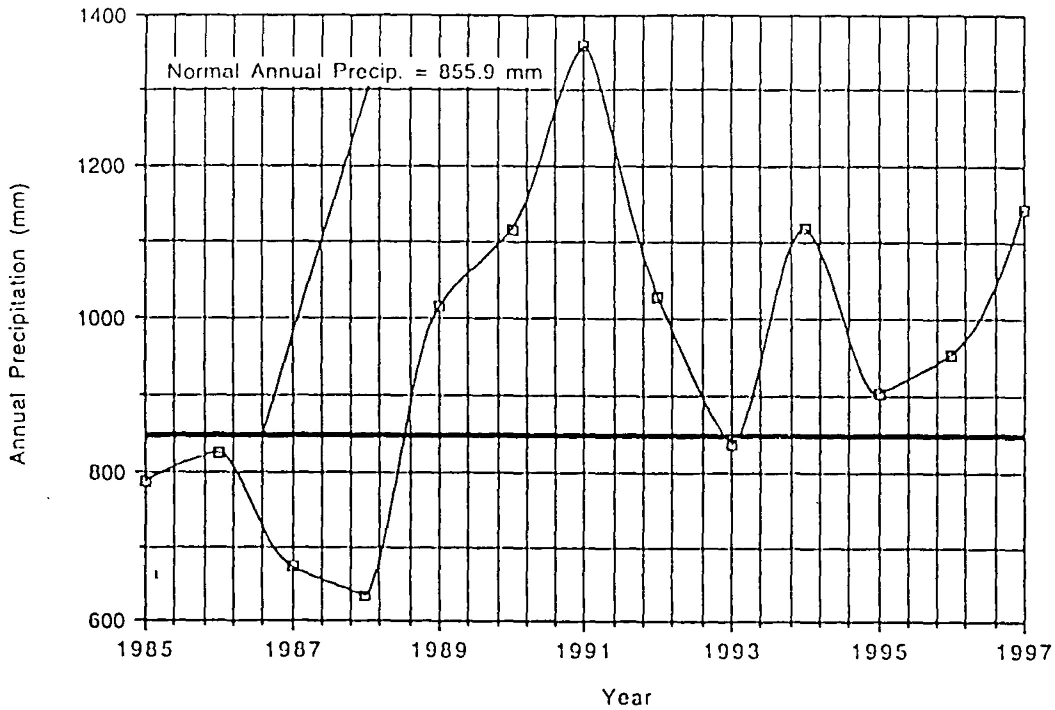

FIGURE 5 Annual precipitation for Dallas–Fort Worth, 1985–1997.

Gay (5) developed a mathematical model to calculate the soil's desorption suction-versus-volumetric water-content characteristic curve. Lytton (4) concludes that higher equilibrium soil suction values can be produced by using the analysis of the desorption suction-versus-volumetric water-content characteristic curve on any given site, and that using the relationships can provide an equilibrium soil suction value routinely.

CONCLUSIONS

On the basis of the results of this research, the following conclusions can be drawn:

1. The measured average total soil suction value or constant soil suction value for the DFW area measured over the last 2.5 years is estimated to be on the order of 979 kPa.
2. The range of total soil suction values was the greatest at the surface and decreased with depth with a minimum measured value of 55 kPa (2.75 pF) and the maximum measured value of 11 246 kPa (5.06 pF).
3. The range of measured total soil suction measurements decreased from 1995 to 1997, which generally corresponds to higher precipitation during the later part of 1996 and 1997.
4. Additional research into the distribution of total soil suction values with depth in the DFW area and across the United States is necessary to understand the variability and range of the total soil suction values used in pavement and slab structure design.
5. Additional research is needed to quantify the differences between the total soil suction values of residual clay and shaley clay soils that weathered from the parent material in place and sandy clay soils deposited from recent river and creek alluvial and fluvial depositional processes.

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PAPER NO. 14

Method of Designing Slab-On-Ground Foundations in Arid and Semi-Arid Climates for Irrigation Lawn Watering and Flower Bed Conditions

The Post-Tensioning Institute Manual for the Design and Construction of Post-Tensioned Slabs on Ground provides estimates of edge moisture variation distances, e_m, for the design of slabs where the available moisture is from rainfall alone and there are no hazardous site conditions such as trees, slopes, or drainage toward the building. This does not match the description of a common condition for which slabs must be designed in which lawns are irrigated or flower beds are placed next to the building. This paper presents a method to design slabs against the uplift that occurs at the edge of the slab due to lawn irrigation and flower beds that is consistent with the PTI method.

The PTI slab design method requires two properties of the expansive soil mass which supports the slab: (a) the differential movement, y_m in inches and (b) the edge moisture variation distance, e_m, in feet. The process described below will arrive at these two soil mass properties. The first part is to determine the differential movement, y_m, and the second part is to determine the edge moisture variation distance, e_m.

1. Differential Movement, y_m

The steps to determine y_m are to determine the following factors:

a. Suction Compression Index, γ_h
b. Long Term Equilibrium Suction Level at Depth, pF_0
c. Surface Suction due to Irrigation or Flower Beds, pF_f
d. Depth of Perimeter Moisture Barrier, d_b
e. Edge Lift Differential Movement, y_m

Although all of these may be input into the computer program VOLFLO (a DOS-based program dating from 1988) or VOLFLO2 (a windows-based program with more capabilities, dating from 1998) and the differential movement, y_m computed directly, two tables have been prepared to give typical results.

The Suction Compression Index (Item a) may be estimated using the chart method which is presented later. A two typical values of γ_h have been used in developing these tables, 0.03 and 0.09. The long term equilibrium suction level at depth, pF_0 (Item b) may be estimated from the following table (Table 1). These values were calculated using the new windows-based program, VOLFLO2, using as a basis a clay soil that reproduces the empirical data published by Russam and Coleman (1) in 1961. The depths of the moisture active zone were found to be between 11.4 and 15.3 feet depth. These suction values at depth are the values that come to equilibrium over a long period of time (several thousand years) with the moisture available from the weather and the loss of moisture due to evaporation from bare soil or transpiration from natural vegetation that grew on the soil surface under pre-development conditions.

Table 1. Equilibrium Suction Values, pF

PRE-DEVELOPMENT SURFACE CONDITIONS

Natural Vegetation	Thornthwaite Moisture Index	Bare-Soil
4.25	- 30	5.10
4.00	- 20	4.80
3.85	- 10	4.45
3.60	0	4.00
3.40	10	3.75
3.20	20	3.50
3.10	30	3.30

As seen in Table 1, as the Thornthwaite Moisture Index moves from a wet climate (+30) to a dry climate (-30) the equilibrium suction that develops at depth depends upon the conditions at the surface of the soil during the long time that was required to reach equilibrium . Thus, for example, a soil profile in a climate with a Thornthwaite Index of –30 may equilibrate at a suction between pF-values of 4.25 and 5.10.

Soil profiles with a different amount of fine clay in the soil than in the clay used in this example will equilibrate at different levels of suction. Also, if the surface vegetation condition varied from bare soil to vegetative cover from year to year, the equilibrium suction at depth will be an intermediate value between 4.25 and 5.10.

The surface suction due to lawn irrigation or flower beds pF_f, (Item c) will never be wetter than a pF of 2.5 even under conditions of excessive water. This has been measured in the field and confirmed repeatedly by actual observation. It is a real wet limit for clays because it is dictated by the physical laws governing the exchange of moisture between the soil and the air at their interface. In lieu of directly measuring the suction under different irrigation conditions, the following table may be used for different levels of watering in an arid to semi-arid climate.

Table 2. Surface Suction Values

Watering Condition	Surface Suction, pF
Excessive Watering	2.5
Normal Watering	3.0
Sporadic Watering	3.5

Moisture barriers (Item d) may be used around the perimeter of the building to reduce the effect of lawn irrigation of flower beds. Depths up to 4 feet have been used. Greater success comes from barriers that are deeper than 2 feet below the ground level. The two tables of typical differential movement values assume two depths of moisture barrier: 0 and 4 feet. The depth of the perimeter beam below ground level may be used a moisture barrier.

The edge lift differential movement, y_m, (item e) for several typical cases may be read from Tables 3 and 4. Table 3 shows typical results of edge lift differential movement, y_m, due to lawn irrigation or watering in semi-arid and arid areas where the long-term Thornthwaite Moisture Index is 0, -10, -20, and –30. Table 4 shows typical results of y_m due to flower beds, also in the same semi-arid and arid areas. For the purposes of these calculations the surface suction in the flower bed is assumed to remain constant to a depth of 4 feet.

Table 3. Edge Lift Differential Movement, y_m, Inches, Due to Lawn Irrigation or Watering

			Pre-Development Condition: Natural Vegetation				Bare Soil			
	Depth of Barrier, ft	Thornthwaite Moisture Index	0	-10	-20	-30	0	-10	-20	-30
		Suction at Depth pF	3.60	3.85	4.00	4.25	4.00	4.45	4.80	5.10
		Surface Suction, pF_f								
$\gamma_h = 0.03$	0	2.5	0.85	1.11	1.28	1.58	1.28	1.84	2.30	2.71
		3.0	0.37	0.59	0.73	1.00	0.73	1.23	1.65	2.03
		3.5	0.05	0.19	0.29	0.50	0.29	0.69	1.05	1.40
	4	2.5	0.05	0.11	0.15	0.24	0.15	0.32	0.49	0.64
		3.0	0.00	0.01	0.03	0.08	0.03	0.14	0.26	0.39
		3.5	0.00	0.00	0.00	0.00	0.00	0.02	0.09	0.19
$\gamma_h = 0.09$	0	2.5	2.51	3.33	3.85	4.76	3.85	5.52	6.90	8.14
		3.0	1.11	1.77	2.20	3.00	2.20	3.68	4.95	6.10
		3.5	0.14	0.56	0.88	1.49	0.88	2.06	3.16	4.21
	4	2.5	0.01	0.32	0.45	0.72	0.45	0.97	1.46	1.93
		3.0		0.03	0.09	0.24	0.09	0.41	0.78	1.17
		3.5	0.01	0.01	0.01	0.01	0	0.06	0.28	0.56

Table 4. Edge Lift Differential Movement, y_m, Inches, Due to Flower Beds

			Pre-Development Condition: Natural Vegetation				Bare Soil			
Volume Change Co-Efficient γ_h	Depth of Barrier, Ft	Thornthwaite Moisture Index	0	-10	-20	-30	0	-10	-20	-30
		Suction at Depth pF_0	3.60	3.85	4.00	4.25	4.00	4.45	4.80	5.10
		Surface Suction, pF_f								
$\gamma_h = 0.03$	0	2.5	1.39	1.86	2.13	2.60	2.13	2.96	3.61	4.16
		3.0	0.47	0.94	1.22	1.68	1.22	2.05	2.69	3.24
		3.5	0.44	0.02	0.30	0.76	0.30	1.13	1.77	2.32
	4	2.5	0.29	0.49	0.62	0.85	0.62	1.05	1 .43	1.78
		3.0	0.03	0.14	0.23	0.41	0.23	0.58	0.90	1.21
		3.5	0.00	0.00	0.01	0.09	0.01	0.20	0.45	0.71
$\gamma_h = 0.09$	0	2.5	4.17	5.57	6.40	7.79	6.40	8.89	10.82	12.47
		3.0	1.42	2.82	3.65	5.04	3.65	6.14	8.07	9.72
		3.5	0.14	0.56	0.91	2.29	0.91	3.39	5.32	6.97
	4	2.5	0.87	1.46	1.85	2.55	1.85	3.16	4.30	5.34
		3.0	0.10	0.41	0.68	1.23	0.68	1.73	2.71	3.64
		3.5	0.00	0.00	0.03	0.27	0.03	0.60	1.35	2.13

Several observations are evident from the y_m - values in Tables 3 and 4.

a. The use of a 4-foot deep moisture barrier greatly reduces the differential movement.
b. The amount of watering, either of lawn irrigation or of flower beds makes a large difference in the amount of differential movement.
c. The suction compression index of the soil makes a large difference in the amount of differential movement.
d. How dry the soil was under predevelopment conditions makes a large difference in the amount of differential movement.

To the question that may be asked, "How do I find what the pre-development Thornthwaite Index and suction at depth is?," the answer is that you can short-cut the process and ask a soils lab to take samples and measure the suction at depth for you. It is a simple procedure that measures the water content of filter paper according to ASTM Standard D-5298. Some soils labs offer this service and others don't, so some amount of searching may be required.

Another question that may be asked is, "How do I find out the suction compression index, γ_h?" There are several answers here. One is that you can measure it if you have the equipment. Another is that you can estimate it using the chart method that is presented later in Figure 1. It requires that you have the Atterberg limits, and the percent of the soil smaller than the No. 10 and No. 200 sieves and the soil finer than 2 microns. Still another possibility is to estimate it from the Expansion Index of the soil, using the approximate relation

$$\gamma_h = \frac{EI}{1700} \qquad (1)$$

2. Edge Moisture Variation Distance, e_m

The edge moisture variation distance, e_m, for both lawn irrigation or flower beds is dictated by the properties of the soil on site to transmit water beneath the foundation rather than by any dependence upon the long-term climatic moisture balance. A relationship between e_m, and the unsaturated soil diffusivity, α (in cm^2/sec) has been developed in a research project conducted at Texas A&M, Texas Tech, and the University of Texas at El Paso for the Texas Department of Transportation (2). Because water moves slower in dry soil being wetted up, than in wet soil being dried out, the e_m-vs-α relationship is different for the edge lift case (wetting up) than for the edge drying case, the e_m – value being smaller for the wetting up case for a prescribed value of the unsaturated diffusivity coefficient, α.

The unsaturated diffusivity coefficient may be estimated from the following empirical equation which was developed in the referenced TxDOT project.

$$\alpha \left(\frac{cm^2}{sec}\right) = 0.0029 - 0.000162\,(S) - 0.0122\,(\gamma_h) \qquad (2)$$

where S = the slope of the straight line portion of the suction (pF) – vs – gravimetric water content curve for the soil

γ_h = the suction compression index of the soil.

An empirical relation for S was also developed in the TxDOT project and may be used to estimate it for use in the equation for α (Eqn. 2).

$$S = 20.29 + 0.155\,(LL, \%) - 0.117\,(PI, \%) + 0.0684\,(\% - \#200) \qquad (3)$$

where LL, % = the Liquid Limit, in percent
PI, % = the Plasticity Index, in percent
% - #200 = the percent passing the #200 sieve, in percent

The Suction Compression Index, γ_h, may be estimated using the chart method, as described below. Using representative values based on laboratory test results in each significant layer, the following parameters are required to estimate the Suction Compression Index.

Liquid Limit, in percent (LL, %)
Plasticity Index, in percent (PI, %)
Percentage of Soil passing the No. 200 sieve in percent (% - #200)
Percentage of Soil finer than 2 microns (% - 2 microns)

The axes on the chart are an Activity Ratio, Ac and a Cation Exchange Activity, CEAc. The definition of these are as follows:

$$Ac = \frac{PI, \%}{\dfrac{\% - 2 \text{ microns}}{\% - \# 200} \times 100} \qquad (4)$$

$$CEAc = \frac{(LL, \%)^{0.912}}{\dfrac{\% - 2 \text{ microns}}{\% - \# 200} \times 100} \qquad (5)$$

The values on the chart are values of γ_{100}, the Suction Compression Index for 100 percent fine clay. The desired estimate of γ_h is given by

$$\gamma_h = \gamma_{100} \frac{\% - 2 \text{ microns}}{\% - \#200} \qquad (6)$$

The chart, which is shown below, is a adaptation of the Chart developed by McKeen (3) and appears in Appendix A of the Texas State Board of Registration for Professional Engineers website as recommended practice for the design of post-tensioned slabs on ground.

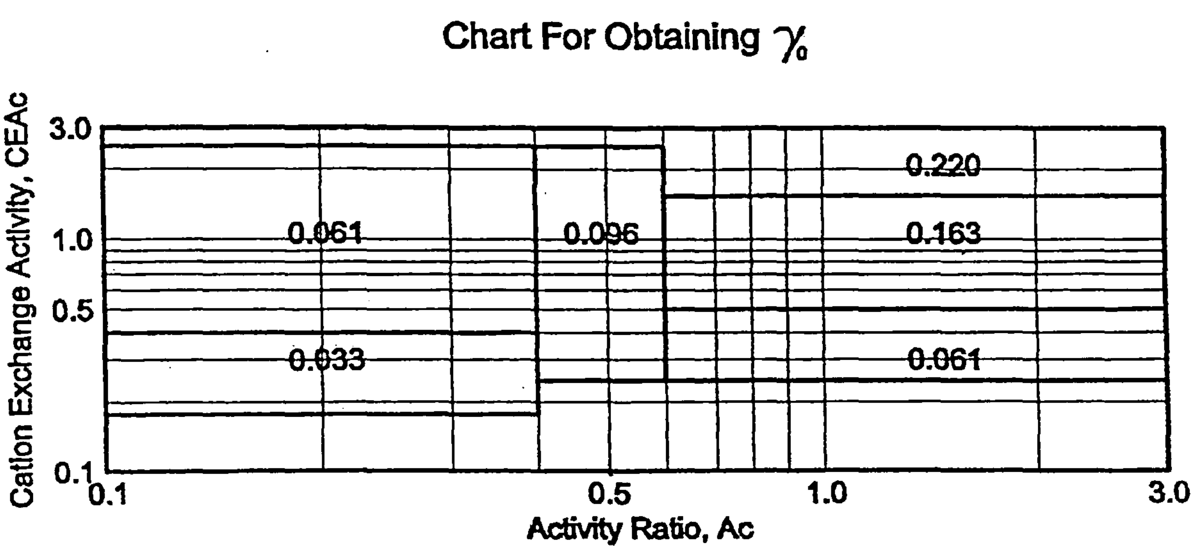

Figure 1. Chart for Obtaining γ_{100}.

This formula for γ_h is predicated upon all of the soils being finer than the No. 200 sieve. Many expansive soils have substantial portions that are larger than this and the chart value of γ_h must be corrected for the percent of the soil that is larger than the No. 200 sieve. The correction must be done on a volumetric rather than on a weight basis. The correction method recommended here is adapted from the method that was developed by the U. S. Department of Agriculture Natural Resources Conservation Service and published by Holmgren (4).

$$(\gamma_h)_{corr} = \gamma_h \left[\frac{100}{F\left(\dfrac{\gamma_t - wet}{\gamma_d - dry}\right) + (100 - F)} \right] \tag{7}$$

$$F = \frac{100}{1 + \dfrac{J}{100 - J} \bullet \dfrac{(\gamma_d)\ wet}{(G_s)_{coarse}\ x\ \gamma_w}} \tag{8}$$

Where

F = percent by volume of the fraction of the soil smaller than the No. 10 sieve (2.0mm) as a percent of the total soil volume

$\gamma_t - wet$ = total unit weight of the soil at the soil wet limit around a pF of 2.5 for clay

$\gamma_d - dry$ = dry unit weight of the soil at its natural water content (around standard proctor optimum water content or shrinkage limit)

J = % of the soil by weight that is larger than the No. 10 sieve (2.0mm)

$(G_s)_{coarse}$ = specific gravity of the soil particles larger than 2.0 mm (This may be presumed to be 2.65)

γ_w = unit weight of water

This volumetric correction will reduce the γ_h – value for all soil particles larger than the No. 10 sieve (2.0mm). The NRCS found that no reduction in the γ_h – value is warranted for soils with particles smaller than the No. 10 sieve. The values of γ_t – wet and γ_d – dry should be for the soil in its natural state and may be estimated for the purpose of this correction.

The chart presented below in Figure 2 implements Equations (2) and (3) to permit the estimation of e_m for edge lift condition. One more modification to the value of the unsaturated diffusivity coefficient, α, has been recommended by the Texas State Board of Registration for

Professional Engineers to take into account the effects of roots, fissures, cracks, or joints in the soil. The α - value that should be used for design should be α' as in Equation (9) below:

$$\alpha'\left(\frac{cm^2}{sec}\right) = \alpha \times F_f \tag{9}$$

where F_f = the crack fabric factor which is taken from the following table, Table 5.

α = unsaturated diffusion coefficient from Equation (2) or Figure 2.

Table 5. Crack Fabric Diffusity Factors

Soil Condition	F_f
Soil profiles contain few roots, fractures, joints	1.0
Soil profiles contain some roots, fractures, joints	1.3
Soil profiles contain many roots, fractures, joints	1.4

The unsaturated diffusion coefficient, α, should be calculated for each significant soil layer to a depth of nine feet by the procedure outlined above. The evaluation of the edge moisture variation distance, e_m, requires using a weighted average of three for the top three feet, two for the next three feet, and one for the bottom three feet.

The values of y_m and e_m determined in this way may be used in the PTI SLAB program or with the design equations in the PTI manual (5) to design the slab for the edge uplift conditions that are caused by lawn irrigation or flowerbeds.

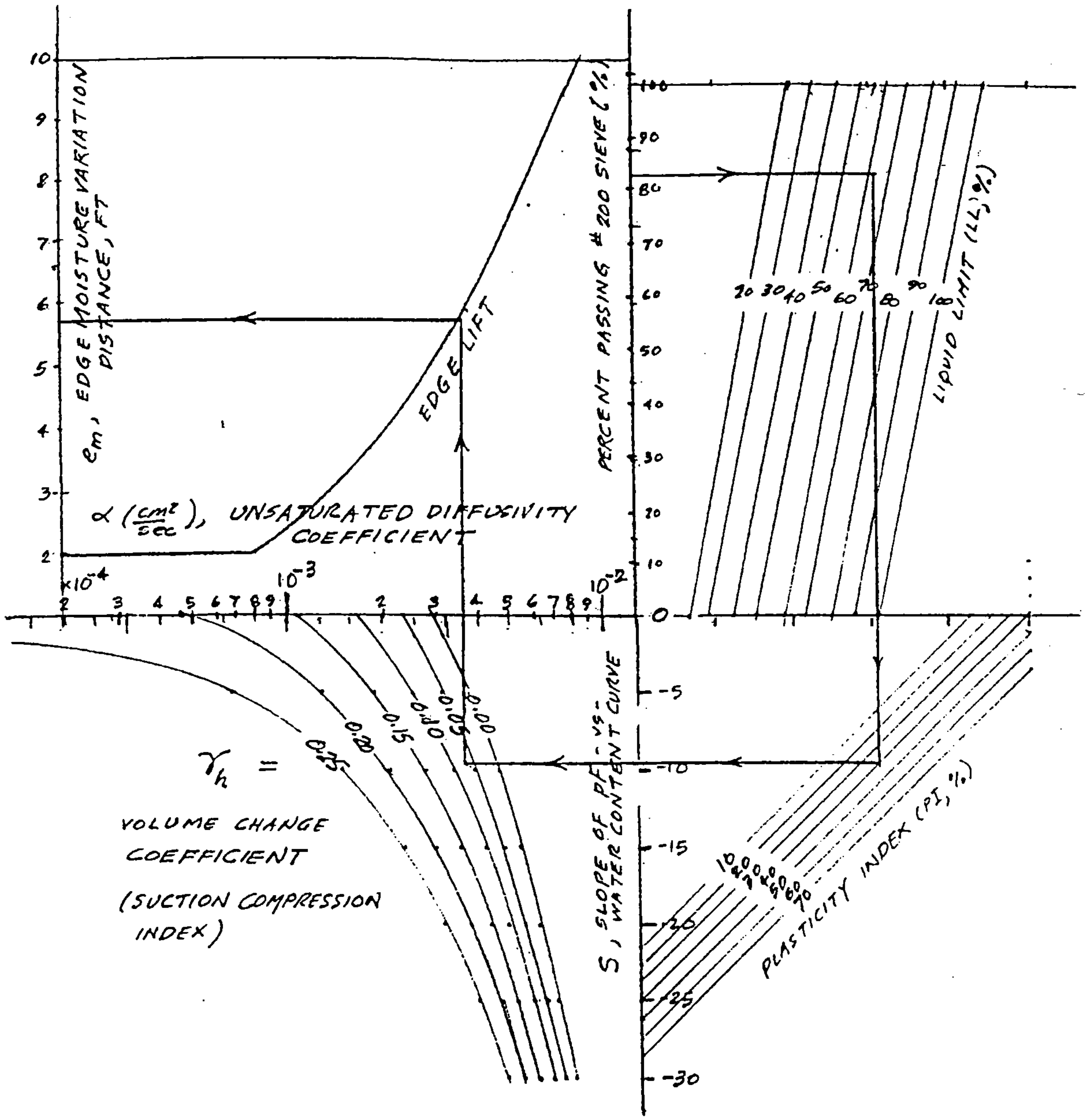

Figure 2. Nomograph to Determine e_m, the Edge Moisture Variation Distance for the Edge Lift Case.

50 year
design period

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Appendix I. Example Calculations

The example calculations shown below are for one soil, the properties of which are noted in each example. Three examples are presented. Example No. 1 shows how to use Figure 1, to get the Suction Compression Index (or volume change Coefficient), γ_h for fine grained soil. Example No. 2 shows how to correct this value of γ_h to account for the amount of coarse-grained particles that are contained in the soil, using Equations (7) and (8) in the text. Example No. 3 shows how to use Figure 2 to estimate e_m, the edge moisture variation distance for the edge lift made of slab distortion. Figure 2 is a nomograph that implements Equations (2) and (3) in the text.

1. Example Use of Figure

Liquid Limit:	62%	
Plastic Limit:	22%	
Plasticity Index:	40	
% Passing #200 Sieve	82	
% Finer than 2 microns:	38	
% Fine Clay	0.466	
Cation Exchange Capacity (LL):	0.912	= 43.1 milliequivalents per 100 gms of dry soil
Cation Exchange Activity (CEAc):	0.92	
Activity Ratio (Ac):	0.86	
γ_{100}:	0.163	
γ_h = % Fine Clay x γ_{100}:	0.076	

2. Example Use of Correction for Coarse Grained Soil (Equations 7 and 8)

% Passing # 10 Sieve:	94
% Larger than #10 Sieve	6
% Passing #200 Sieve	82
% Finer than 2 microns:	35
γ_t, Wet Total Unit Weight:	103.0 lb/cu.ft.
γ_d, Dry Unit Weight:	95.0 lb/cu.ft.
$(G_s)_{coarse}$	2.65

Equation 8:

$$F = \frac{100}{1 + \dfrac{6}{100-6} \bullet \dfrac{103.0}{2.65 \text{ x } 62.4 \text{ pcf}}}$$

$$F = \frac{100}{1 + 0.0638 \text{ x } 0.6229}$$

$$F = 96.2$$

Equation 7:

$$(\gamma_h)_{corr} = 0.076 \left[\frac{100}{96.2 \left(\dfrac{103.0}{95.0}\right) + (100 - 96.2)} \right]$$

$$(\gamma_h)_{corr} = 0.076 \text{ x } 0.925$$

$$(\gamma_h)_{corr} = 0.070$$

Example Use of Figure 2

% Passing # 200 Sieve:	82
Liquid Limit:	62%
Plasticity Index:	40%
Volume Change Coefficient, γ_h:	0.07
Unsaturated Diffusivity Coefficent, α:	3.6 x 10^{-3} cm^2/sec
Edge Moisture Variation Distance, e_m:	5.6 feet

PAPER NO. 15

Nomographic Calculation of Linear Extensibility in Soils Containing Coarse Fragments[1]

George G. S. Holmgren[2]

ABSTRACT

Linear extensibility is a variable being used by the Soil Survey Laboratories to characterize the expansion-contraction properties of soil. It is calculated from bulk density data and must be corrected for coarse fragments when they are present in the field. A pair of nomographs is presented to facilitate these calculations. The volume percent of fine-earth fabric (F) is determined from the first nomograph and transposed to the second nomograph where it serves to correct the extensibility as determined on the fine-earth fabric. The value F may also be used to facilitate conversion of other laboratory data to a field basis where coarse fragments are present.

Additional Key Words for Indexing: fine-earth fabric, shrinkage, linear shrinkage, COLE.

The Soil Survey Laboratories have in recent years provided data which can be used to characterize the expansion-contraction properties of a soil under changes in moisture stress. The resulting variable, termed linear extensibility, is discussed at some length in the accompanying papers by Franzmeier and Ross[3] and Grossman et al.[4] This paper is restricted to consideration of nomographic solution to the equations for calculating linear extensibility from bulk density and particle-size distribution data. The textbook by Douglas and Adams[5] was helpful in developing these nomographs.

i.e. suction

It should be noted that Soil Survey Laboratory data are often reported as a "coefficient of linear extensibility" or COLE. Linear extensibility (LE) and COLE are simply related as follows:

$$\text{COLE} = \text{LE} \div 100. \qquad [1]$$

CALCULATIONS

Linear Extensibility

Bulk density values are usually determined on the less-than-2-mm or "fine-earth" fabric. If no coarser fragments are present, linear extensibility (LE′) is calculated directly from the moist and dry bulk densities as follows:

$$\text{LE}' = 100\left[\left(\frac{Db_d}{Db_m}\right)^{1/3} - 1\right] \qquad [2]$$

where

Db_m = the fine-earth fabric bulk density at ⅓-bar water content;

Db_d = the oven dry fine-earth fabric bulk density.

If coarse fragments are present, they must be corrected for in the calculation. The adjusted value used by the Soil Survey Laboratories is a weighted mean, and is calculated as follows:

$$\text{LE} = 100\left[\left(\frac{100}{F\cdot\frac{Db_m}{Db_d} + (100 - F)}\right)^{1/3} - 1\right] \qquad [3]$$

where

F = the fine-earth (< 2-mm) fabric as a volume percent of the total fabric.

Fine-Earth Fabric Volume Percent

If the > 2-mm material is recorded as a weight percent, F may be calculated as follows:

$$F = \frac{100}{1 + \frac{Jw}{100 - Jw}\cdot\frac{Db}{Dp}} \qquad [4]$$

where

Jw = the 2- to j-mm fragments as a weight percent of the < j-mm fabric (j is usually 20 mm or 75 mm, depending on sampling procedure);

Db = bulk density of < 2-mm fabric (moist or dry);

Dp = density of > 2-mm fragments—assumed 2.65 unless otherwise specified.

If the soil contains fragments > 20 mm (or 75 mm), it is often the practice to make an estimate of the volume percentage of these larger fragments and include this value in the field description. The fine-earth fabric volume percent must then be adjusted again to account for these larger fragments. This is accomplished as follows:

$$F' = F\left(1 - \frac{Kv}{100}\right) \qquad [5]$$

where

F' = F adjusted for field volume estimate of coarser fragments;

Kv = the j- to k-mm fragments as a volume percent of the < k-mm fabric (k is the largest size fragment included in the estimate; it may be any size but is usually 250 mm or less).

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[2] Soil Scientist.

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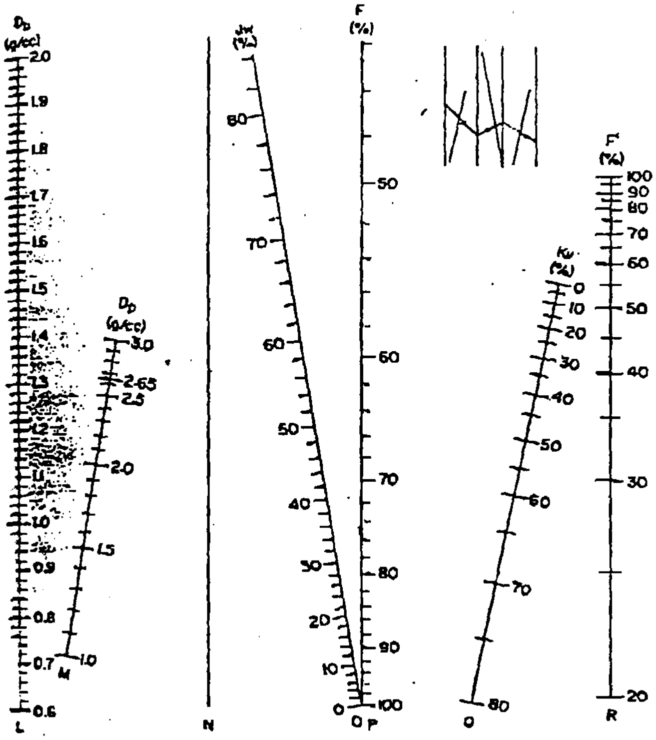

Fig. 1—Fine-earth fabric volume percent.

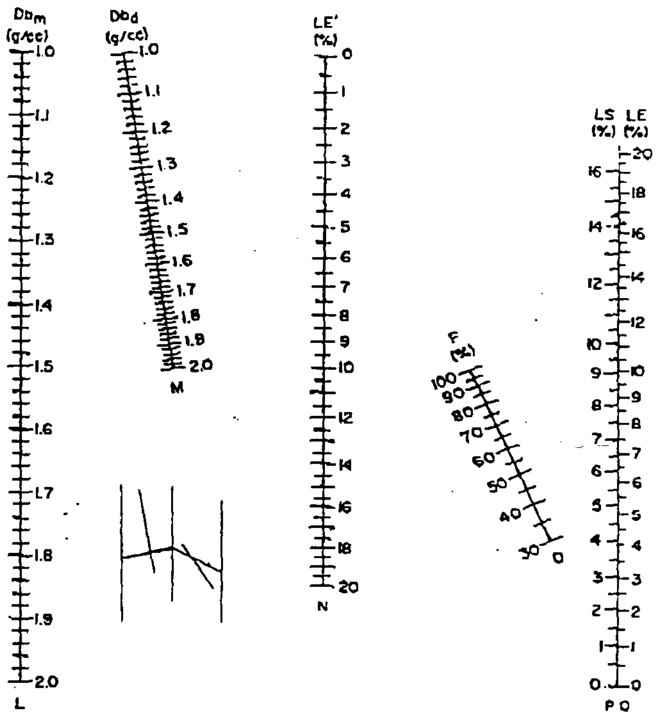

Fig. 2—Linear extensibility and linear shrinkage.

F, when divided by 100, has general use as a coefficient or factor for converting laboratory data to a field basis. The datum, determined on a weight basis of < 2-mm material, is multiplied by the bulk density to convert to a volume basis of < 2-mm fabric. Multiplication by $F/100$ then corrects for the presence of > 2-mm material.

NOMOGRAPHIC PROCEDURE

Fine-Earth Fabric Volume Percent

A value for *F* may be obtained from Fig. 1, as follows:

Pass a line through *Db* on scale *L* and *Dp* on scale *M*;
Rotate about intersection on N and pass through *Jw* on scale O;
Read *F* on scale *P*.

Example

$Db = 1.50$, $Dp = 2.65$.
$Jw = 40\%$, $F = 73\%$.

If Jw falls outside the scale range, the calculation can be accomplished by transposing the scales for *Jw* and *F*; i.e., read Jw on *P* and *F* on O.

Example

$Db = 1.50$, $Dp = 2.65$.
$Jw = 86\%$, $F = 22\%$.

If this procedure is used, however, the following nomographic adjustment for *Kv* is invalid.

If there are no larger fragments, further corrections are unnecessary. If a volume estimate of *j*- to *k*-mm fragments is available, continue as follows:

Pass a line from *F* on scale *P* through *Kv* on scale Q;
Read the combined volume estimate, *F'*, on scale R.

Example

$F = 73\%$, $Kv = 45\%$.
$F' = 40\%$.

Linear Extensibility

With a value for *F* (or *F'*) in hand, it is possible to calculate linear extensibility (LE) on Fig. 2 as follows:

Pass a line through Db_m on scale L and Db_d on scale M;
Read LE' on scale N.

Example

$Db_m = 1.30$, $Db_d = 1.58$.
LE' = 6.7% (COLE = 0.067).

If there are no fragments > 2 mm, LE' = LE and the calculation is finished. If coarser fragments are present, take *F* (or *F'*) from Fig. 1 and proceed as follows:

Pass a line from LE' on scale N through F on scale O;
Read LE on scale Q.

Example

LE' = 6.7%, F = 40%.
LE = 2.5% (COLE = 0.025).

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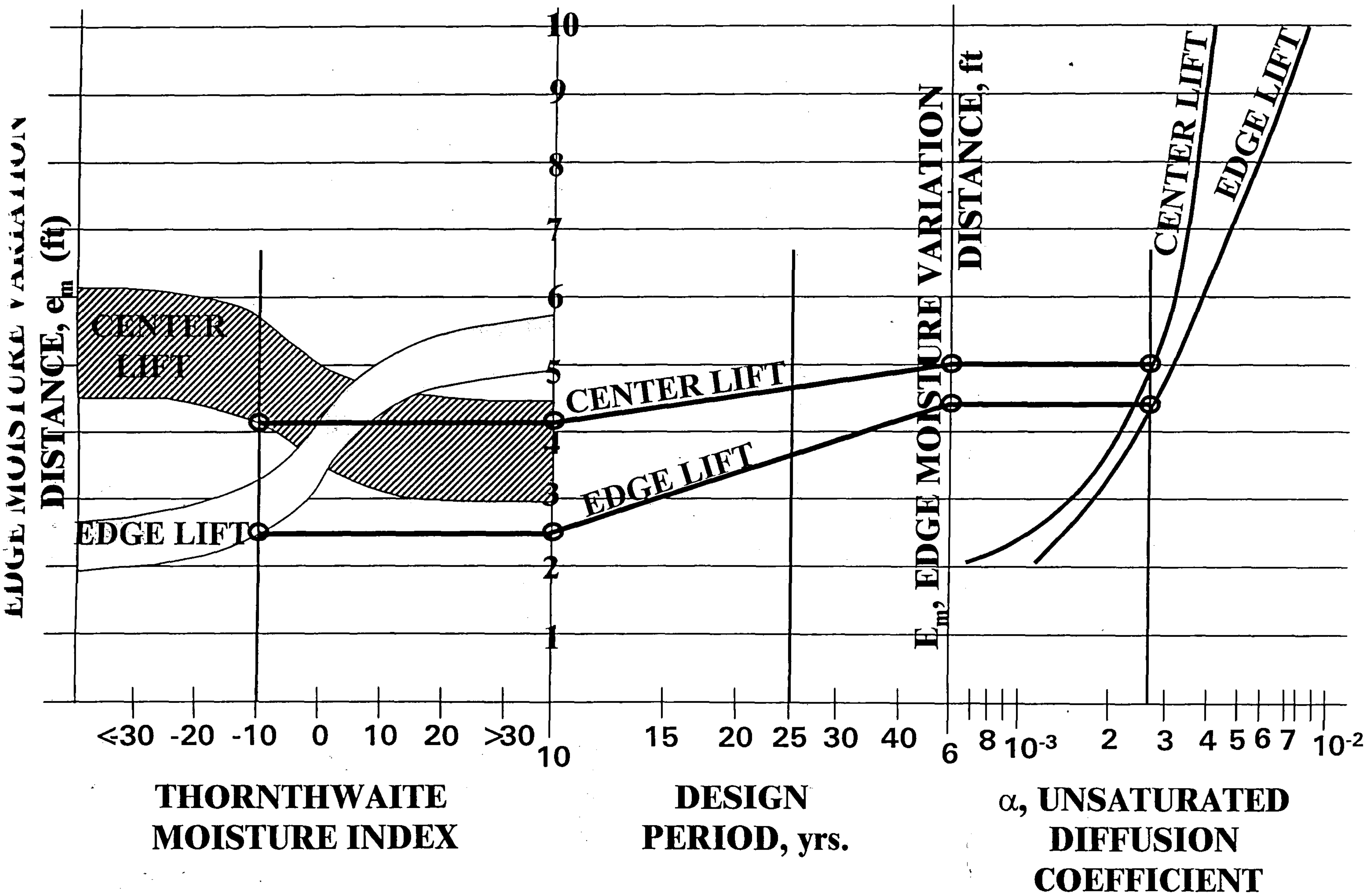


EDGE MOISTURE VARIATION DISTANCE, e_m (ft)
CENTER LIFT
EDGE LIFT
CENTER LIFT
EDGE LIFT
E_m, EDGE MOISTURE VARIATION DISTANCE, ft
CENTER LIFT
EDGE LIFT
1 2 3 4 5 6 7 8 9 10
<30 -20 -10 0 10 20 >30
10 15 20 25 30 40
6 8 10^{-3} 2 3 4 5 6 7 10^{-2}
THORNTHWAITE MOISTURE INDEX
DESIGN PERIOD, yrs.
α, UNSATURATED DIFFUSION COEFFICIENT

* Resistivity

$\frac{l}{400}$ l = side to side distance

www. POST-TENSIONING.ORG

Gilgay – 17' o.c. 1' top to Bottom

e_m – edge can go up or down

* – moments in short direction > long direction

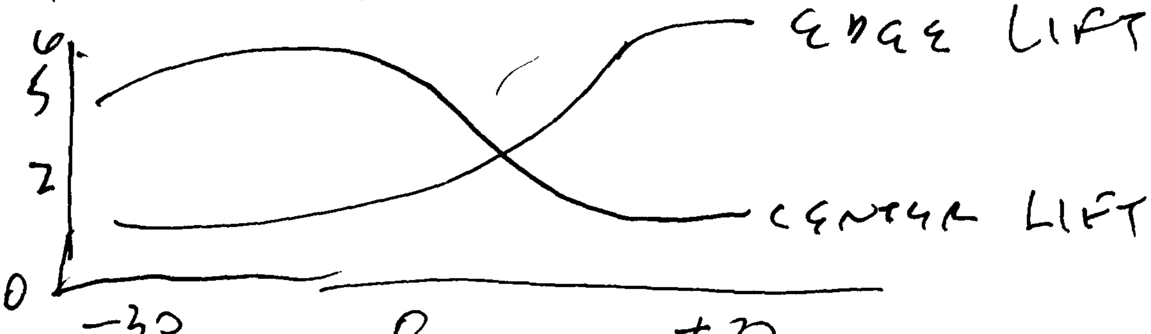
M_{max} @ -12' for large slabs (b) $b > e_m$

maximum shear occurs @ same place

wetter soil ⇒ quiker filter paper comes to equilibrium 5-7 days

Acc to Litton:

depth of active moisture zone is 2' > than the deepest root fiber



2.0 em

lowest H_2O table

1 per change .027/ft of PF

Thornwaite

* Roots active zone (even if roots occurred there 1000 years ago)

* Moisture active zone = root fibers + 2'

$P_F = 3 \Rightarrow 1$ atmosphere

dissolved salts - osmatic

surface suction - matrix

total suction

lowering the table