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A field test for detecting collapse susceptible soils

Macfarlane, Richard Burton, M.S.

The University of Arizona, 1989

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A FIELD TEST FOR DETECTING COLLAPSE SUSCEPTIBLE SOILS

by

Richard Burton Macfarlane

A Thesis Submitted to the faculty of the

DEPARTMENT OF CIVIL ENGINEERING
AND ENGINEERING MECHANICS

In Partial Fulfillment of the Requirements
For the Degree of

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In the Graduate College

THE UNIVERSITY OF ARIZONA

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APPROVAL BY THESIS COMMITTEE

This thesis has been approved on the date shown below:

E. A. Nowatzki
Professor of Civil Engineering

23 June 1989
Date

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I also thank my family and friends for their inspiration and good cheer.

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ABSTRACT

A field test is developed to assess the collapse susceptibility of soils rapidly and inexpensively. The insitu collapse test device measures the vertical deformations which occur in soils when they are subjected to stress and given access to water while under continuous load. Princples of statistics were employed to show that laboratory testing of soil specimens overestimate the magnitude of collapse as measured in the field and that the magnitude of collapse is, in part, a function of the soil moisture content at the time of loading and at the time of collapse. Good correlation was found between the spatial variability of collapsible soils with the location of alluvium terrace deposits and structurally damaged buildings.

CHAPTER 1

INTRODUCTION

Soil subsidence is a major cause of foundation failures in many semi-arid regions. Structural damage due to collapsing soil has been recognized throughout the Tucson area in the form of cracks in the walls and footings of many single-story buildings (Platt 1963; Crossley 1969).

Although such damage is often superficial and not life threatening, it is expensive to repair and reduces the value of the home if left unrepaired. Damage may consist of sagging door frames, broken windows, slumped or separated walls, broken pipes and gas lines, and other hazards. Photographs of several damaged homes within the present study area are presented in Figures 1 through 4.

Statement of the Problem

Soils which form under conditions of rapid deposition of wind or water borne sediments may develop large voids. The saturation of such soils, under the influence of an applied stress, may result in deformation of the intergranular structure. The deformation, referred to as collapse, leads to a rapid decrease in soil volume.

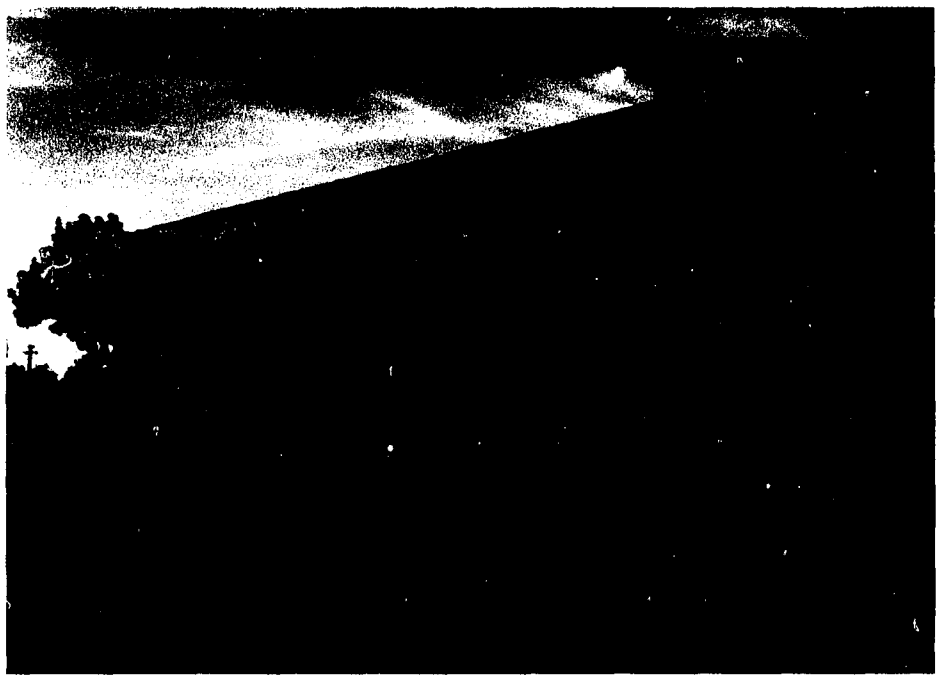
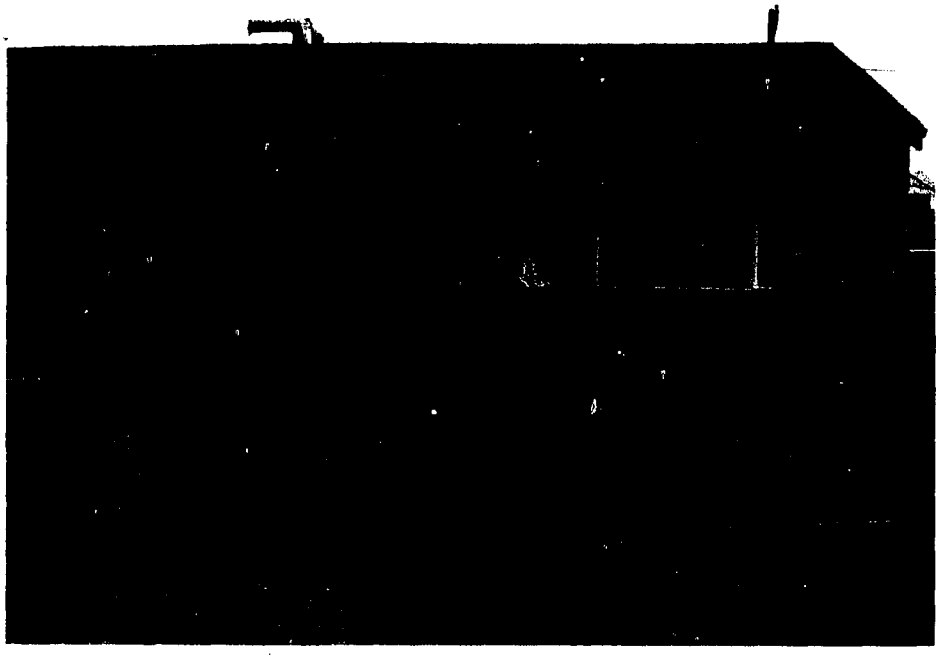


Figure 1. House Near Country Club and Silverlake Roads (within the areas mapped by Crossley 1969; Ali 1987) that exhibited "strong" cracking. Soils in the area exhibit high collapse potential.

A



B

Figure 2-A,B. Complimentary Views of a House in Area of Country Club and Silverlake Roads (note cracking near the water spigot).

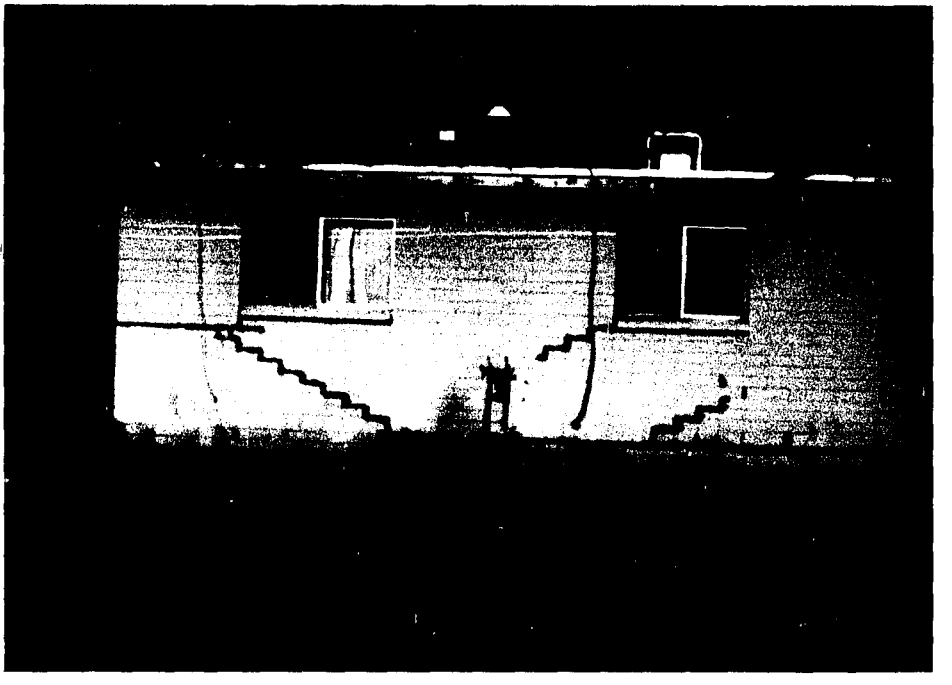


Figure 3. Patching of a Cracked House in the Area of Country Club and Silverlake Roads.

Figure 4. A Condemned Home on 3rd Street, East of the UA Campus (within an area of light cracking and 80% probability of encountering highly collapsible soil) (Crossley 1969; Ali 1987).

A wide variety of theoretical and experimental procedures have been developed to identify collapse-susceptible soils. These procedures are often qualitative observations of elusory soil parameters which may be indigenous only to local soils. Furthermore, no single criterion has yet to be identified which applies to all collapsible soils, although several are believed to have more influence than others (Ali 1987). Many of the identification criteria are also contradictory and not universally accepted (Sabbagh 1982).

Identification criteria for collapse susceptible soils are based on field observations and laboratory tests of core samples. Laboratory "pseudo-consolidation tests" (Nowatzki 1980) are used extensively to predict the magnitude of collapse in the foundation soils of a proposed or existing structure. These tests attempt to duplicate insitu conditions, yet the results are easily tainted by sample disturbance. It is suspected that, due in part to sample disturbance, the true collapse of the soil is not evaluated by pseudo-consolidation test results and that this test generally overestimates the magnitude of insitu collapse.

For this reason, collapse settlement predictions are often calculated as a fraction of the laboratory measurement due to localized adjustments (Dudley 1970; Jennings and Knight 1975; Nowatzki 1980).

Field testing is the most sure and direct means to identify and measure soil collapse. However, the cost in time and labor for such testing is prohibitive for most small projects. The many uncertainties associated with collapsible soils call for developing an expedient and cost effective field test and for further investigating sample retrieval methods.

Purpose

The primary objective of this study is to develop a rapid yet accurate field test for predicting the collapse susceptibility of soils.

A secondary objective is to verify that pseudo-consolidation test results overestimate the amount of collapse a foundation soil might experience.

A final goal is to correlate the data obtained from field tests performed throughout Tucson with previously identified collapse potential zones for foundation failure in the test area.

Scope

Field and laboratory studies were conducted for the purpose of validating the field investigation technique and to challenge the hypothesis that laboratory consolidation tests overestimate the collapse potential of soils.

Results of the field and laboratory investigations were statistically compared to test the aforementioned hypothesis. Further statistical correlations were made between the field results and studies by Ali (1987) to evaluate the collapse criteria developed by Sabbagh (1982). Field investigation results were also compared with the findings of Crossley (1969).

The tentative relationship between insitu moisture content and collapse was developed and its validity evaluated.

CHAPTER 2

REVIEW OF SAMPLE DISTURBANCE INVESTIGATIONS

There is general agreement that it is impossible to obtain a truly undisturbed sample of soil. Sampler geometry and side wall friction produce disturbances which strain the sample. The sampling technique, extrusion, handling, specimen preparation, and laboratory setup processes induce additional disturbances that result in other indeterminant strains. Also, the state of insitu stress is different from the state of sample stress. Neither value of stress is known except by approximation. Therefore, the term "undisturbed" generally refers to a sample in which the disturbance effects have been minimized.

Sampling devices in the United States are described in terms of their area ratio C_a and inside clearance ratio C_i. The area ratio is defined by Hvorslev (1949) as:

$$C_a = \frac{Dw^2 - De^2}{De^2} \qquad \text{(Eq. 2.1)}$$

Dw = outside tube diameter that enters the soil during sampling.

De = inside diameter of the tube cutting edge.

The area ratio approximates the ratio of the soil volume displaced by the sampler to the volume of the sample. The

clearance ratio is given as:

$$C_i = \frac{Ds - De}{De} \qquad \text{(Eq. 2.2)}$$

Ds = inside tube diameter.

The standards used by the The U.S. Army Experiment Station (Marcuson and Franklin 1978) to promote consistent quality sampling of cohesionless soils are:

1. A sampler should have an area ratio (C_a) less than 13 percent;
2. The clearance ratio (C_i) is less than 1.0 but greater than ($1 - 2C_i$).
3. The sample surface should be visibly smooth and devoid of distortions which can be attributed to the sampling or handling technique;
4. There should be no dimensional changes in the sample during shipping or handling.

Collapsible soils are particularly difficult to sample due to the large voids and weakly bonded fabric. Jennings and Knight (1957) recommended the use of block samples carved from a test pit. The block could be trimmed and fitted to a consolidometer ring. Block sampling, however, has proven unpopular. Abdullatif (1969) experienced great difficulty in retrieving samples from the stiff, desiccated soils of the Tucson area. The samples crumbled in transit to the laboratory and much skill and time was required for retrieval and trimming. It has since been determined that relative to other costs

block sampling adds a significant and undesirable cost to a soil investigation for small projects.

Drive sampling is the popular technique used in the Southwest to retrieve soil specimens quickly and inexpensively. Studies show that the drive sampling process tends to densify loose sands and loosen dense sands (Marcuson and Franklin 1978). These changes in density are of particular concern for collapsible soils as are changes in the nature of the grain-to-grain contacts. Badu-Tweneboah (1984) and Duffy et al (1986) determined that the dry density of loessial soils is significantly increased by the drive sampling process, but not by block sampling. They also found the density of drive samples to increase with depth and for clays to densify more than non-plastic soils. The degree of collapse measured in the laboratory for drive samples was determined to be significantly lower than that obtained for block samples; it was also lower than that obtained from field measurements.

Houston et al (1988) suggested that sample disturbance is not as great for collapsible soils as it is for other soils due to a "local shear" form of failure and the collapsible soil's high bonding strength. They found good correlation between a field collapse test and modified collapse potential test results on drive samples despite a sampler area ratio of 56%.

Jennings and Knight (1975) found that samples obtained by pushing thin walled samplers into the soil compressed by as much as 30% of their original length. Sabbagh (1982) compared the dry densities for thick walled and thin walled tube samplers with the insitu dry densities determined by the sand cone method (ASTM D-1557). He found the thin wall and insitu densities to be very close, while the thick wall sample density increased by as much as 20 pcf. The area ratio of the thin wall sampler was 12.9%.

Day (1989) compared consolidation tests results performed on Shelby tube samples obtained by continuous loading with block sample results and found higher strain values for the tube samples. The tube samples were distorted by cracks.

From this review it can be seen that there is much disagreement among the researchers as to the effect of sample disturbance on soil collapse predictions. Most observations of the effects of sample disturbance for collapsible soils have been made relative to difference in compression and changes in dry density. While block sampling has the least effect on density, it is subject to other constraints which limit its usefulness. There also do not appear to be any guidelines for consistent sample retrieval of collapsible soils. This may be due in part to the difficulty of sampling stiff, desiccated soils by

the suggested standards for purely cohesionless soils. There are those who even suggest that the influence of sample disturbance is negligible (Houston et al 1988).

On sample disturbance, Varga (1972) concluded that "the deformation and strength characteristics of loess soils when determined in the laboratory cannot represent, even approximately, the true properties of the soil . . ." It is easily seen from the disparity among research results that effort should be directed towards measuring the sample disturbance effects on collapsible soil investigations.

CHAPTER 3

REVIEW OF COLLAPSIBLE SOILS

Description of Collapse Susceptible Soils

Collapse susceptible soils are characterized by "low insitu dry unit weights (high void ratio), low insitu moisture contents, high unconfined compressive strengths at low moisture contents (about 15%), and the tendency to lose strength rapidly (collapse) when wetted to near-saturation moisture contents while loaded to stress levels above their natural overburden pressures" (Nowatzki and Alfi 1987).

When collapse susceptible soils fail, the result is a more tightly packed particle arrangement. The magnitude of collapse is controlled by the initial moisture content, amount of additional wetting, applied loads, and character of the soil. Dramatically increased moisture content is the principal collapse mechanism. Collapsible soils are typically composed of sand, silt, shale, or flocculated clay platelets. The weakly bonded grains are joined at contact points by clay, by surface tension forces in water which partly fill the capillary spaces, or by chemical weathering products (see Figure 5).

The clay portion of collapse susceptible soils is derived from two sources:

1. Clay colloids are introduced in suspension and deposited to buttress cups by periodic evaporation of the suspensions as they percolate through the pore system;

2. The silt and sand particles may have had a clay coating upon deposition.

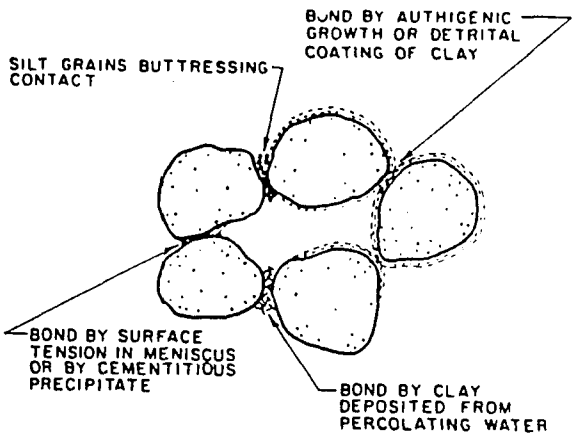


Figure 5. Schematic of Collapsing Soil (From Gillott 1987)

Deposition of Collapsible Soils

The most extensive deposits of collapse susceptible soils are wind-deposited sands and silts (loess). Alluvial flood plains, fans, mud flows, colluvial deposits, residual soils, and volcanic tuffs may also contain collapsible soils.

Wind blown deposits are characteristic of arid regions where the water table is at great depth below the ground surface. The fine silt sized particles are bonded by clay bridges or calcareous material to form a porous structure of low density (less than 90 pcf) and high

compressive strength (greater than 144 ksf). A surface crust of impermeable clay develops, thus protecting the open fabric from weathering.

Loess deposits cover large areas of the earth's surface. Loess is found in about 17% of Eastern Europe, portions of Russia, Siberia and large areas of China, South Africa, and New Zealand. The plains of Argentina and Uruguay also contain loess deposits. About 17% of the United States, predominately in the West and Midwest, contains loessial soils (Clemence 1981).

Not all loessial soils are collapsible. Soils of Southeastern Washington State and Northern Idaho, known as Palous-loess, have a structure that is of a closed fabric with a dry density of 95 pcf. The dominant clay cementing agent is illite, which does not readily adsorb water. Montmorillonite, which does adsorb water, is the cementing agent of most collapsible soils in the United States.

Water Lain Deposits

Water lain deposits are primarily loose sediments which form alluvial fans. Flash floods or mud flows from small watersheds deposit the material at infrequent intervals separated by long dry periods. The soils dry rapidly and do not consolidate under their self-weight. They aren't saturated again until another flood arrives. Over time, many feet of low density soils may form in this

way. Characteristically, the soil particles are weakly bonded by soluble salts or colloidal particles lodged between the larger grains. The flows typically contain large amounts of clay and rock.

Bull (1964) showed that there is maximum subsidence in water lain deposits when the clay content is about 12% of the solids. There is little subsidence when the clay content is below 5%, while swelling may result when the clay mineral content is greater than 30%. Subsidences of up to 15 feet have been observed in the water lain deposits of the San Joaquin Valley, California (Knodel 1981).

Residual Soil

Residual soils are the product of insitu weathering of rock. Chemical and physical weathering produce soil layers which range in thickness from a few inches to several feet, depending on the climate and physico-chemical aspects of the region. The material may consist of gravel, sand, silt, clay colloids and, in instances, organic matter. Leaching of soluble and colloidal material, due to eluviation, is the principal cause of fabric formation (Brink and Kantey 1961).

Environments best suited for the formation of residual soils have relatively high annual rainfall.

Residual soils have been documented in Southern Africa, Southern California, Northern India, and Brazil.

Moisture Content

The natural moisture content of a soil is a significant limiting parameter of soil collapse. This has been tentatively demonstrated by field and laboratory investigations and by statistical analysis.

Badu-Tweneboah (1984) evaluated the stress response of a collapsible soil in the field under changes in soil moisture conditions. He found that the amount of collapse which occurred when the soil became saturated decreased with higher initial moisture content. This observation confirmed that reported by Sabbagh (1982) for specimens tested in the laboratory by a pseudo-consolidation test. A subsequent field investigation (Duffy et al 1986) employed a "down-the-hole" microwave unit to dry the soil to near zero moisture conditions. This allowed the probable maximum collapse that would occur to be measured. The field results compared well with the results of laboratory collapse potential tests of hand-carved block samples.

While studying the behavior of lime-treated collapsible soils, Alfi (1984) found results contradictory to Badu-Tweneboah. He noted that the collapse compression of laboratory samples upon free access to water and the

initial compression before saturation both increased with increasing initial moisture contents. Alfi obtained these results by modifying the moisture content of undisturbed samples prior to performing continuous load laboratory pseudo-consolidation tests on them. He observed that the level of applied stress and the loading and wetting sequence affected the volume change characteristics such that collapse was near instantaneous and the rate of collapse increased with increasingly applied stress.

Compacted Clays

It has been shown that compacted soils may collapse when wetted (Holtz 1948; Barden and Sides 1969). Fine grained soils of low density and low initial moisture content are likely to collapse upon wetting. This is also true for compacted soils with the same initial conditions. Clays with a moisture content greater than the optimum level for compaction show no collapse behavior, while those compacted to the same dry density but at moisture contents dry of optimum do. In addition, a dry compacted soil with a high clay content will swell under low applied pressures and collapse under high pressures upon wetting.

Collapse of compacted soils has been found to be the cause of failure in highway embankments, pavements, earth dams, berms, and backfills.

Collapse Mechanism

The loose packing arrangement of collapsible soil is maintained by interparticle bonds. These bonds are: (1) frictional forces resistant to sliding; (2) surface tension forces due to capillary action; and (3) cementation by clay bonding or other chemical weathering products.

The collapse mechanism involves shear failure in the matrix material between the larger particles. Collapse occurs when the shear force exceeds the resistive bond forces. Frictional resistance is lowered slightly by the application of water, which lubricates the contact points. Surface tension forces brought about by capillary rise are greatly reduced, and for the most part completely destroyed, by moisture uptake. The extent of clay bond weakening depends on the strength of the attractive forces of the clay and how water is adsorbed. The shear strength of the clay is reduced as water is added to dry material. As water is adsorbed, chemical cements may go into solution and the clays may enter into suspension.

As bonds fail, soil grains roll and slide into a more compact arrangement. Regardless of the type of bond, all collapsible soils are weakened by the addition of water.

Effective Stress

The principal of effective stress, as presented by Terzaghi (1954), assumes that the total applied stress σ in a fully saturated soil system equals the effective stress $\bar{\sigma}$ of the grain-to-grain contact pressure and the pore water pressure U so that:

$$\sigma = \bar{\sigma} + U \quad \text{or} \quad \bar{\sigma} = \sigma - U \qquad \text{(Eq. 3.1)}$$

Bishop and Blight (1963) extended the principal to partially saturated soils with the inclusion of a term for pore air pressure Ua and a soil parameter **X**. Pore water pressure becomes Uw and **X** represents the proportion of the pore pressure difference (Ua - Uw) that contributes to the effective stress. The equation for partially saturated soils is then expressed as:

$$\sigma = (\sigma - Ua) + \mathbf{X}(Ua - Uw) \qquad \text{(Eq. 3.2)}$$

The term (Ua - Uw) represents stability at the inter-granular contact points due to the suction forces of: (1) capillary action and (2) osmotic pressures which develop as a result of dissolved salts (Blight 1980). The term ($\bar{\sigma}$ - Ua) represents stability due to normal forces at the contact points.

Changes in effective stress may occur without loss of pore fluid in an unsaturated soil. Hence, the effective stress is increased with the application of a surcharge load. Considerable strengths may be developed in unsaturated soils as a result of loading. Blight (1966)

recorded unconfined compressive strengths of 2000 tsf in some desiccated clays.

The collapse phenomenon may appear to contradict the principles of effective stress. A decrease of the effective stress, due to an increase in pore water pressure, should cause an increase in soil volume rather than a decrease. Indeed, swelling of the very fine colloidal particles which constitute a portion of the bonding material does come about. However, once the soil suction term (Ua - Uw) has been reduced below a critical value, the bonds give way and the larger granular particles fall into the voids.

Collapse-Related Criteria

Criteria for determining the presence of collapsible soil and the degree of susceptibility to collapse are based on field observations and laboratory tests of core samples.

The identification methods are primarily qualitative in nature and they tend to relate collapse susceptibility to identifiable criteria for localized soil problems. The initial or insitu soil parameters typically sought are the dry density γ_d, porosity n_o, void ratio e_o, degree of saturation S_o, moisture content w_o, and Atterberg limits. Values for these parameters are not consistent in collapsible soil, and there is also some redundancy among

the parameters themselves, e.g., $n_o = f(e_o)$. Many of the more commonly used methods have been described by Badu-Tweneboah (1984), Sabbagh (1982), and Sultan (1971).

The double oedometer consolidation test, as proposed by Jennings and Knight (1975), is widely used to predict the collapse settlement of a soil for design purposes. The test provides quantitative information as to the magnitude of the collapse which may be compared to limits developed by Jennings and Knight to obtain a qualitative determination of the possibilities for collapse. The general procedure is as follows:

Two consolidation tests are simultaneously performed on samples of the same soil. One sample is loaded at its natural moisture content while the other is loaded after having been submerged. The difference in strain (void ratios) between the two curves represents the soil collapse.

Jennings and Knight also developed a collapse criterion called the Collapse Potential, C_p, as a quick estimate of the severity of collapse one might expect. A load of 2 tsf is applied to an undisturbed sample of soil which has been fitted into a consolidometer. The specimen is then inundated with water and the resulting displacement is measured as the percent collapse due to saturation. The value of C_p is then computed as:

$$C_p = \frac{\Delta e_c}{1 + e_o} = \frac{\Delta H_c}{H_o} \qquad \text{(Eq. 3.3)}$$

where

Δe_c = Change in void ratio upon saturation

e_o = Initial void ratio

ΔHc = Change in sample height upon saturation

ΔH_o = Initial sample height

A typical displacement curve (Equation 3.3) is presented in Figure 6. Collapse Potential Indices which Jennings and Knight proposed are listed in Table 1.

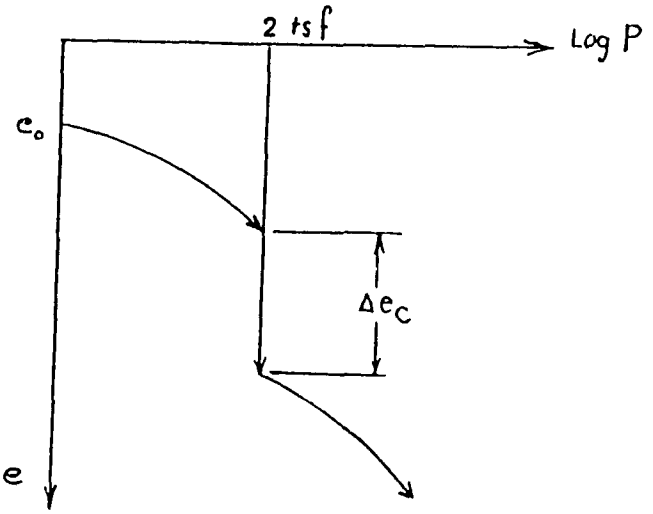


Figure 6. Typical Collapse Potential Test Results (After Jennings and Knight 1975)

Table 1. Collapse Potential Values

C_p	Severity of Problem
6 - 1%	No problem
1 - 5%	Moderate trouble
5 - 10%	Trouble
10 - 20%	Severe trouble
> 20%	Very severe trouble

The "pseudo-consolidation test" (Nowatzki 1980) was used in the present investigation as a means to quickly estimate the collapse settlement for a large number of soils sampled. The test is based on the Collapse Potential parameter C_p, which is obtained in a manner similar to the test described by Jennings and Knight. The procedures used for the investigation are presented in Chapter 7 on laboratory investigations, while the specific procedures, as proposed by Nowatzki, are in the Appendix.

The indices used to evaluate the collapse potential are those proposed by Sabbagh (1982). By calculating the criteria for a number of collapse identification methods and taking an average of the results for each sample, Sabbagh was able to classify each one as high (HC), medium (MC), or low to non-collapsing (NC). Each category possessed distinct characteristics. For instance, the non-collapsing soils (as a group) were of low porosity,

high density, low liquid limit, high plastic limit, zero plasticity, and low collapse values. There were obvious exceptions among the data which Sabbagh attributed to sample disturbance and inconsistencies in the test procedures.

CHAPTER 4

STUDY AREA

The collapsing soil problem has been extensively documented and studied in the Tucson area. Platt (1959) and Crossley (1969) mapped out zones of structural failure according to the severity of cracking they found in walls and foundations. They correlated these failure zones with groundwater withdrawal and collapsing soils. Crossley determined that structural cracking and collapse susceptibility were related to four alluvium terrace deposits within the Tucson Basin. These terraces were formed during cyclical periods of heavy erosion and deposition. Buildings with the most severe cracks were rooted in the more recently formed Cemetery Terrace. A map which illustrates the fracture zones identified by Crossley is presented in Figure 7, followed by a fracture pattern superimposed on the terraces in Figure 8. Older terrace formations are believed to have undergone preconsolidation before erosion exposed the soils. Cracking was found to be less severe for buildings located on the older terraces.

Abdullatif (1969) identified the Cemetery terrace soils as silts and silty sands of low density and high compressive strength at low moisture contents.

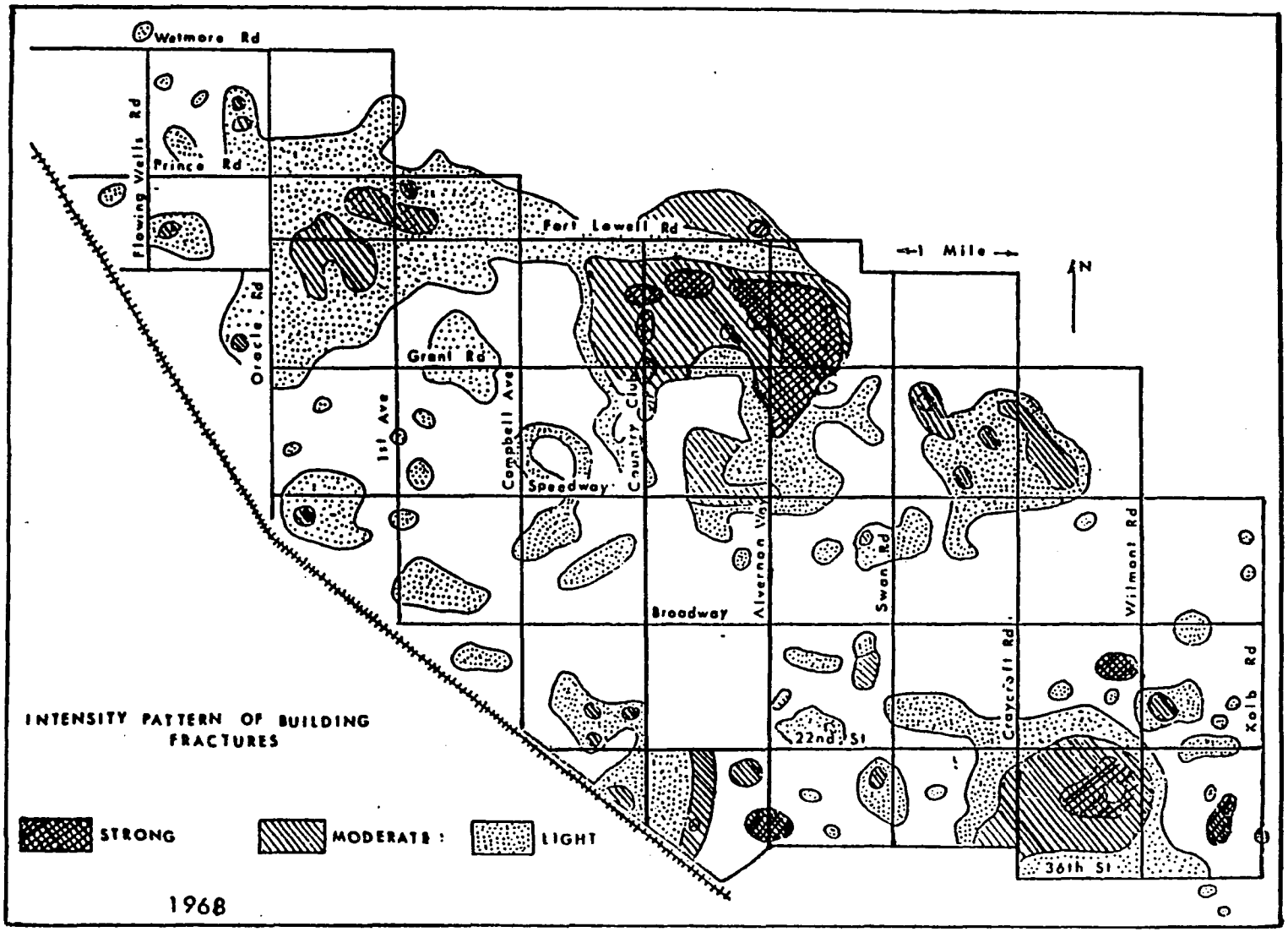


Figure 7. Map Showing Areas of Foundation Failures in Tucson (After Crossley 1969)

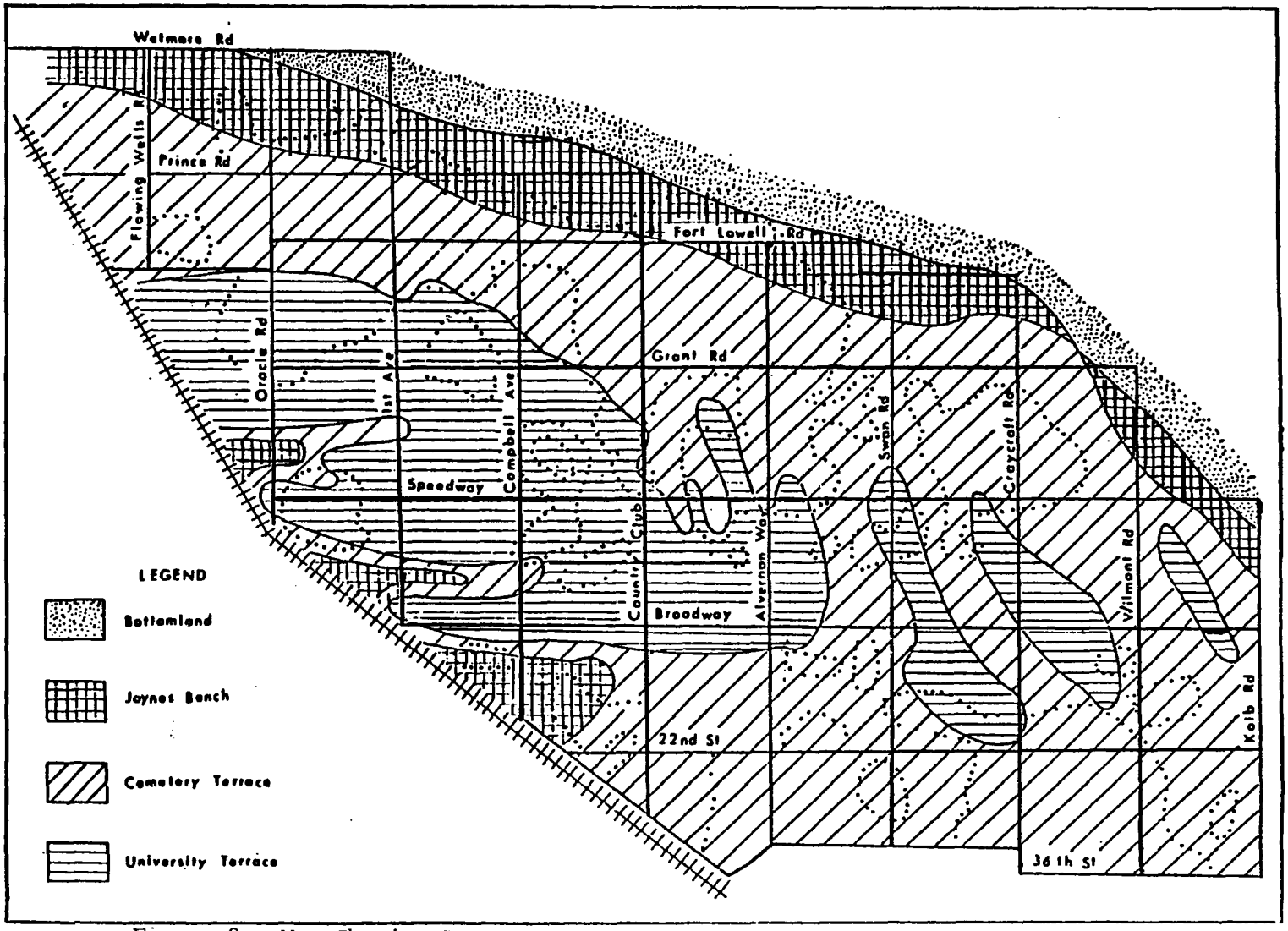


Figure 8. Map Showing Fracture Pattern Superimposed on Terraces in Tucson (After Crossley 1969)

Later studies by Sabbagh (1982) and Alfi (1984) correlate collapse susceptibility with clay mineralogy and the physico-chemical aspects of soil behavior. Alfi concluded that dispersion of the clay buttresses and reduction of capillary tension weaken the support structure of collapsible soil (Figure 5). He proposed a collapse identification paramter (A) based on the initial void ratio, e_o, initial moisture content, w_o, and the dry density, γ_d. Alfi found that lime treatment could significantly modify collapse behavior.

Sabbagh (1982) evaluated collapse criteria based on seven previous theoretical and experimental identification procedures. He identified the soil under investigation according to these procedures and found two dominating parameters: that of dry density and insitu moisture content. By relating the dry density γ_d and the natural degree of saturation S_o to compression due to saturation C_p, Sabbagh was able to classify the samples as highly, moderately, and low or non-collapsing.

Later, Ali (1987) undertook a statistical analysis of 992 soil samples from 411 locations throughout Tucson. Most of the data used for the analysis were from pseudo-consolidation tests performed by local geotechnical engineering consultants. From the insitu dry density, γ_d, insitu moisture content w_o, and percent consolidation upon loading and saturation, C_p, other collapse-related

parameters were identified. Ali evaluated the variability of specific collapse-related criteria and determined which parameters had the most effect on collapse prediction methods. He found that there was much variability among all the collapse-related soil parameters and that these dispersive tendencies increased with depth. He noted that 80% of the variation of collapse susceptibility encountered in the data was related to the dry unit weight, d, and the degree of saturation, S_o, and that these were significantly correlated with the collapse criterion, C_p.

Ali applied the geostatistical method of Ordinary Kriging to plot probabilistic contour maps of the Tucson area. He:

1. Developed contours of values of the collapse-related soil parameters and collapse criteria.
2. Developed probability contours for the occurrence of low, medium, and highly collapsing soils based on the values of the collapse-related soil parameters and collapse criteria.

These can be used to estimate the various collapse parameters for an untested site with some degree of confidence. The maps are part of the comparative analysis in this investigation.

From the literature review of sample disturbance and collapse investigations in Tucson, it can be seen that pseudo-consolidation test results are affected by the type and quality (degree of disturbance) of soil sample

retrieved from the field. There are many uncertainties among the soil parameters on which the pseudo-consolidation test and other prediction methods are based. Nor are the parameters consistent for all collapsible soil.

Cost effective field measurements of soil collapse would eliminate the need for laboratory testing or at least provide a base line from which to judge the results of laboratory tests.

CHAPTER 5

DEVELOPMENT OF TEST EQUIPMENT

The variability of the parameters which describe collapse susceptible soil and the inconsistency among identification procedures are some of the reasons to field test soils in their natural state. The insitu collapse test device was designed to perform a reliable yet inexpensive field test that could be incorporated expediently into routine geotechnical investigation field procedures.

An insitu pseudo-consolidation test was performed using a specially designed insitu collapse testing device. The depth of moisture penetration which occurred during a test was then measured with the aid of a "plug" sampler. "Undisturbed" samples were retrieved prior to an insitu test for laboratory pseudo-consolidation testing and for density and moisture content determination. The following sections give more details on each component of the equipment with a description of the test procedures in Chapters 6 and 7. The results of the investigation and subsequent statistical analysis, discussion, conclusions, and recommendations are in the remaining text.

Insitu Collapse Test Device

The test device is primarily an apparatus which applies stress to the soil at the bottom of a shallow augered hole. It is also a conduit for water to saturate the soil after a certain stress level is reached.

The test equipment consists of a rod, foot, surface plate, collar, weights, and dial gage assembly. The rod is a 5-foot (ft.) length of drill stem that weighs 21.1 pounds (lbs.). Attached to the base of the rod is a 2.375-inch diameter steel foot having a surface area of 4.43 $inch^2$. A .25-inch diameter hole drilled through the center of the foot allows water to access the soil in a controlled fashion. An adjustable collar is attached to the upper portion of the rod to support the weights. The total weight of the rod, base, and collar is 23.74 lbs., for an applied pressure of 772 psf at the base of the foot. The rod assembly is a "seating" load effective to about 7 ft. of depth for soils with a density of approximately 115 pcf. Additional stress is applied by adding the weights listed in Table 2. These weights are added to the assembled insitu collapse test device to stress the soil to levels typical for light structures. Foundation pressures for one to two-story buildings in the Tucson area generally range from 1500 psf to 2500 psf, depending upon allowable settlements. Footings are

Table 2. Weights for Insitu Collapse Test Device

Weight #	Weight (lbs.)	Additional Stress (psf)
1	9.7000	315
2	7.3128	238
3	7.2930	237
4	2.9900	97
5	2.9600	96
6	19.742	642

typically located at depths of 1 to 2 feet below ground level.

A surface plate slipped over the rod provides lateral support for the rod assembly. Rod displacement is measured with a dial gage that is magnetically attached to the rod. The gage has 1 inch of travel and reads to the nearest 0.001 inch. Water poured into the top of the rod enters the soil through the foot. A schematic diagram of the rod assembly is illustrated in Figure 9. Photos of the equipment are presented in Figures 10 and 11.

Sampling Equipment

Equipment for obtaining the laboratory samples consists of 3 brass rings and a loading system. The rings have dimensions of 2.46-inch O.D., 2.42-inch I.D., and 1-inch length. They are stacked and bound together by several tight wraps of 2.5-inch wide strapping tape. The

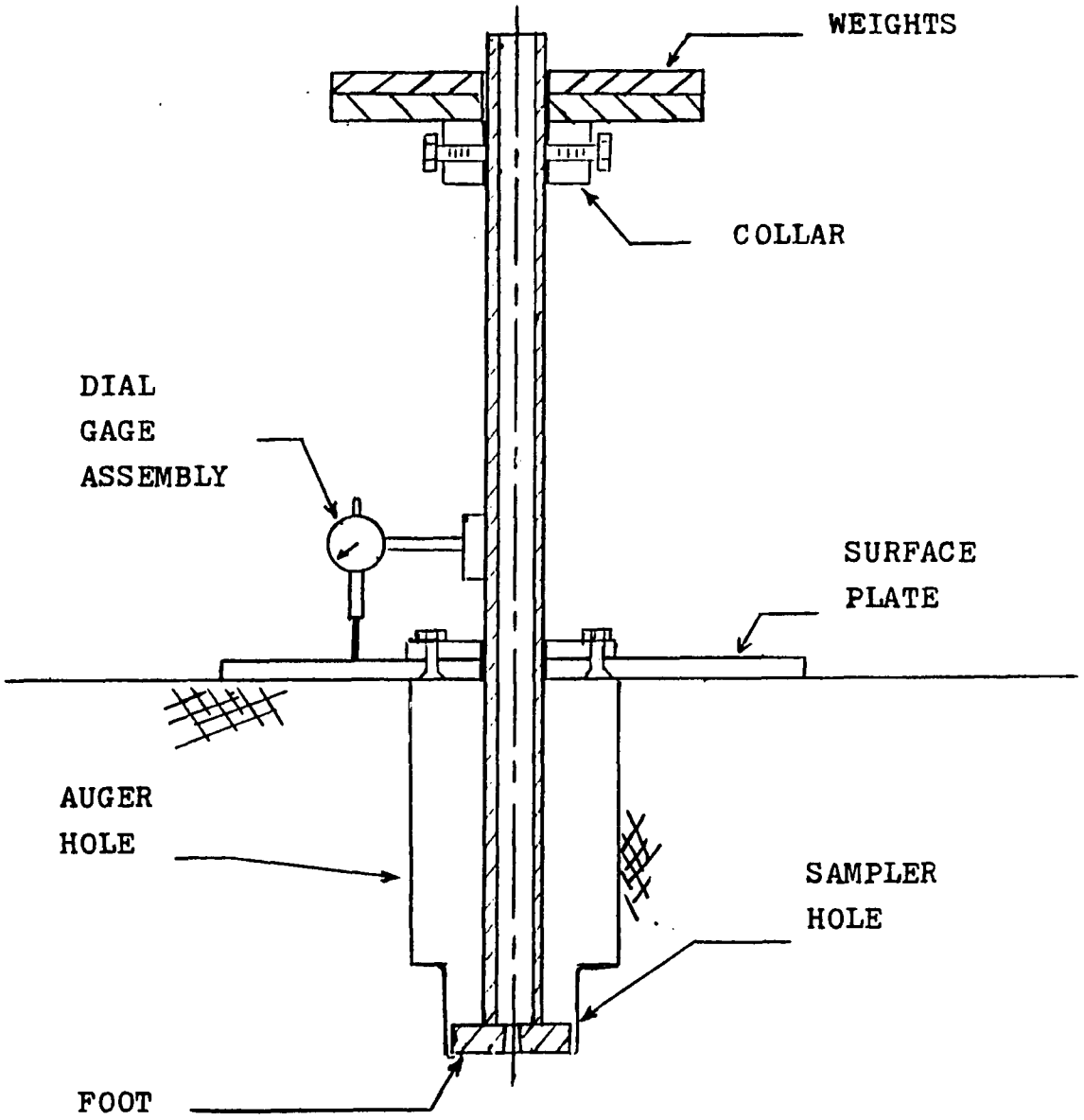


Figure 9. Schematic of the Insitu Collapse Test Device

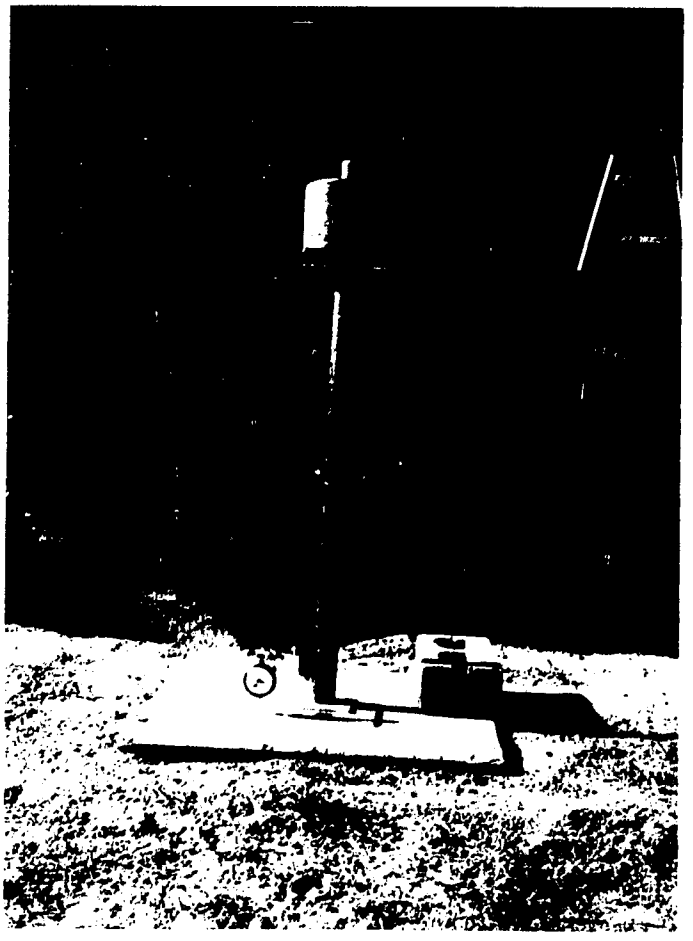


Figure 10. Assembled View of the Insitu Collapse Test Device

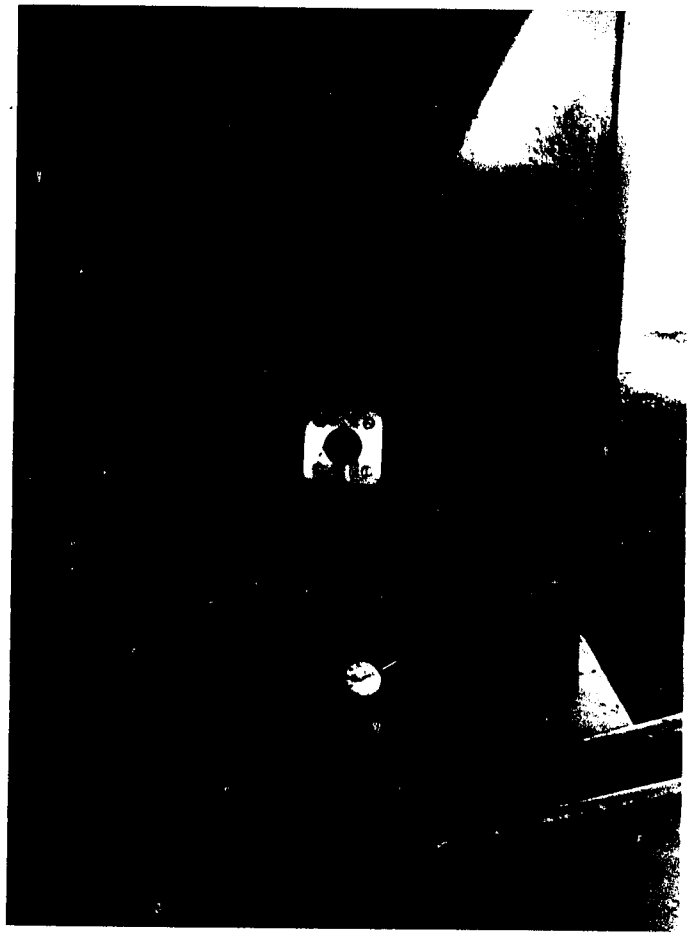


Figure 11. Disassembled Test Equipment

sampler area ratio (C_a)is 3.3%. An assembled 3-ring sampler is shown in Figure 12.

Loading is attained with a bumper-jack, base plate, and reaction load. The base plate supports the jack and provides an even load distribution to the rings. A truck serves as a reaction for forcing the sampler into the soil. Loading is applied by a 2-ton, manually operated bumper-jack. Figure 13 is a view of the sampler system at the surface prior to loading.

Moisture Penetration

The volume of wetted soil and the degree of saturation which is attained are two controlling parameters of soil collapse.

As there are no practical ways to measure the wetting front during performance of the collapse test insitu, a "plug" sampler was devised to measure the extent of wetting immediately following the test. The plug sampler is a 5-ft. section of galvanized iron pipe with an inside diameter of 1 inch and outside diameter of 1.2 inches. The penetration end of the tube is bevelled to reduce frictional resistance and compression of the soil upon sampling. A 1/2-inch wide observation slit cut along a 6-inch portion of the sampler allows for direct viewing and measurement of the wetting front after the plug sampler is removed from the hole. The plug sampler is in

Figure 12. Assembled 3-Ring Sampler

Figure 13. Sampling System Prior to Loading at the Ground Surface

the lower portion of the photograph shown in Figure 11 of the test equipment.

CHAPTER 6

FIELD INVESTIGATIONS

Characteristics of the Study Area

The study was conducted at 42 sites in the Tucson area depicted within the grid system shown in Figure 14, and one site north of Phoenix.

The climate of Tucson is arid to semi-arid with mean monthly temperatures varying from 50°F in January to 86°F in July. Summer temperatures often exceed 100°F while the winters are mild. The mean annual precipitation is about 11 inches and is greatest through the months of December, January, July, and August. The evapotranspiration rate of 10 to 13 inches per year is a function of the mean daily temperature and annual precipitation (Hendricks 1985).

Tucson is a part of the Basin and Range Province which extends through Nevada, western and southern Arizona, and portions of New Mexico, Utah, and California. Uplifted mountain blocks, which lie roughly parallel to one another, are separated by vast basins which have filled with water, rain, and windblown deposits in the form of pediments and alluvial fans. The geology of the mountains surrounding the Tucson Basin consists of foliated gniess and rocks of volcanic origin. The most

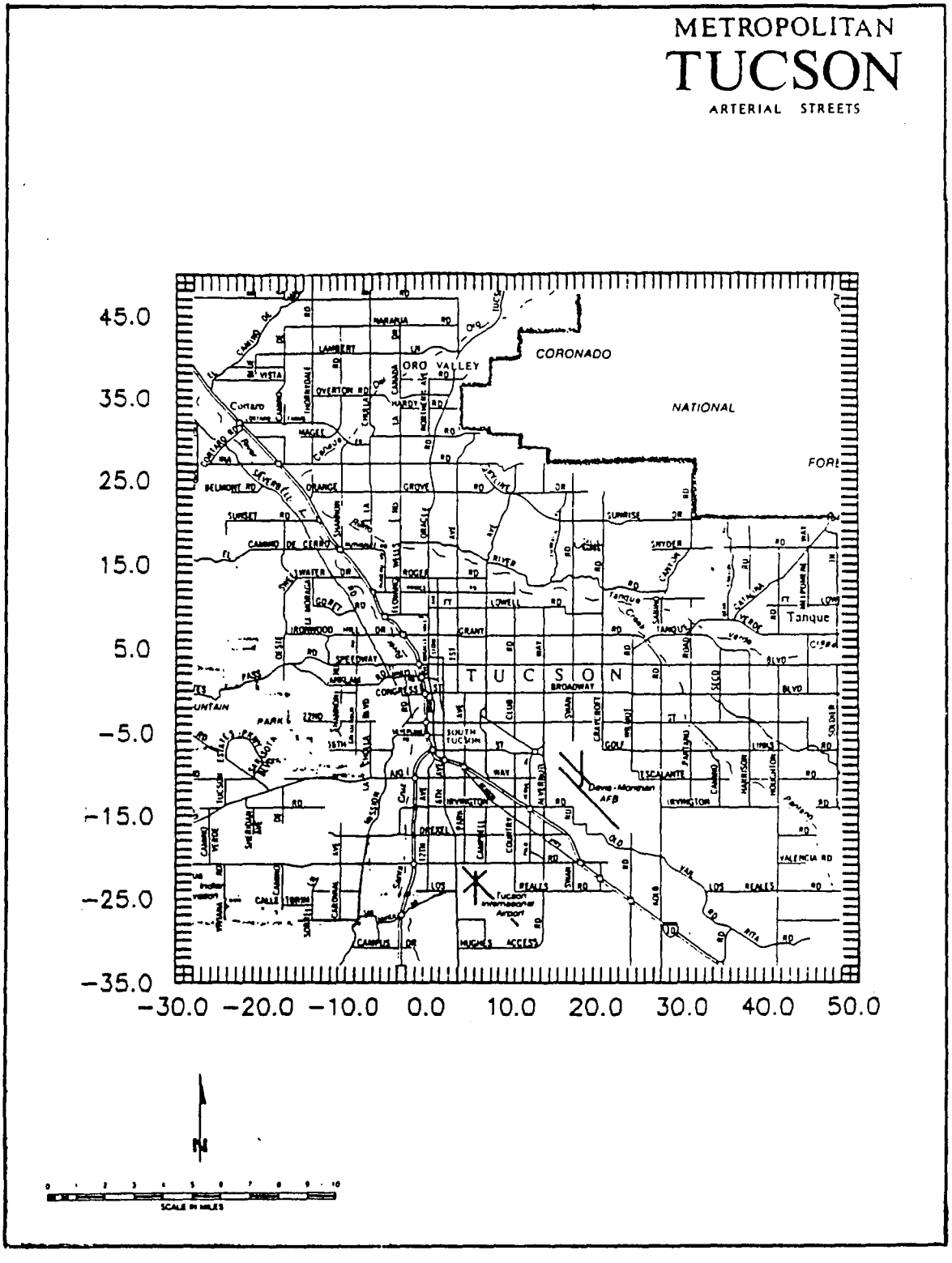


Figure 14. The region of study. (Adapted from Ali, 1987)

recent soil deposits are chiefly mineralized silts found in the Cemetery Terrace formation, which has been discussed in Chapter 4. These form the collapsible soils of the current investigation. The rolling Tucson terrain has also undergone much cut and fill by developers who have restructured the landscape to suit human needs. An extensive description of the geology and soils in the Tucson Basin was given by Crossley (1969).

A majority of the sites for the present investigation appeared to be on soils previously undisturbed by man at depths below the first 6 inches. Three of the sites were on fill material. Caliche was encountered in the soil profile at many of the sites. It was highly resistant to sampling and testing. Such calcareous layers may be several inches thick to many feet thick and are found throughout the Tucson Basin at various depths.

Field Investigations

The field investigation consisted of the following features:

1. Performing insitu collapse tests
    a. Throughout the Tucson area in company with a geotechnical engineering firm's field crew;
    b. At randomly selected sites in areas previously identified as having collapsible soils.
2. Obtaining undisturbed soil samples from all test sites.

Sampling Procedures

Much of the field work was done with the support of Western Technologies, Incorporated (WTI), a local geotechnical engineering firm. Upon completing a boring for their soils' investigation, WTI's field crew would auger a 10-inch diameter hole to a depth of 1 foot for the performing of an insitu collapse test. The test hole was augered at a distance of 1 to 2 feet from the initial boring. The procedure provided a hole with minimal time loss for the drill crew and it enabled the results obtained from laboratory tests conducted by the soils investigation firm to be correlated with those obtained from field and laboratory tests performed by the author. In many instances, undisturbed soil samples at the 1-ft. depth level were not obtained from the initial boring, as sampling at that depth was not always a part of the investigation, nor was it always possible to obtain drive samples in the conventional manner. There were 19 field tests performed in conjunction with WTI. The remaining 24 tests were performed solely by the author.

Undisturbed drive samples were obtained by the drill crew using a Dames and Moore Ring Sampler. Undisturbed continuous load samples were obtained by the author using the sampling device described previously.

Drive sampling was done by WTI in accordance with ASTM-D 3550 standards. The drive sampler consisted of a

3-inch O.D. barrel lined with 2.42-inch I.D. brass rings for an area ratio, C_a, of 53.7%. The sampler was driven into the soil with the aid of a manually operated cathead system. Upon removal from the borehole, the rings containing the samples were extruded from the barrel and sealed in watertight containers.

Continuous load sampling was done by the author similar to methods described in ASTM-D 1587 for thin-walled samplers. In sampling, the bottom of the test hole was leveled and cleared of loose material. The sampler was placed in the hole and the base plate set upon it. A truck was positioned over the hole so that the jack, which was set on the base plate, could be braced against the bumper of the truck. Loads applied to the sampler forced the rings into the soil. Loading was stopped when the top ring was nearly flush with the bottom of the test hole. Forcing the ring any farther would unduly compress the soil.

The loading system was removed and the sampler extracted by loosening the upper 2 inches of soil around the rings with a digging bar or long screw driver. Care was taken not to strike the sampler lest the soil structure be altered. By slightly rotating the sampler, the lower contact soil was sheared so that the sampler could be removed and sealed in a watertight bag.

The samples were transported to the laboratory in a box lined with packing material. The box was placed on the seat of the truck to reduce the chance of vibratory disturbance even further.

Insitu Collapse Test Procedures

Recall that the test hole was either augered by the drill crew from WTI or hand dug by the author with a shovel and digging bar. The depression which formed in the bottom of the test hole, as a result of sampling, was subsequently used for the insitu collapse test. The bottom of the hole was smoothed, loose soil removed by hand, and the test device assembled.

The support rod was lowered into the hole until the foot rested lightly on the bottom. The surface plate was slipped over the rod and placed on top of the soil at ground surface. The collar was then slipped over the rod and secured about 6 inches from the top of the rod. The assembly was plumbed and a weight was set on the surface plate to prevent lateral movement.

To load the system, weights were slipped over the support rod and set on the collar. After a set of weights had been added, the dial gage was allowed a maximum of 5 minutes to stabilize and the displacement of the support rod recorded. The procedure was repeated until the expected footing pressure was reached. In

general, there were two loading sequences for the investigation. These are shown in Table 3.

Table 3. Loading Sequence

From (psf)	To (psf)	Weight(s) Already On	Weight(s) to be Added
Sequence 1:			
772	1010	rod assembly	2
1010	1652	rod, 2	6
Sequence 2:			
772	1010	rod assembly	2
1010	2064	rod, 2	1,4,6

After the last displacement was recorded, 2 ounces (oz.) of water was added at the top of the hollow drill rod to induce collapse. No more than 5 minutes was allowed for gage movement to cease once collapse began. The final reading was recorded and the assembly checked for plumb.

At the completion of the test, the device was quickly disassembled and the area around the foot of the support rod was visually inspected for unusual soil displacement and radial moisture migration. Any unusual or unexpected observations such as soil which appeared to be pushed up the sides of the foot, or radial moisture migration which

did not encompass the diameter of the foot, were duly noted.

Immediately following this inspection, the extents of the saturation and wetting fronts were measured with the aid of the plug sampler dsecribed previously in Chapter 5. The plug sampler was centered in the hole and driven to a depth of 6 inches, or to a depth of 1 inch beyond any firm resistance encountered. The sampler was rotated to shear the soil at the bottom of the tube and then removed. The length of the saturation and wetting fronts were measured to the nearest 1/16 inch through an observation slit in the side of the tube. Measurements were taken from the upper portion of the soil plug to the transition boundaries.

A second measure of the saturated zone was made by probing the bottom of the sampling hole with a 1/2-inch diameter plastic rod. The rod was gently forced into the soil at various points within the contact area of the foot until a noticeable increase in resistance was felt. The probe depth was taken as a measure of confidence for the plug sampler results and the measurements were often in close agreement, particularly for soils with a high clay content. For a number of soils, however, there was no discernible saturated zone due to the rapid infiltration of moisture away from the point of moisture application.

For these soils an average of the probe depth and wetted front was taken as the depth of the saturated zone.

The saturation zone was described to be that portion of the wetted front which glistened to the eye or otherwise appeared to contain more moisture than the remaining wetted zone.

The wetted front was described to be that portion of the soil profile from the contact surface with the foot to a vertical depth where the moisture had visibly ceased to infiltrate at the time of measurement.

Each field test including sampling, equipment assembly, testing, disassembly, and moisture penetration determination required approximately 45 minutes to perform.

CHAPTER 7

LABORATORY INVESTIGATIONS

The pseudo-consolidation test was used to determine the collapse characteristics of the soil retrieved from the field. Drive samples were tested in the Western Technologies' laboratory according to its standard procedure. Continuous load samples were tested by the author in the following manner:

1. The ring samples were separated and the most intact specimen (preferably from the bottom or middle of the ring) was trimmed, measured, and weighed.

2. The ring specimen was placed between two porous stones in the empty reservoir of a consolidometer and a dial gage sensitive to 0.0001 inches was attached to measure the vertical deformation. The Model C-220 levermatic consolidometer (Soil Test, Inc., Chicago, Illinois) was employed for the test.

3. The dial gage was zeroed and a 750 psf seating load was applied to match the initial load applied by the insitu collapse test device. Dial readings were recorded every 5 minutes for the first 15 minutes and then every 15 minutes thereafter until the difference between two consecutive hourly readings was less than 0.001 inches.

4. The applied stress was then increased by the same order of magnitude as in the field and the deformation readings were recorded as described above. The same procedure was followed until the vertical stress on the sample was approximately equal to the final stress applied in the field.

5. The sample was then inundated with water and displacement readings were again taken as previously described. After the final reading, the sample was weighed, oven dried, and reweighed to determine the final moisture content and dry density. Specific procedures for the pseudo-consolidation test as described by Nowatzki (1980) are presented in the Appendix.

CHAPTER 8

FIELD AND LABORATORY RESULTS

Sampling

Of the 43 soils that were field and laboratory tested, drive sampling was successful at 3 of 6 locations, while continuous load sampling was successful for all but 2 locations. Several attempts at sampling were needed at 6 sites, including the sites where drive samples were unobtainable. The soil structure of one continuous load sample crumbled in transit to the laboratory.

Continuous load sampling was most difficult in caliche soils and well graded, fine grained soils. Gravel and other coarse grained material impeded the penetration of the sampler and often deformed the edge of the leading brass ring of the 3-ring sampler.

Field and consolidation testing of soils which contained coarse grained particles also proved difficult. Gravel sized particles were too large for the scale of testing that was undertaken and they inhibited compression.

Collapse Testing Concepts

The collapse settlement of partially saturated soils was measured using the pseudo-consolidation test described

in Chapters 6 and 7. The test measures the initial ("dry") compression, C_i, upon loading and the collapse compression, C_p, upon soaking so that the total amount of collapse, C_t, is the sum of the two components; i.e.,

$$C_t = C_i + C_p \qquad \text{(Eq. 8.1)}$$

where

C_i = Percent compression of a partially saturated soil under a specified load. The deformation is the change in soil length to the initial length.

C_p = Percent compression of a "saturated" soil under the same specified load as C_i. The deformation is a percent change of the saturated length.

Insitu Collapse Test

The insitu collapse test device measures the amount of compression which occurs within a prescribed volume of soil when the soil is subjected to specific pressures over a controlled area of wetting. The total percent compression, C_t, encountered in the field ranged from 0.06% to 14.39%. Initial compression, C_i, varied from 0.0% to 4.5%, and the percent of collapse, C_p, upon wetting varied from 0.0% to 11%.

The initial compression, C_i, was calculated as a percent strain relative to the "length" of soil which was stressed in the field. For the field test,

$$C_i = \frac{\Delta H_1}{B} \qquad \text{(Eq. 8.2)}$$

where

ΔH_1 = displacement of the rod on the "dry" soil.

B = diameter of the foot of the collapse test device.

The influence depth of the applied load was assumed to be one diameter of the test foot for reasons explained subsequently.

The collapse compression which occured when water was introduced was calculated as a percent of the length of wetted soil; i.e.,

$$C_p = \frac{\Delta H_2}{d_{sat}} \qquad \text{(Eq. 8.3)}$$

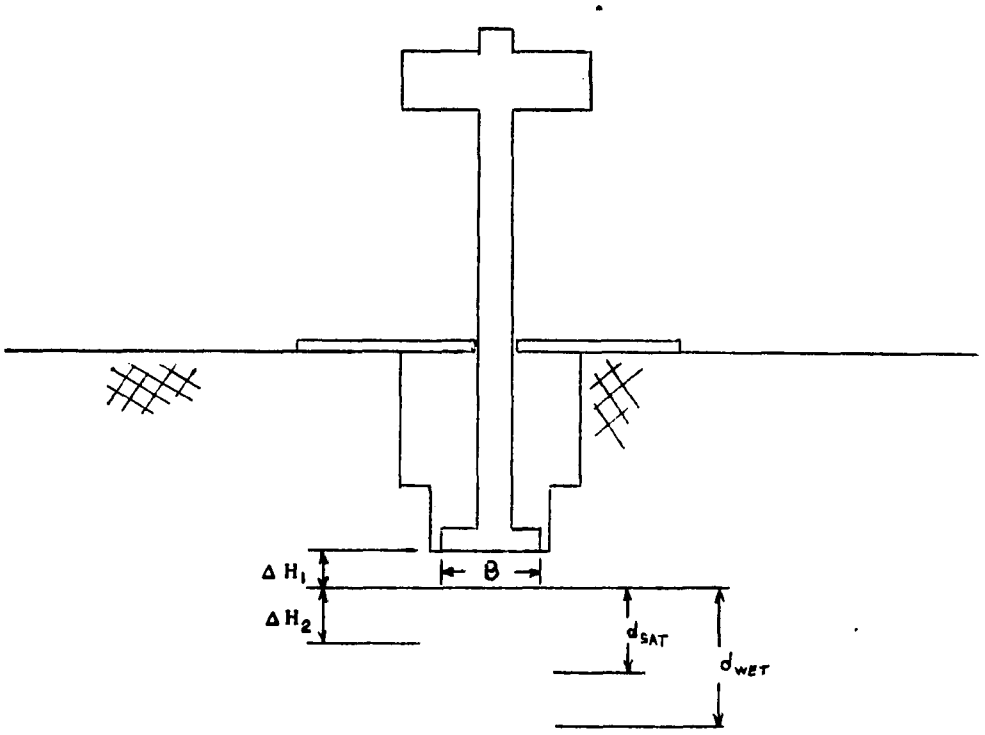
where

ΔH_2 = displacement of the rod after wetting has occurred.

d_{sat} = depth of maximum saturation.

It should be noted that d_{sat} does not necessarily coincide with d_{wet}. The distinction between the two was discussed in Chapter 6. Features of the insitu collapse test are illustrated in Figure 15. The results of all the tests are presented in Table 4.

Several problems encountered in the field were related to the depth of wetting. The degree of saturation and collapse is greatest in soil nearest the base of the foot where stress and water are applied to the soil. Since the influence of applied load and degree of wetting decrease with depth, the amount of collapse also decreases. Collapse continues with depth until the



d_{wet} = depth of wetted front.

d_{sat} = depth of maximum saturation.

ΔH_1 = displacement of the rod on the "dry" soil.

ΔH_2 = displacement of the rod after wetting of the supporting soil.

B = diameter of the rod.

Figure 15. Instiu Collapse Test

Table 4. Results of Field and Laboratory Collapse Tests as Measured in Percent at Loads of 1.5 ksf or (2.0 ksf)

Site No.	FIELD COLLAPSE TEST			PSEUDO - CONSOLIDATION TEST									
				UA soils lab			initial moisture content	dry unit wt.γ	WTI soils lab			initial moisture content	dry unit wt.γ
	C_i	C_p	C_t	C_i	C_p	C_t	w_i	(pcf)	C_i	C_p	C_t	w_i	(pcf)
1	0.17	0.40	0.57	1.65	4.96	6.61	1.61						
2	4.46	0.40	4.86	8.55	4.30	12.85	6.38						
3	0.25	0.00	0.25	3.53	0.15	3.68	9.00						
4	3.71	0.12	3.83	2.15	0.76	2.91	9.24						
5	0.21	4.20	4.41	1.10	3.91	5.01	1.60						
6	0.08	1.60	1.68	1.81	5.69	7.50	3.40		1.50	5.80	7.30	3.8	91
7	4.00	2.00	6.00	2.16	2.30	4.46	14.42		1.50	2.80	4.30	15.4	86
8	0.06	0.00	0.06	8.97	2.90	11.87	8.20						
9	0.11	0.00	0.11	2.34	2.28	4.62	7.89						
10	0.72	7.44	8.16	3.64	7.05	10.69	1.88						
11	0.00	3.66	3.66	1.00	1.00	2.00	3.20						
12	0.00	0.80	0.80	5.19	1.89	7.08	9.92						
13	0.59	2.00	2.59	9.61	2.34	11.95	9.61						
14	0.42	1.33	1.75	2.94	2.66	5.60	1.75						
15	0.67	1.60	2.27	2.14	3.95	6.09	2.26						
16	0.17	0.60	0.77	3.46	6.05	9.51	4.78						
17	0.67	0.10	0.77	6.95	2.17	9.12	13.01						
18	0.34	1.40	1.74	6.57	8.90	15.47	11.70						
19	0.21	0.70	0.91	1.99	1.30	3.29	3.13		2.02	1.36	3.38	3.4	
20	0.76	0.80	1.56	1.20	5.20	6.40	5.78						
21	0.00	1.60	1.60	0.65	3.01	3.66	4.23						
22	0.55	0.00	0.55	2.10	2.77	4.87	5.37						

Table 4. (Con't)

Site No.	FIELD COLLAPSE TEST			PSEUDO - CONSOLIDATION TEST									
				UA soils lab			initial moisture content	dry unit wt. γ	WTI soils lab			initial moisture content	dry unit wt. γ
	C_i	C_p	C_t	C_i	C_p	C_t	w_i	(pcf)	C_i	C_p	C_t	w_i	(pcf)
23	0.42	0.40	0.82	1.99	1.30	3.29	3.30		1.44	3.90	5.34	3.3	109
24	0.17	6.40	6.57	1.44	11.67	13.11	3.42						
25	0.55	0.80	1.35	3.47	0.57	4.04	8.22						
26	0.84	6.00	6.84	3.57	5.08	8.65	4.07						
27	0.00	2.00	2.00	1.00	1.00	2.00	6.00						
28	0.17	6.23	6.40	3.17	7.53	10.70	7.40	75.59					
29	0.29	2.40	2.69	3.99	5.49	9.48	7.76						
30	0.13	7.88	8.01	1.68	11.30	12.98	7.09	83.17					
31	4.21	8.67	12.88	2.86	10.26	13.12	7.35	79.71					
32	0.13	6.40	6.53	0.96	8.19	9.15	5.16	70.70					
33	0.13	0.00	0.13	1.58	4.97	6.55	8.90	95.94					
34	0.08	5.40	5.48	1.12	13.82	14.94	2.81	92.69					
35	0.04	9.44	9.48	5.58	7.33	12.91	5.16	82.61					
36	0.13	6.93	7.06	2.15	14.01	16.16	6.48	90.37					
37	3.71	10.68	14.39	1.89	9.11	11.00	5.31	81.49					
38	0.13	7.69	7.82	2.56	7.83	10.39	8.11	98.48					
39	0.29	11.00	11.29	1.78	10.92	12.70	4.20	78.42					
40	0.21	6.50	6.71	3.30	7.50	10.80	4.01	87.95					
41	0.29	6.40	6.69	2.21	9.39	11.60	4.77	87.36					
42	0.07	7.60	7.67	3.11	15.78	18.89	7.76	86.13					
43	0.21	1.20	1.41										

saturation and stress levels can no longer induce displacement.

The full depth of influence beneath a footing (generally considered to be 2B) will rarely become excessively wetted; hence, the magnitude of collapse is some fraction of the collapse potential determined by the submerged conditions of an oedometer test (Nowatzki 1980). The purpose of using a controlled amount of water when performing the insitu test was to restrict the wetted field to a small area within the influence of the applied load. The wetting process was regulated by the rate which 2 oz. of water introduced through a 1/4-inch opening in the foot could infiltrate the soil. The problem was in measuring the extent and degree of wetting during the collapse process, as these were not clearly established for 11 of 43 soils field tested by the methods described in Chapter 6.

A more precise method for measuring the extent and degree of saturation attained in the field needs to be developed. Time-domain reflectometry (TDR) has been used to determine the volumetric moisture content of soils to a depth of several feet (Topp and Davis 1985). The "cable tester" measures the propagation velocity of a high frequency voltage impulse down two or more parallel metal probes inserted into the soil. The wave velocity is proportional to the dialectric coefficient of soil and is

used in conjunction with the known dielectric constant of water. Abrupt changes in the soil moisture profile are noted, but the precise depth at which these changes occur is not yet quantifiable. The application of TDR may prove to be useful for insitu collapse testing.

Pseudo-Consolidation Test

The laboratory studies consisted of pseudo-consolidation tests and moisture content determination. As with the field tests, laboratory consolidation tests were used to determine both components of total compression C_t expressed by equation 8.1. For the laboratory test,

$$C_i = \frac{\Delta H_1}{T} \qquad \text{(Eq. 8.4)}$$

where

C_i = percent compression (strain) of a partially saturated soil under a specified load.

ΔH_1 = change in sample height

T = specimen thickness

and

$$C_p = \frac{\Delta H_2}{T} \qquad \text{(Eq. 8.5)}$$

where

C_p = additional deformation of the sample under the same load upon inundation with water.

ΔH_2 = additional change in sample height

Loads of approximately 1 tsf normal stress were applied, which is near the stress level of most light structures in the Tucson area. The loads were applied in a manner similar to the field test since homogeneity between the test procedures was needed for statistical purposes.

Total percents of collapse of up to 16.7% were measured in 40 laboratory samples tested. Initial compression varied from a low of 0.7% to a high of 9.6%, and the percent collapse upon wetting varied from 0.2% to 15.8%. The collapse parameters were determined from undisturbed samples retrieved at shallow depths (1 to 2 ft.) from all areas of Tucson and from sites where collapse susceptible soils were known to exist. Results of the pseudo-consolidation tests are presented in Table 4.

Insitu Moisture Content and Sample Density

The natural moisture content of near surface soils in the Tucson Basin is typically less than 10% except during the rainy seasons except during the rainy seasons from mid-December through January and the "monsoon" season during July and August. The water table is very deep and soil moisture is controlled by precipitation and evapo-transpiration. The active depth over which seasonal moisture variations occur in Tucson is less than 6 ft.

CHAPTER 9

ANALYTICAL INVESTIGATIONS

A statistical analysis was undertaken to test the hypothesis that pseudo-consolidation testing in the laboratory produces a collapse potential, C_p, greater than what occurs in the field. Further analyses was undertaken to evaluate the collapse criterion used by Sabbagh (1982) as it relates to field testing results.

A geostatistical comparison was made between the results of this research and those of previous studies to correlate structural damage to buildings with the collapse potential of the underlying soil.

Finally, the possible relationship between the natural moisture content and collapse was investigated.

Statistical Analysis

Statistics employs information obtained by sampling or experimentation in order to make inferences about the population from which the information came.

Statistical tools used for this study are the mean, standard deviation, probability distribution of a continuous random variable, coefficient of variation, the paired data test, and the chi-square test.

The mean and standard deviation are descriptive measures of the population and sample data. They are used to create an image of the frequency distribution for a set of measurements. The mean ($\bar{y}$) is a measure of central tendency computed as an arithmetic average. The standard deviation (s) is a measure of dispersion from the mean, computed as the square root of the variance. The variance is an average of the squared deviations about the mean. Expressions for the mean and standard deviation are as follows:

$$\bar{y} = \frac{1}{n} \sum_{i=1}^{n} y_i \qquad \text{(Eq. 9.1)}$$

where

$\bar{y}$ = sample mean

y_i = random variable

n = total number of measurements

$$s = \left(\frac{1}{n-1} \sum_{i=1}^{n} (y_i - \bar{y})^2 \right)^{1/2} \qquad \text{(Eq. 9.2)}$$

where

s = sample standard deviation

y_i = random variable

$\bar{y}$ = sample mean

n = total number of measurements

A relative measure of dispersion is the coefficient of variation (cov), which is a percent ratio of the

standard deviation to the mean. The cov is a measure of uncertainty for the random variable; the higher the percent, the greater the variability of the random variable and the less confidence can be placed in the data. The cov is calculated as:

$$cov = \frac{s}{\bar{y}} \times 100 \qquad \text{(Eq. 9.3)}$$

Probability is a way to describe how likely an event is to occur. It may be expressed as the ratio of actual events to possible outcomes or it may be interpreted as the relative frequency with which an event may happen. Since all possible outcomes for many experiments cannot be known, the relative frequency interpretation is more useful.

Normal Distribution

The normal distribution curve is a good approximation of the relative frequency distribution for a large number of n measurements. According to the Central Limit Theorem, the sample distribution curve will be approximately that of a normal distribution (bell-shaped) regardless of the shape of the population distribution curve. The normal distribution curve has the form:

$$f(y) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[\frac{-(y-\mu)}{2\sigma^2}\right] \qquad \text{(Eq. 9.4)}$$

$$\sigma > 0$$
$$-\infty < y < \infty$$

where

μ = population mean

σ = population standard deviation

y_i = random variable

The sample relative frequency histogram will have a mean μ and standard deviation σ/n. This is typical for samples of size $n \geq 30$; the normal approximation becomes more precise as n increases (Ott 1988).

The probabilities associated with a normal distribution are expressed as the area beneath the curve with bounds of $\pm z\sigma$ standard deviations from the mean where Z = the number of standard deviations that a variable y lies from the mean. The value of Z is found using the formula

$$Z = \frac{y_i - \mu}{\sigma} \qquad \text{(Eq. 9.5)}$$

The probability that a random variable lies within $\pm 1\sigma$ of the mean is .68, for $\pm 2\sigma$ is .95, for $\pm 3\sigma$ is .99.

Figure 16 shows the normal distribution curve with z bounds.

It has been shown that the normal distribution curve will fit most soil properties (Lumb 1966; Holtz and Krizek 1971). Ali (1987) has also applied the normal distribution curve to collapsible soil parameters but found the Gamma and Weibull distributions to be better descriptors for most parameters. The normal distribution

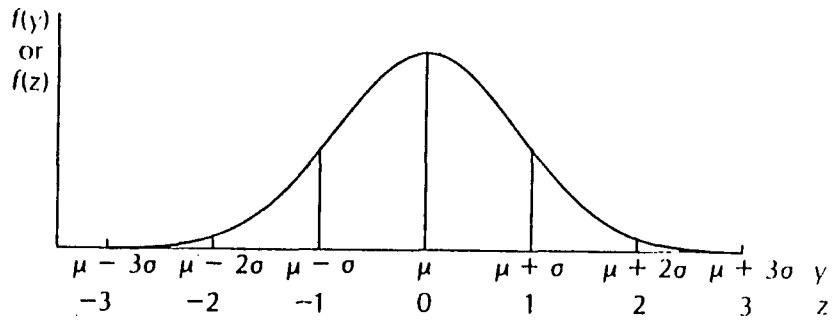


Figure 16. Relationship Between Specific Values of y and z (After Ott 1988).

of collapse data was found to be acceptable for the purposes of this investigation.

Paired Data Statistical Test

The paired data statistical test is useful when comparing two sets of dependent measurements with inherent differences. Data for the collapsible soil experiment are dependent in that the tests are performed on the same soil under like conditions for each test procedure. Although the insitu collapse test was designed to be homogeneous with the pseudo-consolidation test, some differences do exist. These include: sampling disturbance, variability of the volume of insitu soil which is wetted, the degree of wetness achieved, temperature, frictional resistance from grain to grain, and grain to metal contact, and others.

The range of collapse potential (C_p) measurements for the two test procedures vary from 0% to 16%. These variations are due to differences in structure, composition, and moisture content of the soils tested. By pairing the experimental units, i.e., by testing the differences in collapse measurements statistically, the disparity between the two procedures is evaluated independently of the soils tested. Thus, a paired-data statistical test is useful in that it eliminates the extraneous variability of the experimental units.

A paired-data analysis makes use of the difference in dependent sample measurements to test one of two contradictory hypotheses. The test validates a research hypothesis H_1 by rejecting the null hypothesis H_o. The weight of evidence for rejecting H_o is determined by a test statistic, Z, which is calculated from the sample data in the same manner discussed previously and as given by equation 9.5. The smallest level of significance (p-value) for rejecting H_o is the area which lies beyond z standard deviations from the mean of a normal distribution curve of the sample data. The p-value is the probability of observing a value of the mean difference μ_d equal to the predicted value $\mu_d = 0$, assuming that H_o is true. Ott (1988) states that the smaller the value of p, the heavier the weight of evidence for rejecting H_o.

The assumptions in using a paired-data test are: (1) The distribution of the sample differences approximates a normal distribution curve; and (2) the samples are obtained in a random manner.

The normal distribution assumption for this investigation is verified by the Central Limit Theorem, as the sample size is greater than 30. However, the random sample assumption is violated. For the data to be truly random, all soils in the Tucson area would have an equal chance of being tested. Testing was conducted on properties for which Western Technologies, Inc., was contracted. The author also tested soils on vacant lots, public property, and other sites where permission was granted. Most testing was purposely restricted to areas known or suspected to contain collapsible soils.

The paired data analysis for the experiment required the identification of the following statistical parameters:

Parameter of interest. The mean difference $_d$ between the collapse potential C_p measured by the pseudo-consolidation test and insitu collapse test for all soils in the Tucson test area.

Null hypothesis H_o. The true mean difference between the C_p for all soils in the Tucson test area measured by the pseudo-consolidation test and the insitu collapse test is zero.

Research hypothesis H_1. The true mean difference between the C_p for all soils in the Tucson test area measured by the pseudo-consolidation test and the insitu collapse test is greater than zero.

Test Statistic Z: The test statistic is the number of z standard deviations that the observed mean difference d is from the expected mean difference of $_{d0} = 0$. The test statistic for the paired data analysis is calculated in a manner similar to that described previously and given by equation 9.4. For the paired data analysis, Z is calculated as:

$$Z = \frac{\bar{d} - {}_{d0}}{{}_{d} / \sqrt{n}} \qquad \text{(Eq. 9.6)}$$

where

d = mean difference between random variables of sets 1 and 2.

$_{d0}$ = the expected mean difference under H_o

$_{d} / \sqrt{n}$ = standard error of sample data (s_d can be substituted for $_{d}$ for $n \geq 30$).

$$\bar{d} = \sum_{i=1}^{n} \frac{yi_1 - yi_2}{n} \qquad \text{(Eq. 9.7)}$$

where

yi_1 = random variable for first data set.

yi_2 = random variable for second data set.

n = total number of random variables for one set.

Student's-t Test

A second paired data analysis was performed on the data which pertained only to soils tested in areas previously identified to be collapsible. The test statistic was calculated using a Student's-t correction factor due to the relatively small sample size of n = 14. There is a higher rejection rate of the null hypothesis for sample sizes less than 30. To compensate, t-distribution curves have been developed for sample of various degrees of freedom (df = n-1). The t-distribution approaches the z standard normal distribution as n increases and becomes asymptotic at $n \geq 30$. Figure 17 illustrates the t-distribution curve.

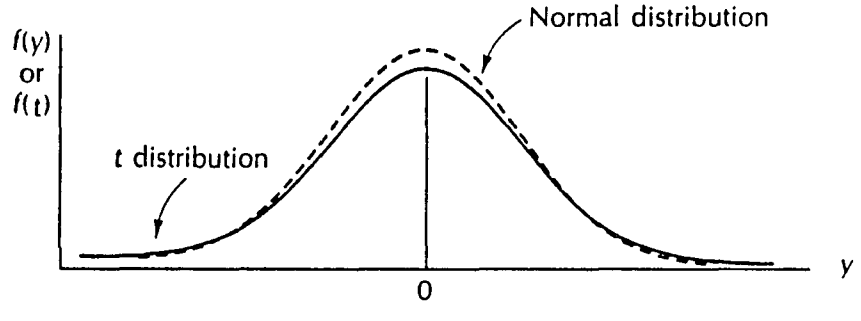


Figure 17. A t-Distribution with a Normal Distribution Superimposed

Chi-square Test

To assess the possible relationship between the distribution of field collapse results with the collapse criteria evaluated by Sabbagh, the Chi-square ^{2} "goodness of fit" test was used.

The ^{2} test determines whether the observed frequencies of occurrence are significantly close to those expected to occur under an hypothesized distribution. The test statistic ^{2} is calculated as the summation of the normalized squares of the difference between the observed and expected values for all categories, i.e.,

$$^2 = \sum_{i=1}^{n} \frac{(n_i - E_i)^2}{E_i} \qquad \text{(Eq. 9.8)}$$

where

n_i = the observed frequencies measured in the field.

E_i = the expected frequencies based on past experience or on a hypothesized distribution function.

If there is no discrepancy between the observed and expected values for each category, ^{2} is small, and the data correlates well with the expected frequency distribution.

CHAPTER 10

RESULTS AND DISCUSSION OF STATISTICAL ANALYSIS

Introduction

Paired data z and t tests of the results from field and laboratory investigations at 40 locations (Figure 18) throughout the Tucson area were used to test the hypothesis that the laboratory pseudo-consolidation test overestimates the collapse potential which occurs in the field. The χ^2 goodness of fit test was employed to evaluate the collapse potential probability distribution of the field results with collapse potential indices. The χ^2 test was further used to correlate the field and laboratory results with collapsible soil distributions and damaged building distributions identified by Ali (1987) and Crossley (1969), respectively. Contour plots were produced of the field and laboratory C_p values for the area of study to further enhance the correlation with previous works.

The position of the test sites depicted in Figure 18 was established by superimposing a grid coordinate system over a map of the Tucson area shown in Figure 14 (p. 54). The nearest cross streets and the grid coordinates for each test site are included in Table 5. A transparency of

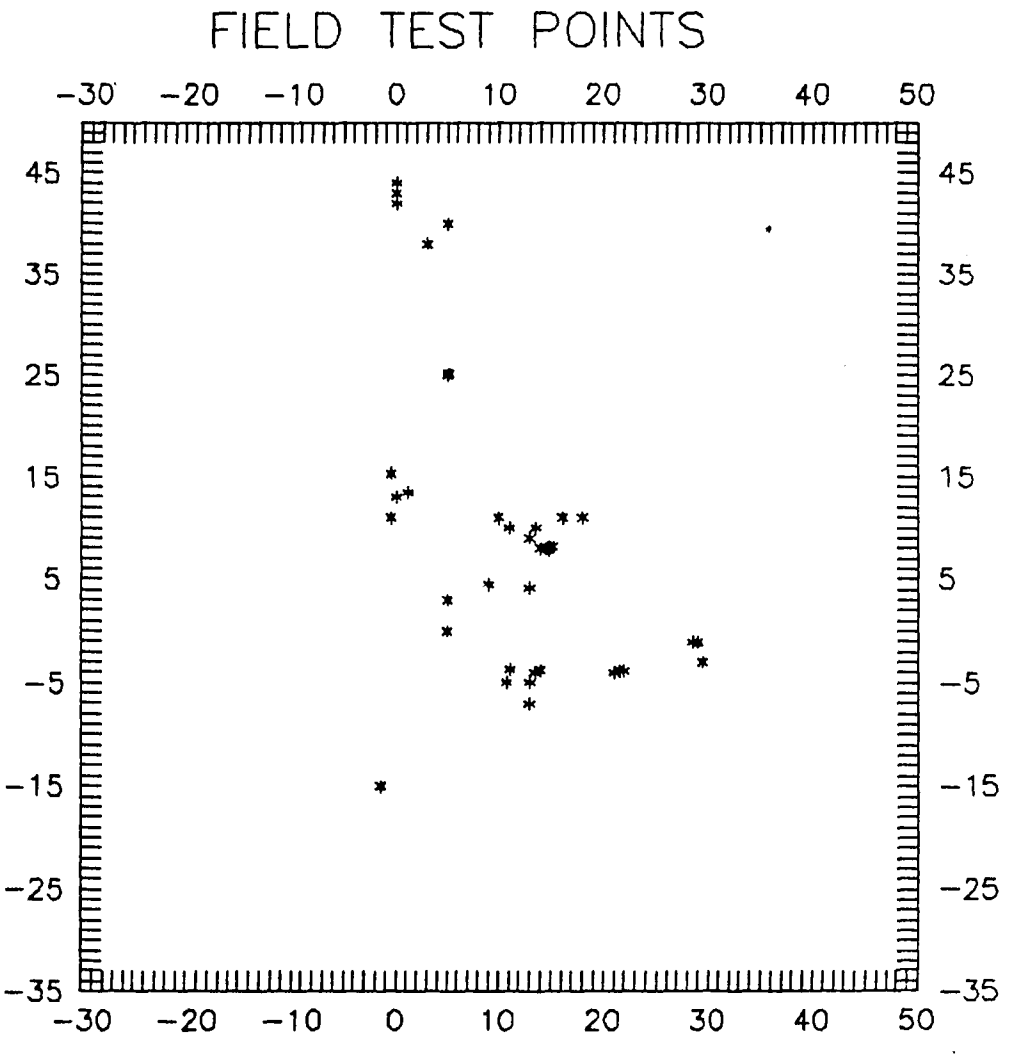


Figure 18. Distribution of Test Location Points

Table 5. Location of Test Sites Within the Study Area

Site #	Nearest Cross Streets	Grid Coordinates X	Y
1	Oracle and River Rd.	-5.0	15.3
2	Palo Verde and 32nd St.	10.8	-5.2
3	" " "	11.0	-5.0
4	" " "	11.2	-4.8
5	North Phoenix	--	--
6	Fontana and Drexel	-1.5	-15.0
7	Fair Oaks and Grant	14.0	8.0
8	Swan and Grant	15.0	8.0
9	" "	15.2	8.2
10	" "	14.8	8.2
11	" "	14.5	8.0
12	" "	14.8	7.8
13	" "	16.1	11.0
14	Swan and River Rd.	18.0	11.0
15	Kino Blvd. and Broadway	5.0	0.0
16	1st Ave. and Lambert	3.0	38.0
17	" "	0.0	42.0
18	" "	0.0	43.0
19	" "	0.0	44.0
20	" "	5.0	40.0
21	Cherry and 5th St.	5.0	3.0
22	1st Ave. and Wetmore	1.1	13.5
23	Skyline and Orangegrove	5.0	25.0
24	" "	5.1	25.1
25	Pantano Rd. and Broadway	29.0	-1.0
26	" "	28.5	-1.0
27	" "	29.0	-1.1
28	Van Buren and 29th St.	21.0	-4.0
29	" " "	21.5	-3.8
30	Freemont and 29th St.	17.0	-4.2
31	Oracle and Navajo Rd.	-0.5	11.0
32	Stone and Prince	0.0	13.0
33	Chrysler and Dodge	13.0	-7.0
34	Saguaro and 32nd St.	25.0	-3.8
35	Woodland and 32nd St.	14.0	-4.5
36	Freemont and 29th St.	13.0	-4.2
37	Eastgate and Monte Vista	13.5	9.0
38	Columbus and Grant	13.0	9.2
39	Walnut and Blacklidge	13.6	10.0
40	Country Club and Speedway	9.0	4.5
41	Kelvin and Ft. Lowell	10.0	11.0
42	Winstel and Blacklidge	11.0	10.0
43	Pantano Wash and 22nd St.	29.5	-3.0

Figure 18 is included in the jacket on the back cover for use with Figure 14 and subsequent plots.

Descriptive Results

Descriptive statistics for the initial compression due to loading (C_i) and the compression upon saturation (C_p) are given in Table 6. As shown in the table, the mean values for both components of collapse and for total collapse itself, as determined by the laboratory pseudo-consolidation tests, were higher than the mean values determined from the field tests. Field results for collapse due to saturation upon loading suggests that soils in the Tucson area are moderately collapsible (mean C_p = 3.5%), while the laboratory results suggest higher collapsibility (mean C_p = 5.2%). The standard deviation indicates that the response among collapsible soils when loaded and wetted varies more than when simply loaded "dry," regardless of the test type. The cov is higher for the field tests than for the laboratory tests. This indicates that there is greater uncertainty in the field-determined values for C_p and C_i than in the values determined from the laboratory tests. The variability is greatest for measurement of the initial insitu compression.

Descriptive statistics for the insitu moisture content at the time of testing and the density of the soil

Table 6. Descriptive Statistics for Collapse at 1-2 Ft. of Depth

	FIELD COLLAPSE TEST			PSEUDO-CONSOLIDATION TEST				
	Initial Compr. (%)C_i	Compr. Upon Wetting (%)C_p	Total Compr. (%)C_t	Initial Compr. (%)C_i	Compr. Upon Wetting (%)C_p	Total Compr. (%)C_t	Insitu Moisture Content (%)w_i	Sample Dry Density $\gamma_{d(pcf)}$
n	43	43	43	40	40	40	42	14
mean(y)	0.706	3.506	4.212	3.178	5.196	9.092	6.087	85.04
st dev(s)	1.240	3.406	3.695	2.210	4.025	4.112	3.092	7.798
cov	1.756	0.972	0.877	0.695	0.775	0.452	0.508	0.092

samples that were determined in the laboratory are also presented in Table 6. On average the soils were of relatively low density ($\overline{\gamma_d}$ = 85 pcf) and were tested under dry conditions (% $\overline{w_c}$ = 6%) with little deviation from the means. The low cov values for w_i and γ_d points out with a fair degree of certainty that the data is both consistent and reliable.

Paired Data Analysis

The paired data tests were performed using the computer package "MINITAB" (Ryan et al 1985) with the data listed in Table 7. The results of the paired data z-test are summarized in Table 8.

As indicated in the table, the weight of evidence for rejecting H_o is highly significant, i.e., $p < 0.0000$. The paired-data test proves conclusively that the mean difference γ_d between the percent of collapse measured by the pseudo-consolidation and insitu collapse tests for all soils in the Tucson test area is significantly greater than zero. Therefore, the results strongly support the hypothesis that laboratory consolidation tests over-estimate the collapse potential in the field.

Student-t Test

A set of field and laboratory test results were selected from regions previously identified as having soils with high collapse susceptibility and where

Table 7. Collapse Potential Data for the Paired Data z-Test

Site No.	Intensity Collapse (%)C_p	Pseudo-Cons. (%)C_p	Difference in (%)C_p ($y_{i1}-y_{i2}$)
1	0.40	4.96	4.56
2	0.40	4.30	3.90
3	0.00	0.15	0.15
4	0.12	0.76	0.64
5	4.20	3.91	-0.29
6	1.60	5.69	4.09
7	2.00	2.30	0.30
8	0.00	2.90	2.90
9	0.00	2.28	2.28
10	7.44	7.05	-0.39
12	0.80	1.89	1.09
13	2.00	2.34	0.34
14	1.33	2.66	1.33
15	1.60	3.95	2.35
16	0.60	6.05	5.45
17	0.10	2.17	2.07
18	1.40	8.90	7.50
19	0.70	1.30	0.60
20	0.80	5.20	4.40
21	1.60	3.01	1.41
22	0.00	2.77	2.77
23	0.40	1.30	0.90
24	6.40	11.67	5.27
25	0.80	0.57	-0.23
28	6.23	7.53	1.30
29	2.40	5.49	3.09
30	7.88	11.30	3.42
31	8.67	10.26	1.59

Table 7. (Con't.)

Site No.	Intensity Collapse (%)C_p	Pseudo-Cons. (%)C_p	Difference $(y_{i1}-y_{i2})$
32	6.40	8.19	1.79
33	0.00	4.97	4.97
34	5.40	13.82	8.42
35	9.44	7.33	-2.11
36	6.93	14.01	7.08
37	10.68	9.11	-1.57
38	7.69	7.83	0.14
39	11.00	10.92	-0.08
40	6.50	7.50	1.00
41	6.40	9.39	2.99
42	7.60	15.78	8.18

Table 8. Results of the Paired Data z-Test

<u>Statistics</u>

$n = 40$ $\bar{d} = 2.317$ $s_d = 2.623$

<u>Hypothesis</u>

H_0: $\mu_d = 0$

H_1: $\mu_d \neq 0$

<u>Test Statistic and p-Value</u>

$z = 5.59$ $p \approx 0.0000$

extensive damage to buildings had been observed. A Student-t test was used to test the hypothesis that the collapse potential, C_p, is over-predicted by laboratory pseudo-consolidation tests of all soils in collapse susceptible areas of Tucson where building damage has been identified. Data for the t-test are presented in Table 9. The results are presented in Table 10.

The parameter of interest chosen for the student-t test was the true mean difference μ_d between the percent of collapse measured by the pseudo-consolidation and insitu collapse tests for all collapsible soils in the Tucson test area.

As indicated in Table 10, the weight of evidence for rejecting H_o is significant, (i.e., $p = 0.016$ and $t_{.05} = 1.706$). However, for a highly significant critical value of $t_{.01} = 2.479$, the weight of evidence for rejecting the null hypothesis (H_o) is insufficient. The paired data Student-t analysis indicates that the true mean difference between the percent of collapse measured by the pseudo-consolidation and insitu collapse tests is significantly greater than zero.

The results of both the t-test and the z-test suggest that the magnitude of collapse which was attained in the field with free access to water was less than that attained by laboratory pseudo-consolidation methods of undisturbed samples. There is also an indication that the

Table 9. Collapse Potential Data for the Paired Data t-Test

Site No.	Insitu Collapse (%)C_p	Pseudo-Cons. (%)C_p	Difference in (%)C_p (y_{i1}-y_{i2})
26	6.00	5.09	-0.91
28	6.23	7.53	1.30
30	7.88	11.30	3.42
31	8.67	10.26	1.59
32	6.4	8.19	1.79
34	5.40	13.82	8.42
35	9.44	7.33	-2.11
36	6.93	14.01	7.08
37	10.68	9.11	-1.57
38	7.69	7.83	0.14
39	11.00	10.92	-0.08
40	6.50	7.50	1.00
41	6.40	9.39	2.99
42	7.60	15.78	8.18

Table 10. Results of the Paired Data t-Test of the differences between the field and laboratory C_p results for collapsible soils in the test area.

Statistics

$n = 14$ $\quad \bar{d} = 2.231$ $\quad s = 3.454$

Hypothesis

$H_0: \quad \mu_d = 0$

$H_1: \quad \mu_d \neq 0$

Test Statistic

$t = 2.42$

Critical Values

$t_{.05} = 2.42$ $\quad t_{.01} = 2.479$ $\quad p = 0.016$

---

severity of damage anticipated from soil collapse may be less than previously expected due to lower degrees of differential settlement.

However, there is no evidence to declare that the amount of collapse achieved by either method was the maximum that might occur. The amount of collapse is a function of the amount of collapse susceptible soil that actually becomes saturated. It is also responsive to the environmental conditions of stress, structure, cementation, and initial moisture conditions. The degree of saturation prior to collapse limits the amount of compression which occurs and is discussed more fully in the Insitu Moisture Content section of this chapter. There may also be some question about the saturation reached in the field upon free access to water. It has been demonstrated in the laboratory (Houston et al 1988) that reduced vertical deformations of a soil sample result when the soil is less than fully saturated. Others have expressed similar concerns when evaluating collapse conditions in the field (Duffy et al 1986).

As was discussed in Chapters 5 and 6, accurately measuring the extent of wetting in the field for this investigation proved a difficult task. An overestimate of the saturated zone reduces the value of the collapse potential, as indicated by equation 8.3.

Normal Probability Plot

A plot of the differences in sample data versus a theoretical normal probability distribution of the differences verifies the normality assumption for the t-test. Figure 19 is a normal probability plot of $(Y_{i1} - Y_{i2})$ for the sample of all collapsible soils in the Tucson test area listed in Table 9. The plot roughly emulates a straight line.

As shown in Figure 20, a normal probability plot of the pseudo-consolidation test data from Table 4 (p. 68) also approximates a straight line, as expected. However, a similar plot of the field test data (Figure 21) reveals a curve with three distinct slopes. This suggests that there is a change in soil behavior for various levels of collapse, given a normal probability distribution. The influence of the soil collapse parameters may shift in dominance as the degree of deformation increases. The changes in slope of the normal probability plot occur at approximately C_p = 2% and C_p = 5%, which correspond to the collapse criteria indices evaluated by Sabbagh (1982) and depicted in Table 11.

χ^2 Test - Collapse Potential Indices

The χ^2 test was used to assess the possible relationship between the field collapse normal probability distribution and the criteria of high (HC), medium (MC),

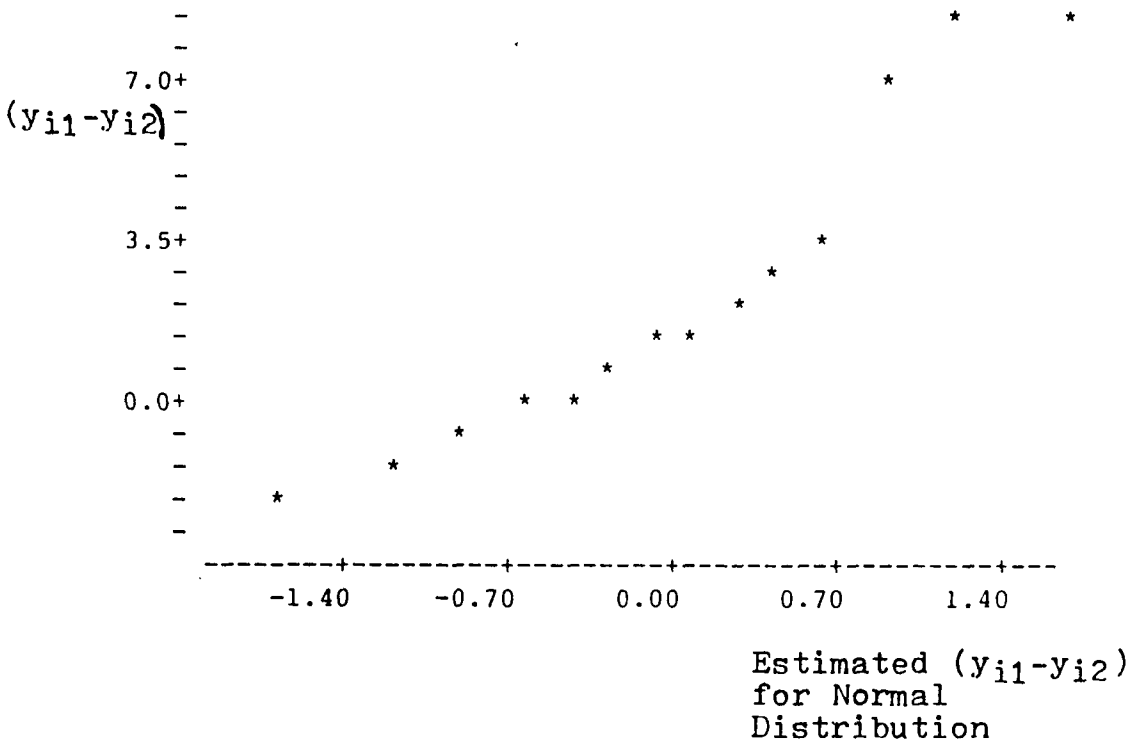


Figure 19. Normal Probability Plot of the Differences in Test Data for the Paired Data t-Test

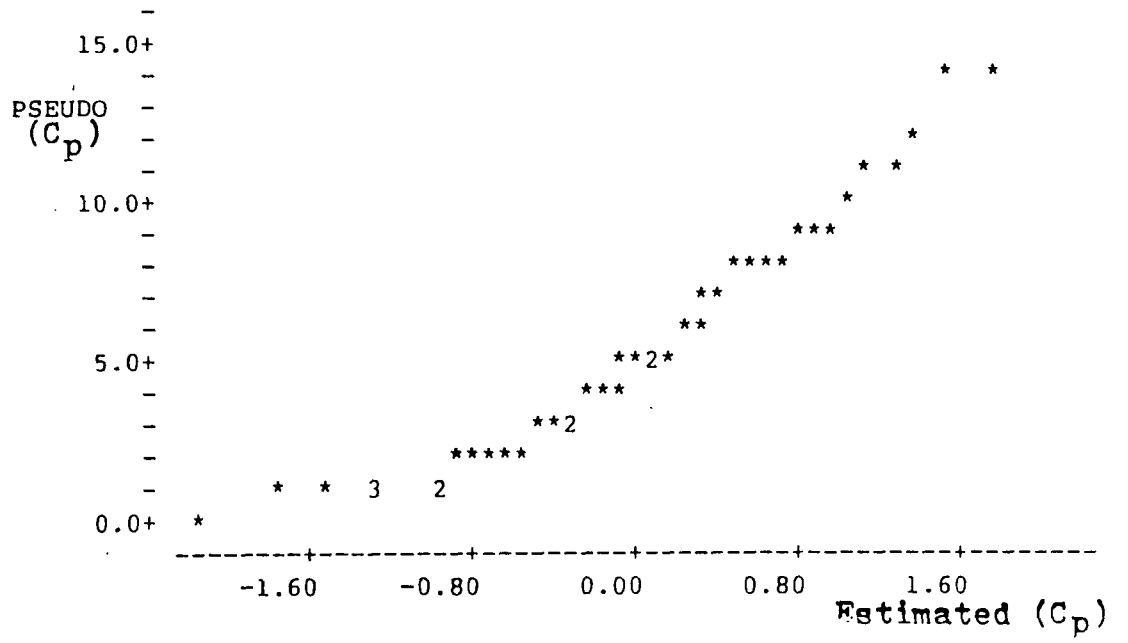


Figure 20. Normal Probability Plot of the Sample Collapse Potentials

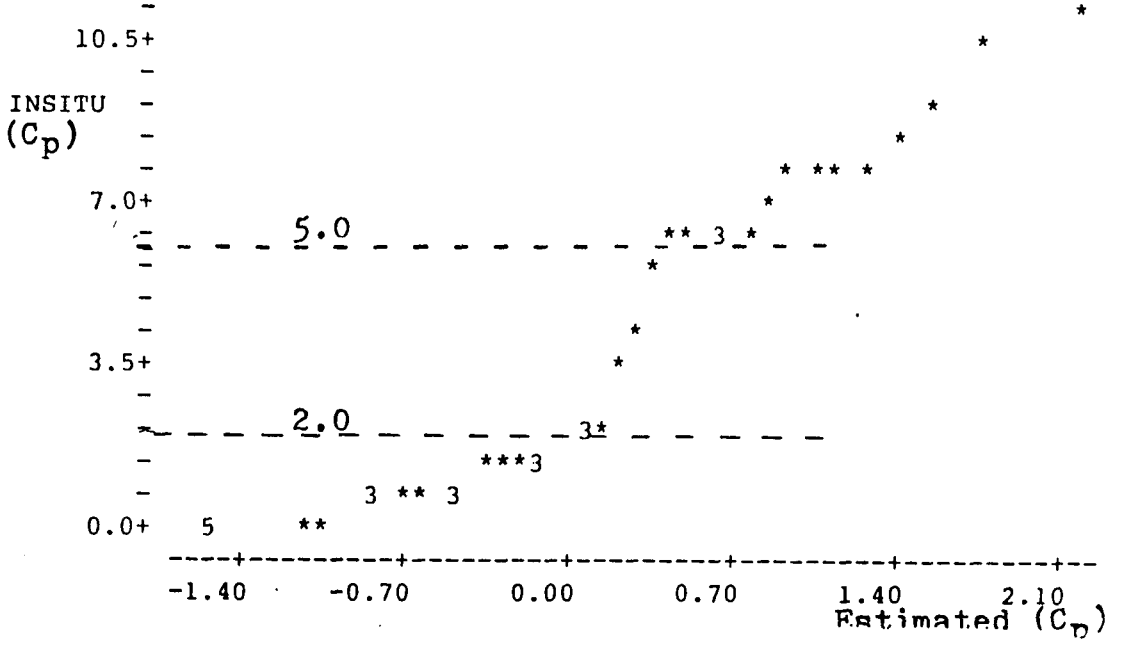


Figure 21. Normal Probability Plot of the Field Collapse Potentials

and low or no collapse (NC) as presented in Table 11. To determine the expected values, an overlay of Figure 18 was used with the contour plots of collapse potential values in the Tucson area that were developed by Ali (1987) using Kriging Methods (Figures 22, 23). The expected values for the ^{2} test were determined by counting the number of data points within the range of contour intervals for the three collapse potential categories. The observed values were the number of field observations with collapse potential values corresponding to the C_p indices of Table 11. The C_p contours presented in Figures 22 and 23 were developed from a data base of 125 test points in the 0 to 1 ft. depth range and from 286 test points in the 1 to 2 ft. depth range.

Results of the χ^2 test and the critical value at the 0.05 probability level are presented in Table 12.

It is observed that the total test statistic is very large, $\chi^2 = 252.51$, and that there is no relation between the distribution of collapse potential values measured in the field and collapse potential indices used by Sabbagh. The test statistic is very large relative to the critical value of $\chi^2_{crit} = 5.99$ for a rejection region of $\alpha = 0.05$. The rejection region (α) is the area in the upper tail of the probability distribution curve for χ^2 and denotes the region for rejecting the null hypothesis. The

Table 11. Collapse Potential Indices
(From Sabbagh 1982)

(%) C_p		CLASSIFICATION
$C_p > 5$	HC	highly collapsible soil
$2 < C_p \leq 5$	MC	moderately collapsible soil
$C_p \leq 2$	NC	low to non-collapsible soil

TAble 12. Goodness of Fit Test - Collapse Potential Indices

		Expected Values(C_p)	Observed Values(C_p)	χ_i^2
$\chi^2 = 252.51$	HC	35	16	10.31
$\chi^2_{crit} = 5.99$	MC	5	4	0.20
	NC	2	24	242

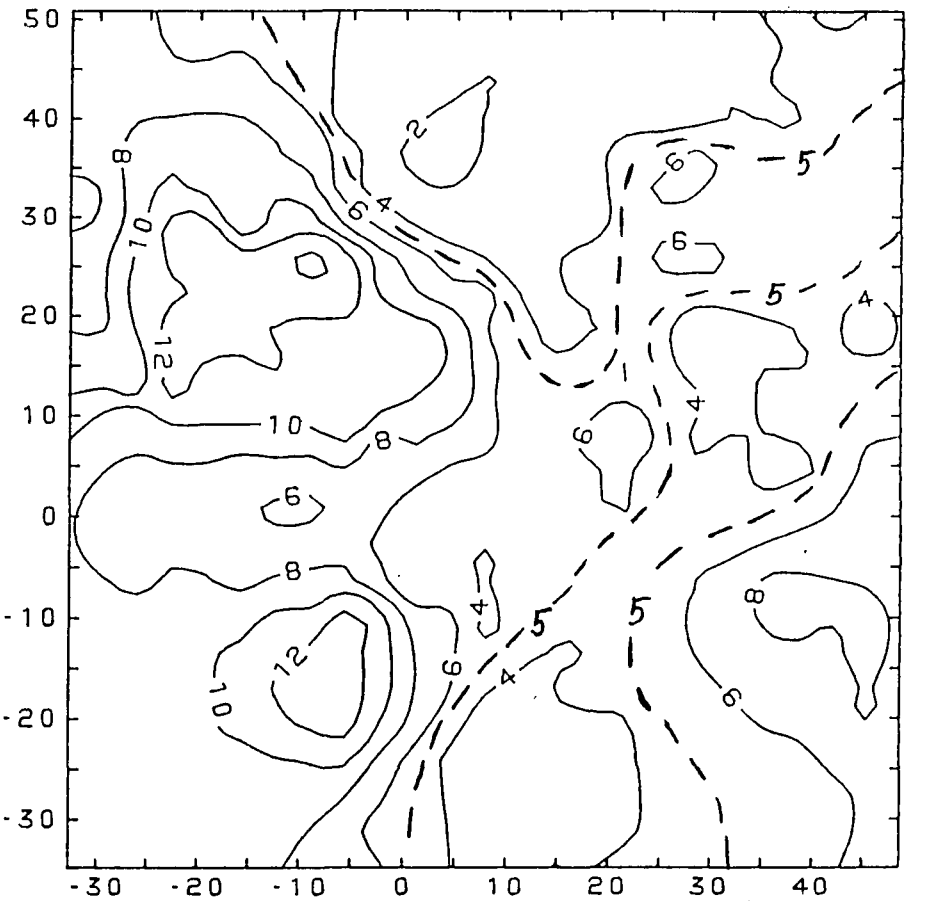


Figure 22. Contour Plots of the Values for C_p of Soil at 0-1 ft of Depth by Ordinary Kriging (From Ali 1987)

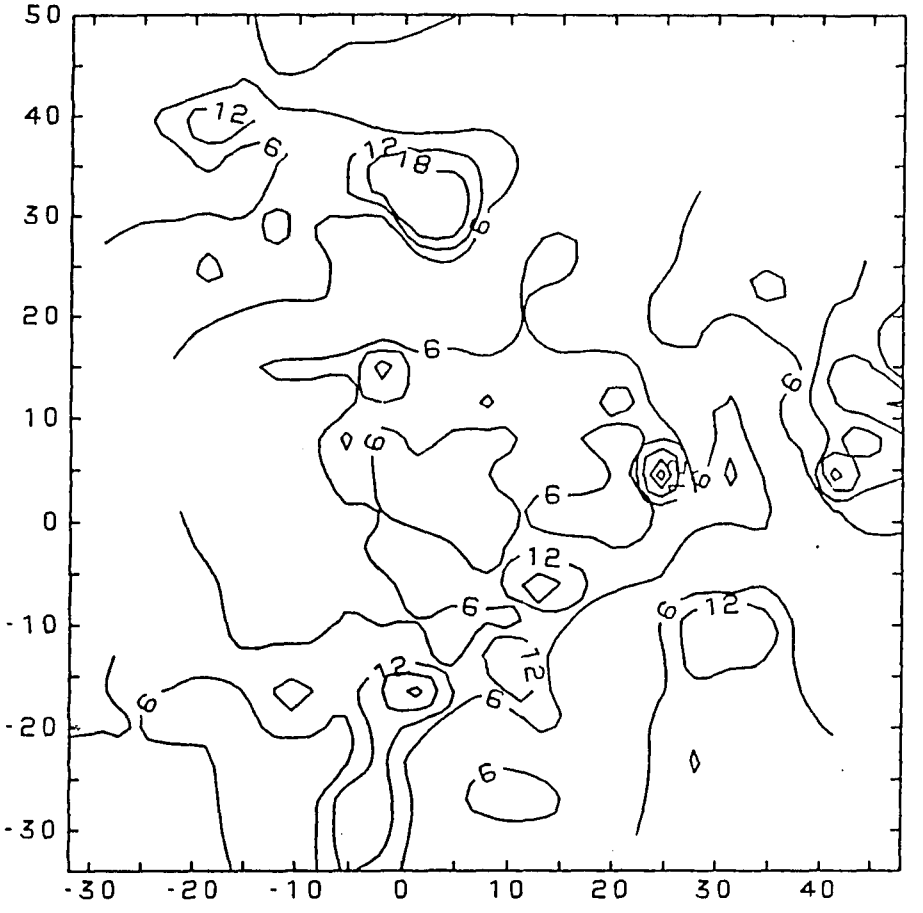


Figure 23. Contour Plots of the Values for C_p of Soil at 1-2 ft of Depth by Ordinary Kriging (From Ali 1987)

null hypothesis indicates that the observed values "fit" the expected probabilities or proportions.

An inspection of the individual components of the χ^2 test statistic indicates that there may be good correlation for the HC and MC values but the NC observed values greatly exceed the expected values. Part of the disparity is because the expected values are the result of laboratory consolidation tests, which have been shown to overestimate the collapse potential of insitu soil. Another explanation is that the observed values were not a truly random sample of soils in the test area and the results are skewed. The χ^2 test assumes random sampling. Judgment was used to identify the C_p = 5% contour interval of Figure 22.

A relation between the field collapse distribution function and collapse criteria could possibly be developed if a repeat of the current investigation were to produce similar results; i.e., three distinct slopes in a normal probability plot of the field data with breaks occurring at C_p's of approximately 2% and 5%. To have greater credibility, the investigation should field test more than 30 sites throughout the study area and the sites should be randomly selected.

χ^2 Test - Contour Plots

The collapse potential plots developed by Ali (1987) were based on laboratory pseudo-consolidation tests performed by WTI on "undisturbed" samples. The present investigation results of pseudo-consolidation tests performed by the author (Table 4, p. 68) were checked for "goodness of fit" with Ali's plots. The results of the χ^2 test, shown in Table 13, indicate a relatively good correlation between the laboratory results obtained in this study and those of previous investigations.

A contour plot of the pseudo-consolidation test results is presented in Figure 24. A plot of the field collapse potential results is shown in Figure 25 for comparison purposes. The plots from the current study were mathematically generated from only 42 locations and should not be projected beyond their positions on Figure 18. The figures show that the collapse potential zones produced from the field results are much smaller in intensity than those from the laboratory results. Comparison of the contours shown in Figure 24 with those shown in Figures 22 and 23 indicates poor correlation despite the good relation between the data as shown in Table 13. The disparity in the contour plots suggests that caution should be exercised when interpreting cartographs generated from limited data over a small area.

Table 13. χ^2 Goodness of Fit Test - Contour Plotting

		Expected Values(C_p)	Observed Values(C_p)	χ_i^2
$\chi^2 = 6.03$	HC	27	21	1.33
$\chi^2_{crit} = 5.99$	MC	10	12	0.20
	NC	2	6	4.50

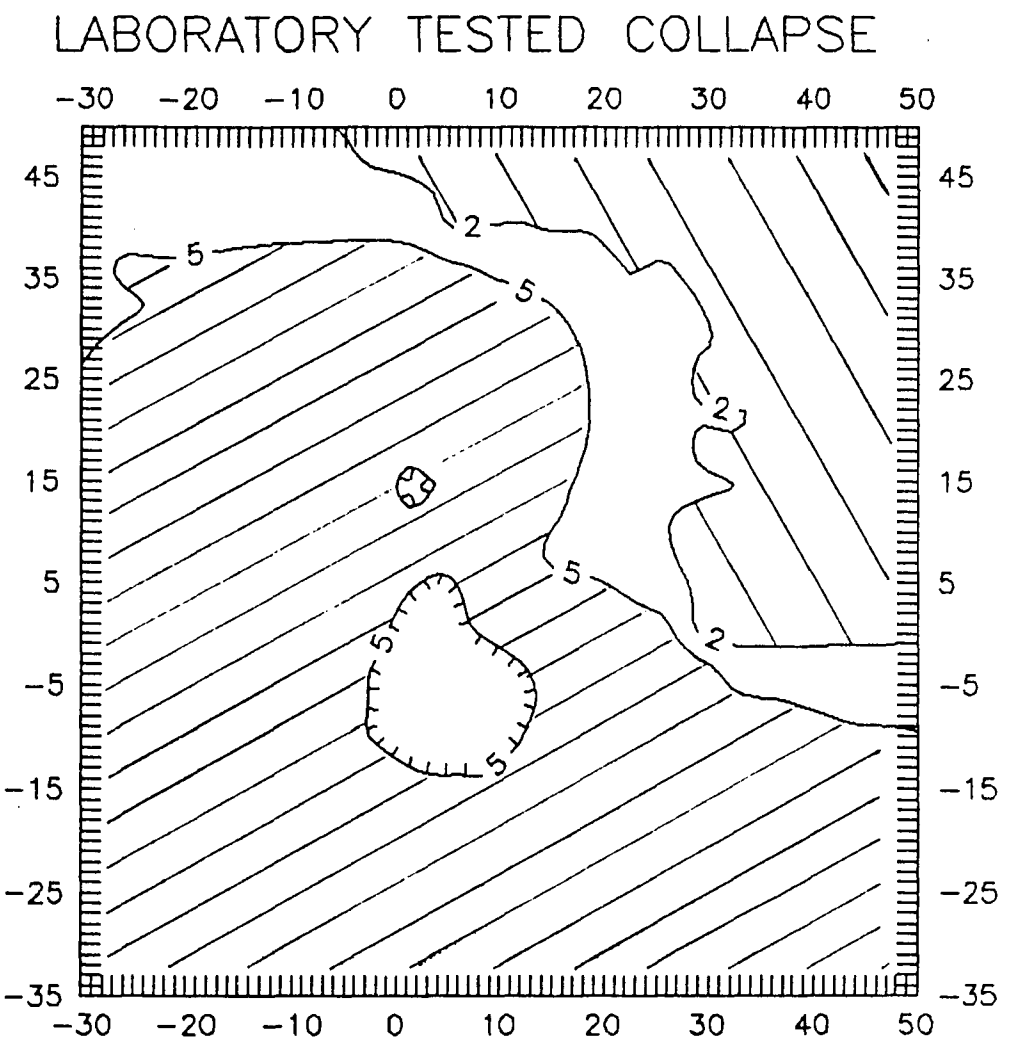


Figure 24. Contour Plots of the Laboratory Values for Cp of Soil at Depths of 1 to 2 ft by Ordinary Kriging

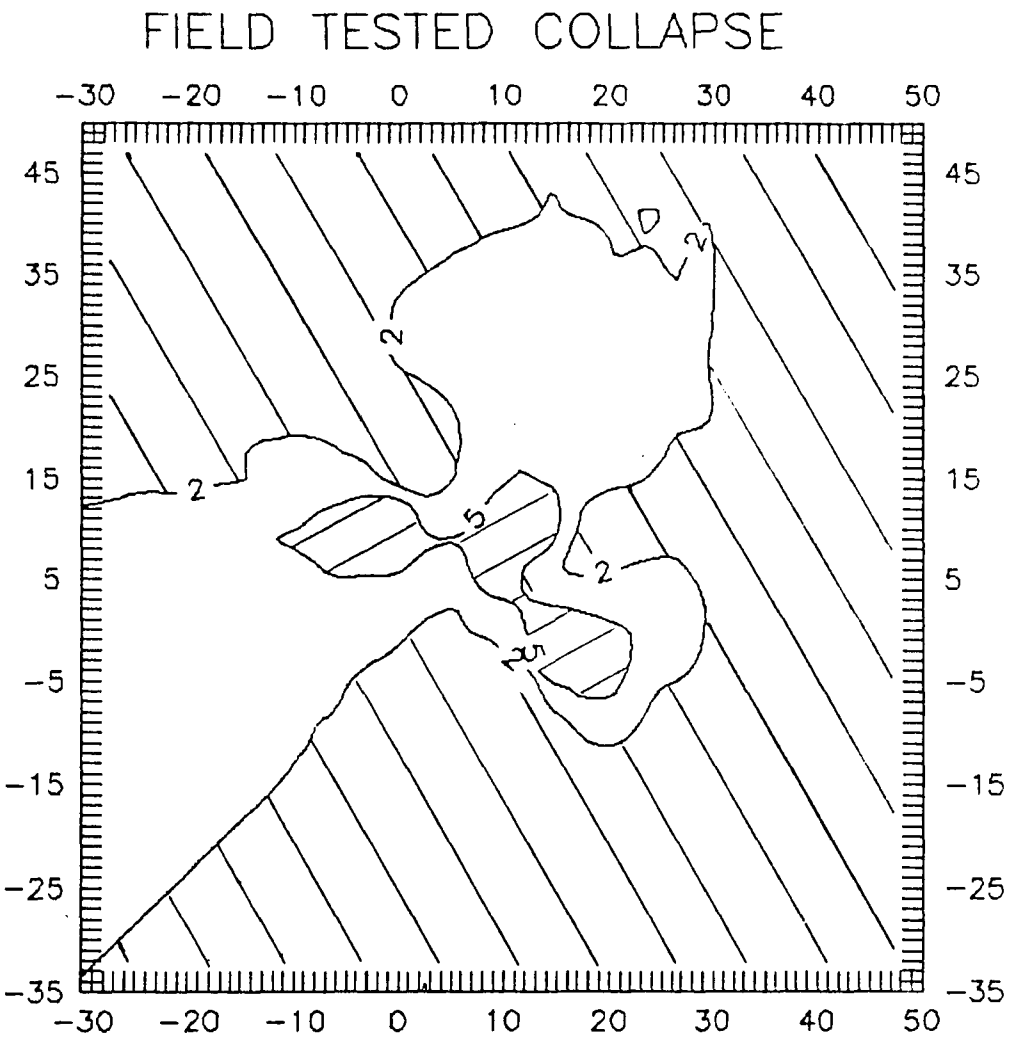


Figure 25. Contour Plots of the Field Values for Cp of Soil at Depths of 1 to 2 ft by Ordinary Kriging

Cartographs of Collapse Susceptibility

Collapse potential contour plots were produced from the laboratory pseudo-consolidation test and insitu collapse data sets. The irregularly spaced data points shown in Figure 18 (p. 84) were analyzed geostatistically by using the program GRID (Golden Software, 3.1). This program contains a geostatistical package for performing Kriging analysis on such data as that developed in this research. The program TOPO (Golden Software, 3.1) package was used to generate the contour plots from the "gridded" data set. The contour plots of the collapse potential values, as presented in Figures 24 and 25, use a co-ordinate system and scale which match the plots produced by Ali (1987). A similar plot, generated to encompass the study area used by Crossley, is shown in Figure 26.

The C_p contours in Figure 26 are in general agreement with the areas of foundation failures in Tucson depicted in Figure 7 (p. 38). An overlay of the data points for the field results shown in Figure 27 reveals that nearly all of the sites at which high collapse was measured are located on the Cemetery Terrace formation (Figure 8, p. 39) and correspond to areas of foundation failures depicted in Figure 7. The grid system used in Figures 26 and 27 correspond to Ali's plots but are on the same scale as Crossley's figures. Transparencies of Figures 26 and 27 are included in a packet inside the back cover which

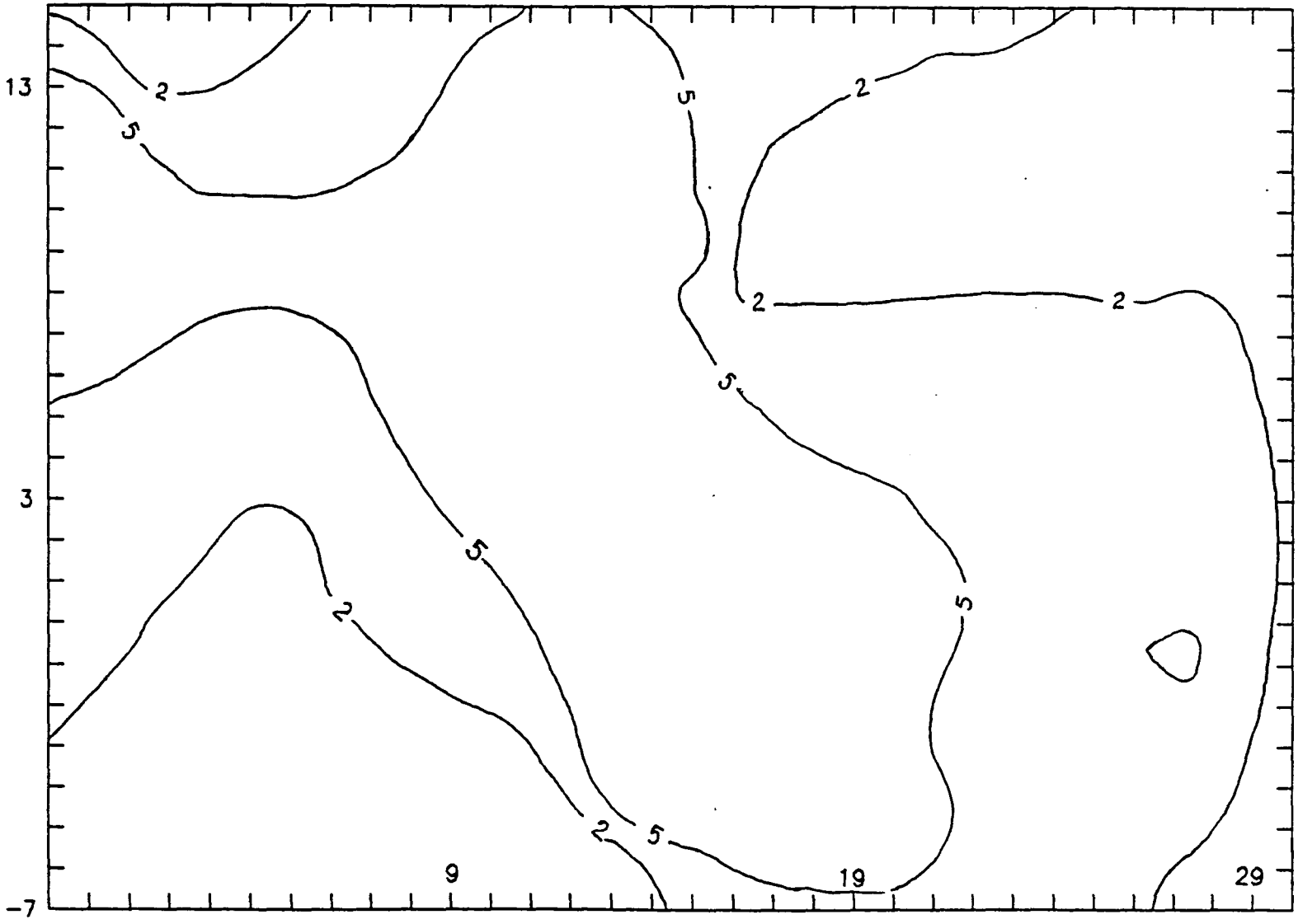


Figure 26. Contour Plots of the Field Values for C_D of Soil at Depths of 1-2 ft. by Ordinary Kriging for the Crossley Study Area

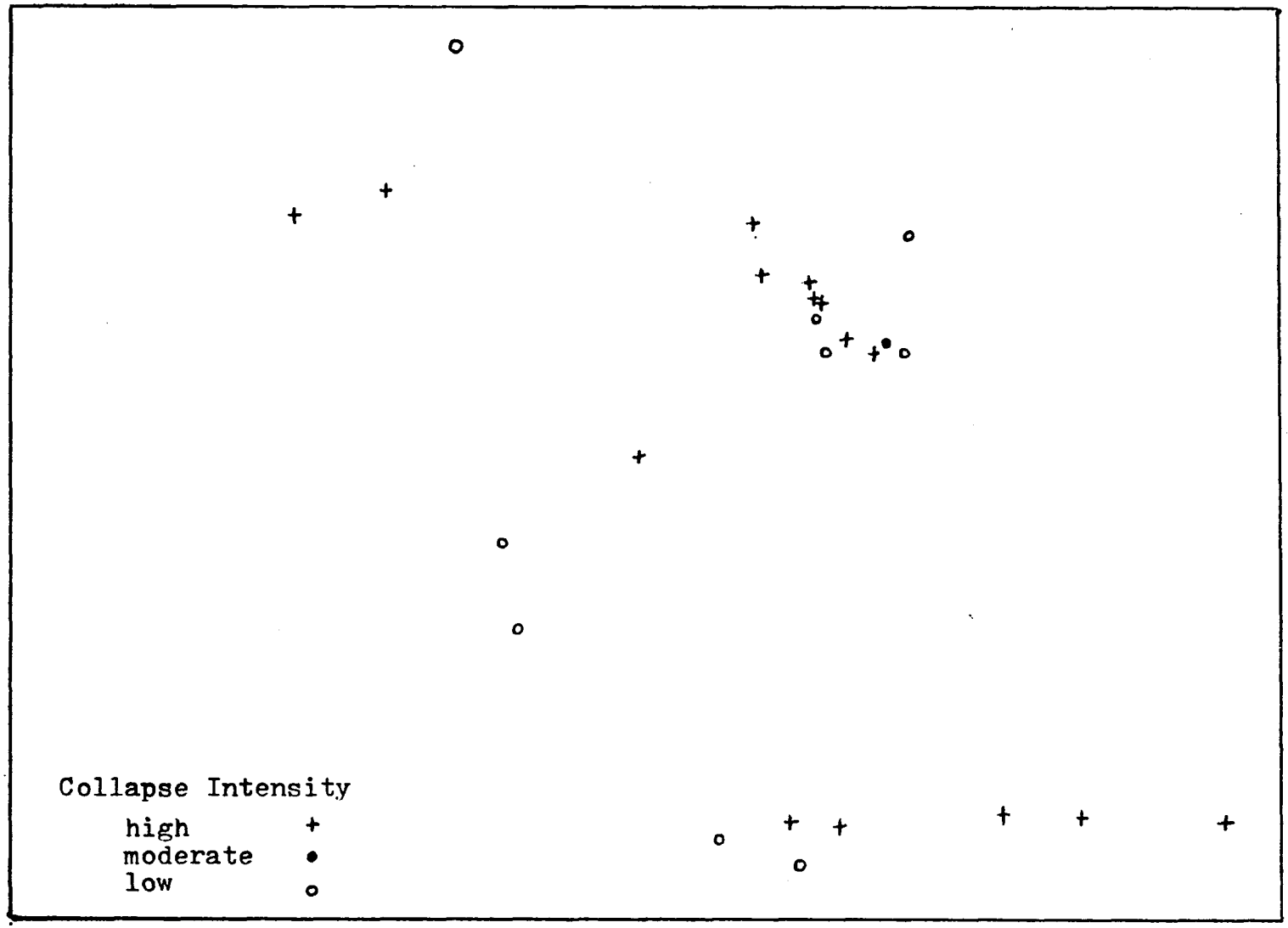


Figure 27. Overlay of Field C_p Values Superimposed on Areas of Foundation Failure and Terrace Deposits

relate the contours and data points for C_p to Figures 7 and 8.

Cartographs of Collapse Potential Probabilities

Ali also developed probability contour plots which could be used to determine probability zones for encountering soils having high, medium, and low or no-collapse susceptibility. The advantage of these plots to the engineer is the ability to predict with some measure of certainty the likelihood of encountering a collapsible soil in previously untested ground. To measure their reliability, data points from the laboratory results of the present study were plotted on Ali's probability contour maps for high, medium, and low collapse. The plots are presented in Figures 28, 29, and 30.

The distribution of the data points, as the data relates to probability contours for the various collapse potential indices, is presented in Table 14.

Table 14. Distribution of Laboratory C_p Data and C_p Probability Contours.

		Collapse Potential Indices		
		HC	MC	NC
Probability of Encountering a Collapsible Soil	.8-1	6	0	1
	.6-.8	10	0	0
	.4-.6	2	0	1
	.2-.4	0	2	2
	0-.2	5	11	1

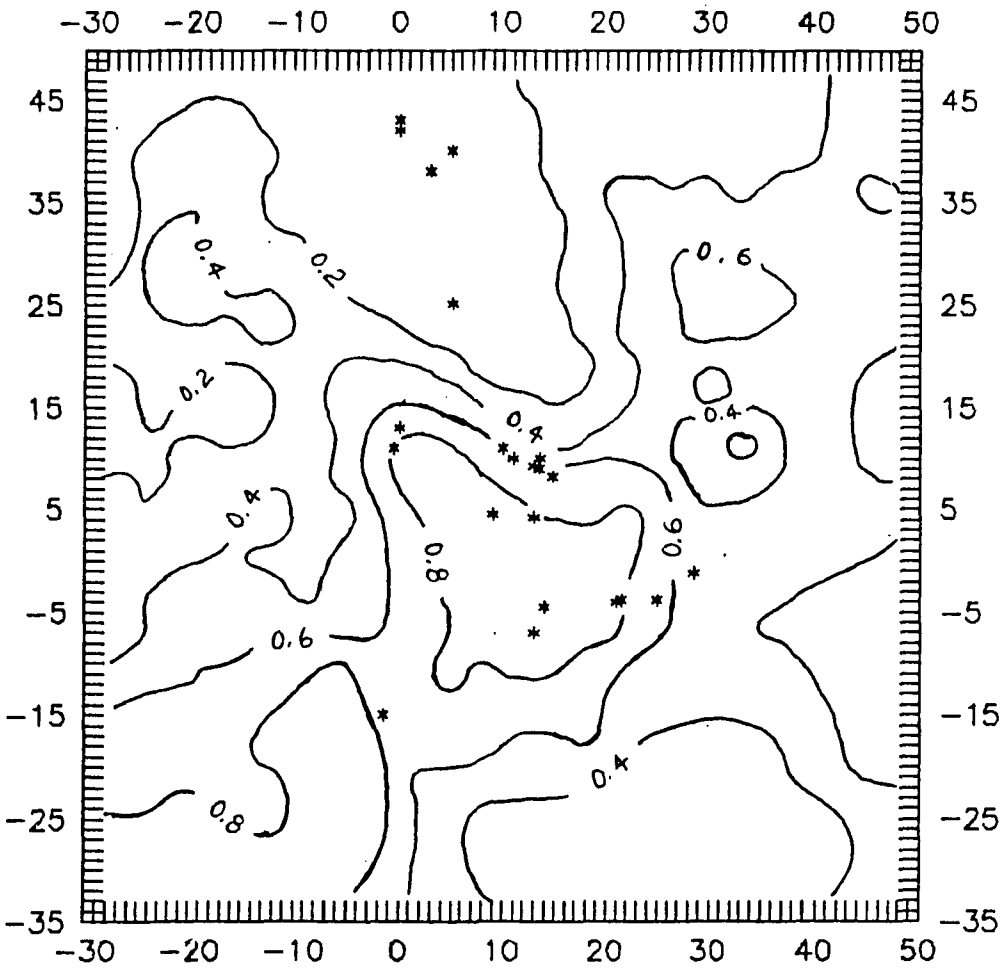


Figure 28. Location of Laboratory C_p Values Superimposed on the Probability Contour Map for HC

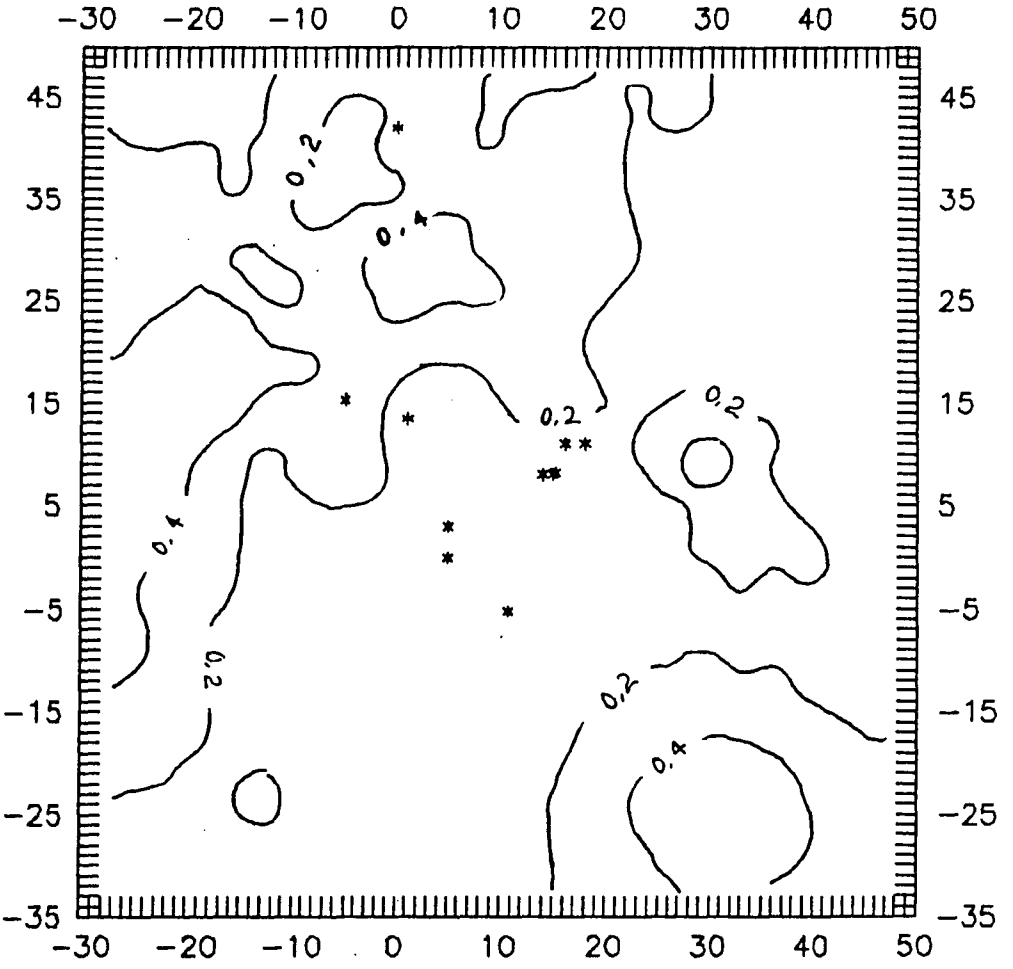


Figure 29. Location of Laboratory C_p Values Superimposed on the Probability Contour Map for MC

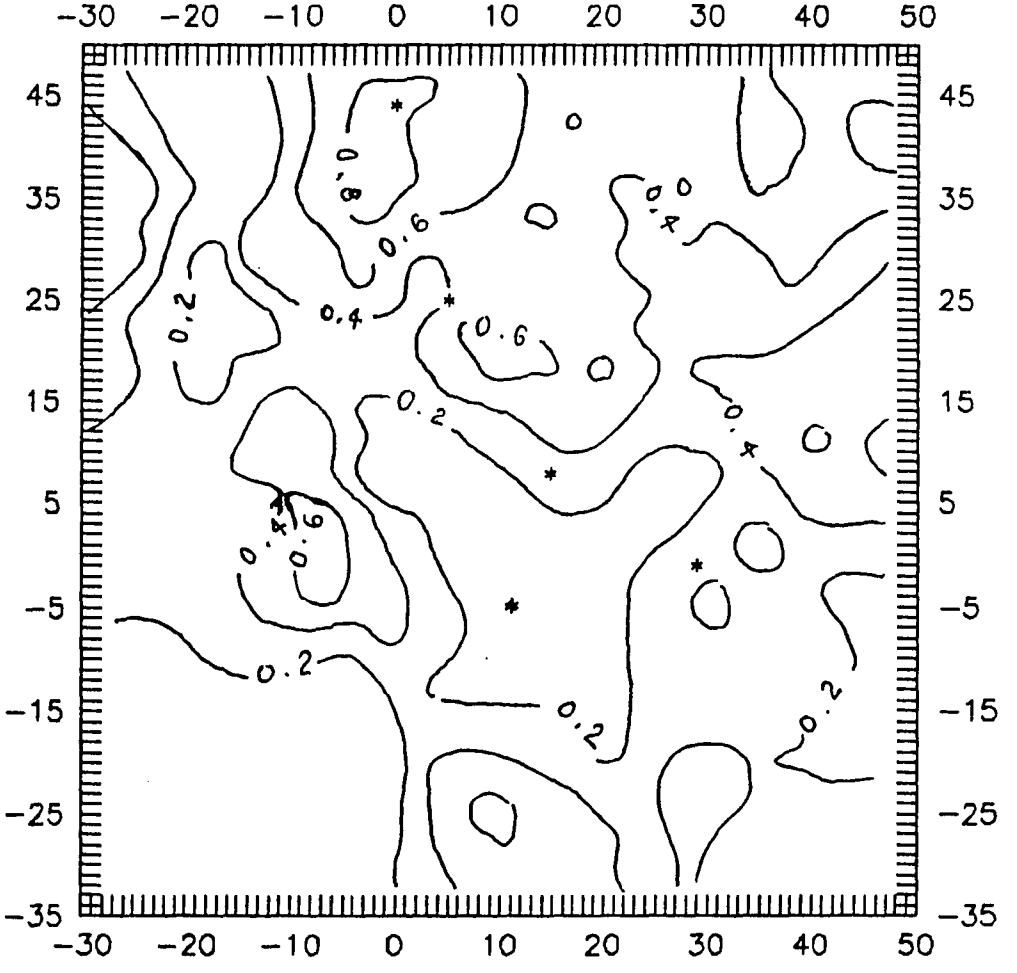


Figure 30. Location of Laboratory C_p Values Superimposed on the Probability Contour Map for NC

Areas with a 60% probability or greater of encountering soil with an HC rating contain 70% of the data points which were classified HC as a result of the laboratory pseudo-consolidation test. Areas with a 60% or greater probability for encountering soil with MC or NC collapse potential rating contain only 6% of the data points with C_p values associated with MC or NC. The high number of soils which were classified as MC or NC and are located in an area where the probability of encountering such a soil is low may be related to how the soils were tested and the moisture conditions at the time of testing. There is good representation of the soils associated with the estimated probabilities for HC.

Moisture Content Analysis

The results of collapse tests performed in the field and on undisturbed samples at various initial moisture contents are presented in Table 4 (p. 68). It is difficult to discern from the table any relationship between collapse potential and natural (initial) moisture content due to the differences among the soils and the environmental conditions (e.g., applied stress) at the time of testing.

However, regression plots of C_i and C_p versus initial moisture content reveal the patterns shown in Figures 31 through 36. The curves suggest that the collapse

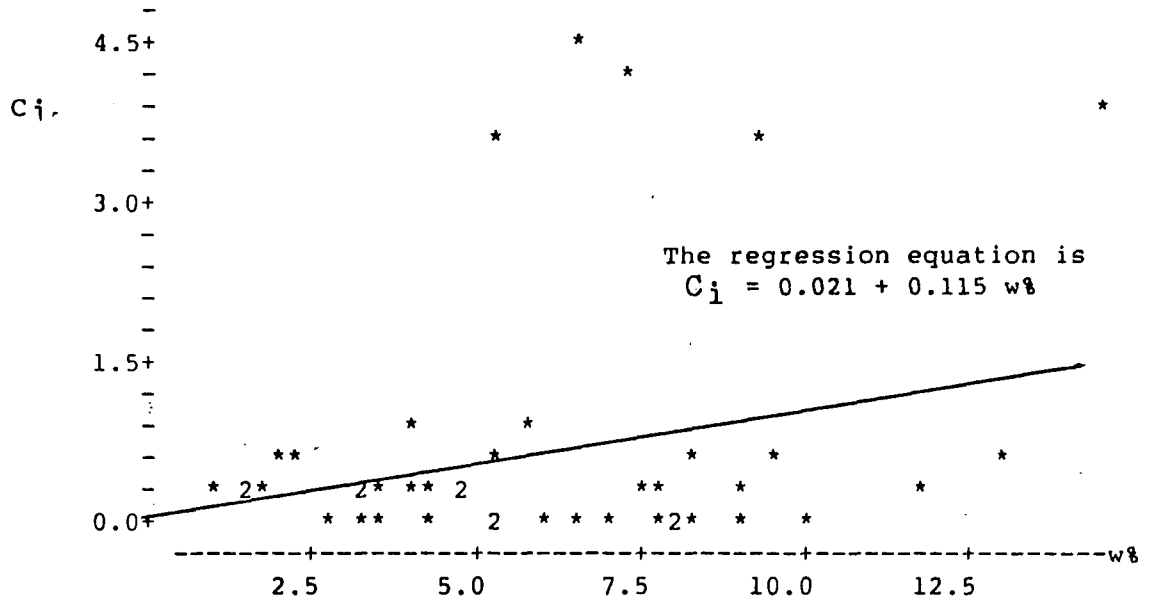


Figure 31. Regression Plot of Field C_i Values

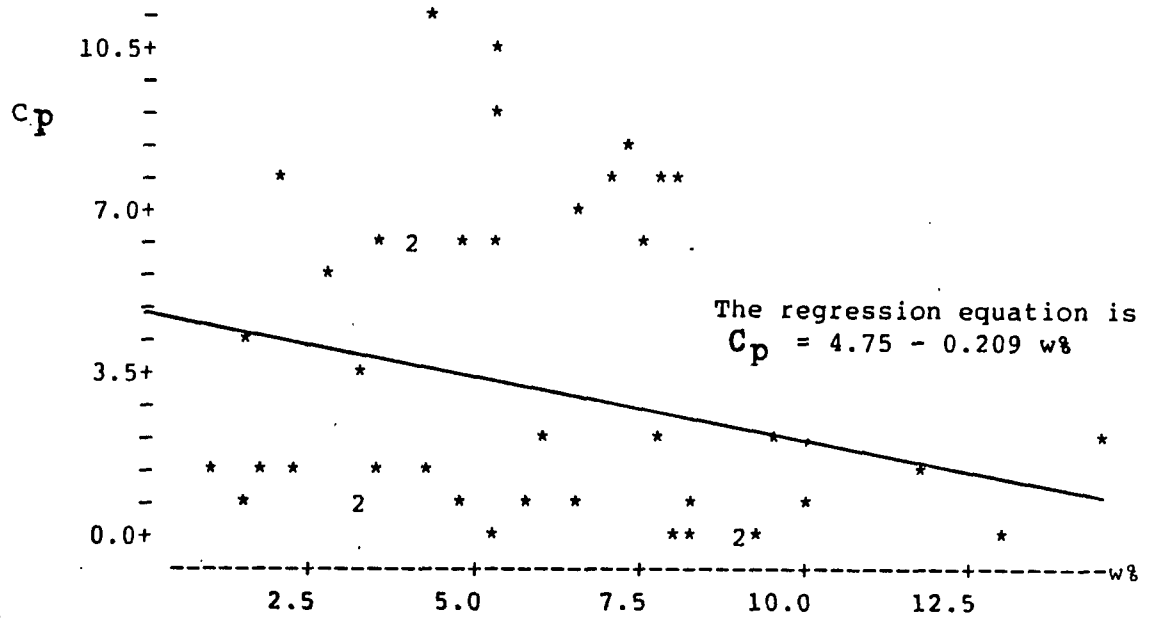


Figure 32. Regression Plot of Field C_p Values

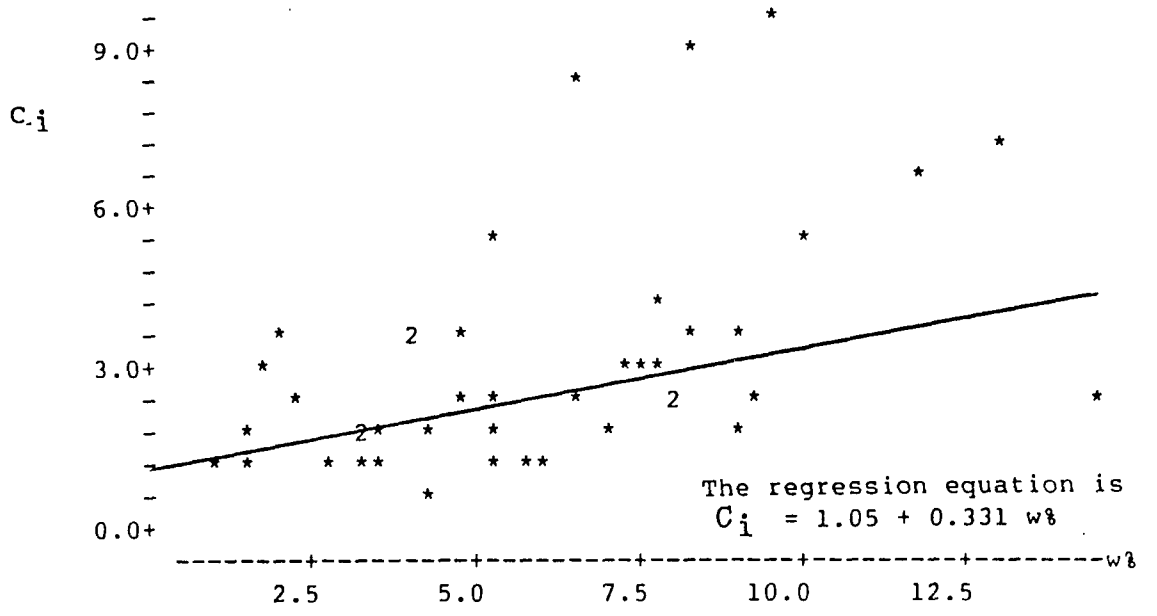


Figure 33. Regression Plot of Laboratory C_i Values

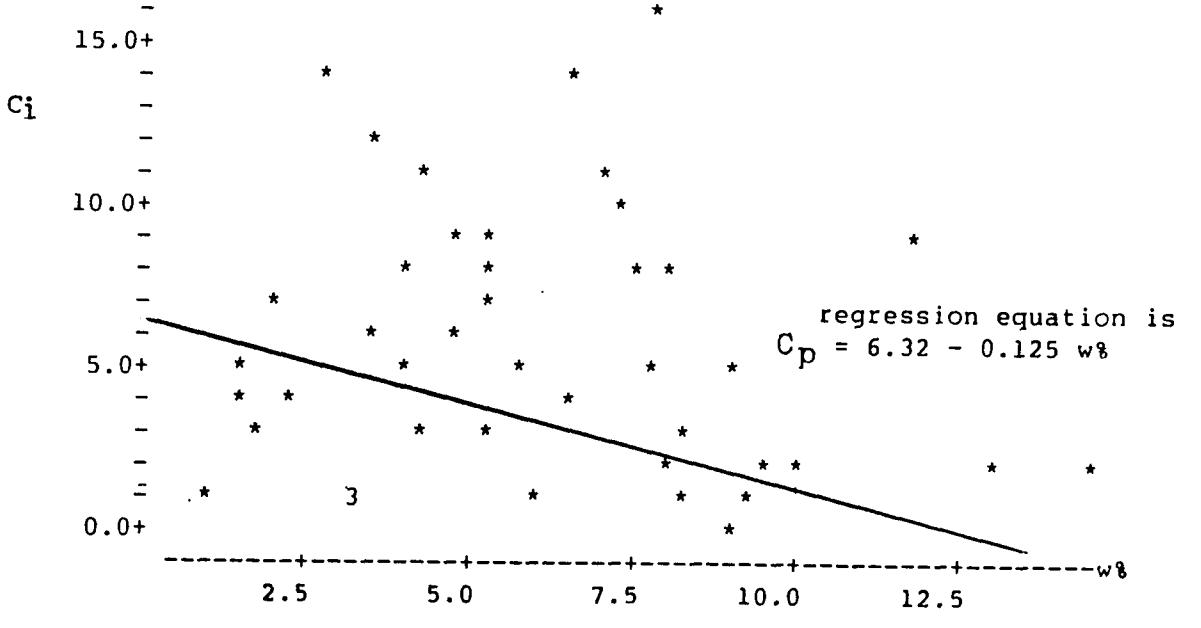


Figure 34. Regression Plot of Laboratory C_p Values

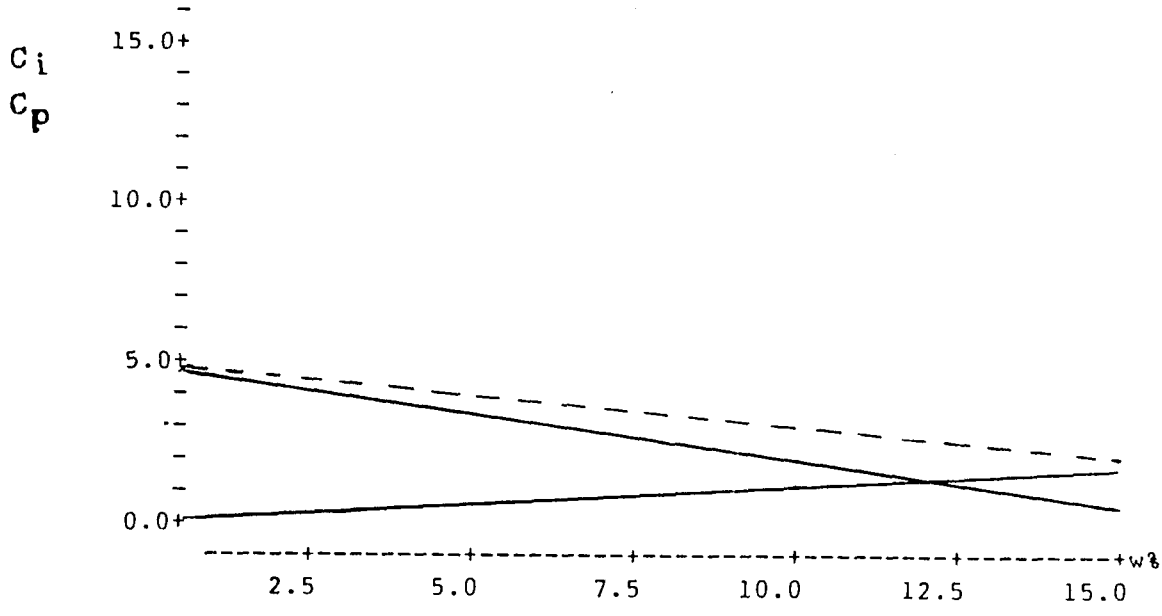


Figure 35. Combined Regression Plots for Field Tests

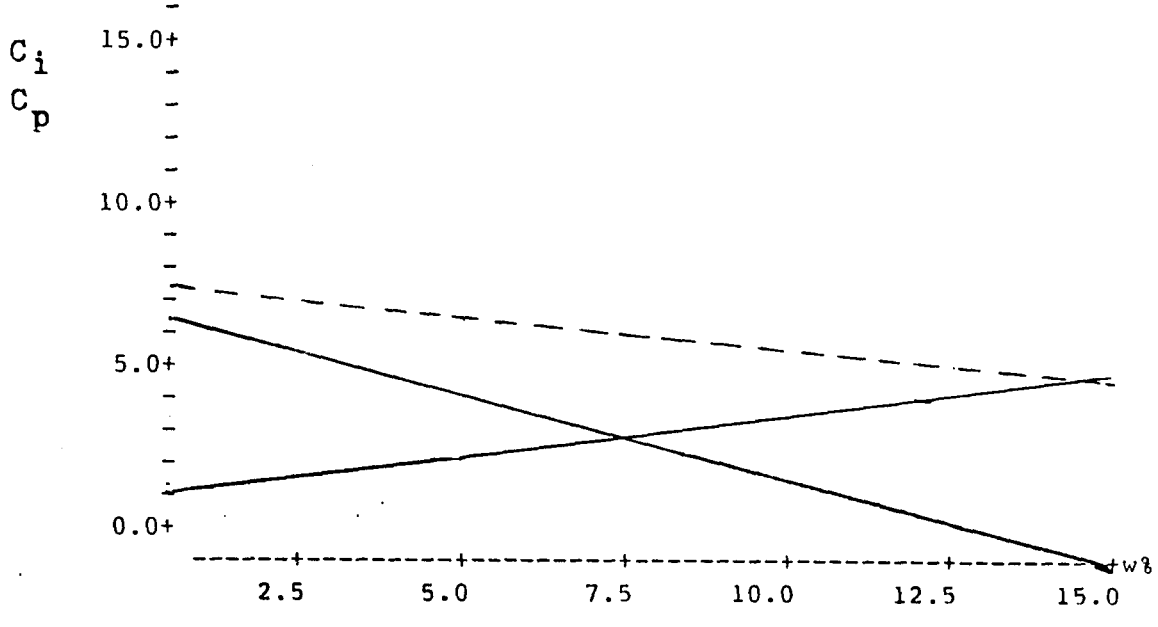


Figure 36. Combined Regression Plots for Laboratory Tests

deformation (C_p) tends to be larger than the initial deformation (C_i) for soils whose initial moisture content is less than approximately 8%, with the reverse being true for soils whose initial moisture content is greater than 9% (Figure 36). The following is given to explain these trends. When the moisture content is increased, the resistance between soil particles decreases. Capillary tension is reduced, and the clay bonds are weakened as the shear strength of the clay is reduced. Therefore, an applied stress will initiate greater particle movement under wet conditions than when the conditions are dry and all interparticle bonds are stronger. With full saturation the resistive forces are further reduced and the collapse process continues until equilibrium is reached.

When C_p values of the regression lines in Figures 35 and 36 are added for corresponding moisture contents, the results show a slight decrease in total compression for increased initial moisture content.

CHAPTER 11

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

A field test has been developed to assess the collapse susceptibility of soils rapidly and inexpensively. The results of insitu tests performed with the device and the results of laboratory pseudo-consolidation tests performed on samples taken from the same site were used to evaluate the severity and extent of collapse susceptibility of soils throughout the Tucson area. Principles of statistics were employed to analyze the data. Contour plot cartograms were developed to illustrate the findings and compare the results with those of previous studies. The following conclusions are drawn:

1. The insitu collapse test device measures the vertical deformations which occur in partially saturated soils that are subjected to stress and given access to additional moisture while under load. The test allows the collapse potential to be determined directly and avoids the uncertainties associated with sampling disturbance.

2. Laboratory pseudo-consolidation testing of ring sample specimens (both driven and continuous load) over-estimate the magnitude of collapse as measured in the field.

3. The collapse potential is, in part, a function of the soil moisture content at the time of loading and at the time of collapse.

4. A significant re_ation was not found between the collapse potential distribution of soils tested in the field and the cut off values of collapse potential used by Ali to define the indices HC, LC, and NC, although one may exist.

5. There is good correlation between the spatial variability of collapsible soils, as determined by geostatistical analysis, and the locations of recent aluvium terrace deposits and structurally damaged buildings.

Recommendations

1. Pseudo-consolidation test results are a function of the type and quality of the sample retrieved from the field. Standards for sampling equipment are directed towards cohesive clays and cohesionless sands. Standards need to be developed to enhance the quality of sampling for collapsible soils. It is recommended that sampling device standards be developed which account for the uniqueness of collapse susceptible soils.

2. A problem encountered during the field testing program was to identify the wetted zone during collapse.

Further study of the degree and extent of the wetting front would enhance field testing techniques.

3. It is suggested that a study be conducted to evaluate the possible relationship between the behavior distribution function of soil collapse in the field and collapse potential indices. The investigation should include test results from at least 30 locations randomly selected from within the region of study.

APPENDIX

SPECIFIC PROCEDURE

FOR THE PSEUDO-CONSOLIDATION TEST

Specific Procedure for the Pseudo-Consolidation Test

In this proposed method, the results of a one-dimensional "pseudo-consolidation test" are used to estimate "immediate settlement" under in situ moisture conditions and "collapse settlement" under saturated conditions. The test is called a pseudo-consolidation test because the sample is not saturated prior to load application as is the case in a conventional consolidation test. Instead, an undisturbed sample, approximately two inches in diameter and 0.75 inches thick, is placed at an in situ moisture content in an oedometer. Following the application of a nominal seating load, the vertical stress on the sample is increased to approximately 0.55 ksf. Displacement readings are taken periodically (usually 15 minutes apart for the first hour) until the difference between two successive readings, taken one hour apart, is less than .001 inches. The applied stress is then increased to a value double that of the existing stress, and displacement readings under the new stress are recorded as described above. The same procedure is followed until the vertical stress on the sample is approximately equal to or greater than the anticipated allowable foundation pressure. The anticipated allowable foundation pressure is predetermined on the basis of design loads, the foundation soil strength properties and the foundation system (type and size) that is more economical. The sample is then saturated while still under load, and displacement readings taken, again at 15-minute intervals for the first hour and after that approximately one hour apart,

until the difference between two successive readings is less than 0.001 inches. In general, the pseudo-consolidation test as described above can be completed within 24 hours; the results are usually reported in terms of total applied stress ($_a$) and % compression (strain) or void ratio.

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