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Settlements Due to Arid Collapsible Soils

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THESIS APPROVAL

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ABSTRACT

An abstract of the thesis of Heather L. Devine for the Master of Science in Civil Engineering presented May 14, 1997.

Title: Settlements Due to Arid Collapsible Soils

The arid alluvial deposits common to the western United States frequently undergo moderate to severe collapse when wetted that can threaten or destroy the stability of structures that are founded on such soils. Additionally, in some areas collapse has been known to occur due to overburden stresses alone. As development expands in these areas a method to estimate the degree of this collapse is essential.

Current settlement methods are reviewed and evaluated as they apply to collapsible soils, providing a basis for the development of a proposed settlement method for determining collapse. To aid in the determining the basis for the proposed settlement method finite element analyses are conducted to determine if the collapsible silts found near Nephi, Utah are cohesive or frictional in their behavior.

The role of the pressuremeter (PMT), an in situ testing device, and its value in determining relevant soil properties for collapse prediction is discussed. Testing methods are presented for dry, wet, and dry/wet combination conditions in the soil. Finite element analyses are conducted to determine the influence zone about the pressuremeter probe and the effects of non-coalescing moisture patterns on PMT test results.

A comprehensive evaluation of a field test conducted near Nephi, Utah provides necessary data for the development of the proposed settlement method. A component of

settlement particular to collapsible soils is identified and factors that contribute to this component are discussed. The collapse strain, a property of metastable soils that is essential to the proposed settlement method, is defined and methods of determining its value are presented. Making use of the yield strain, loading conditions, and limit pressures of the soil in the dry and wet state methods for determining collapse strain are proposed. Within the collapsible layers zones of collapse that are dependent upon the loading conditions are identified and a method of estimating the height of these zones is proposed. The values determined for collapse strain and the height of the collapse zone are then used to predict the collapse portion of settlement. For the silts tested near Nephi, Utah the proposed method gives results within 5% of the measured average settlements for the loading conditions of overburden alone, existing structures, and structures on wetted collapsible soil. Extrapolation of the data provides a method of determining collapse for embankment loading.

The proposed method for predicting average collapse is applied in an example problem making use of a PMT test pair from a debris earth dam in Colorado.

SETTLEMENTS DUE TO ARID COLLAPSIBLE SOILS

by

HEATHER L. DEVINE

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NOTATION

The following symbols are used in this paper:

A_1 = Janbu's settlement correction factor

A_2 = Janbu's settlement correction factor

B = Foundation width

B_o = PMT reference width of 2 ft. (60 cm)

C = Collapse soil constant

C_1 = Schmertmann's correction factor for foundation embedment depth

C_2 = Schmertmann's correction factor for creep

c = Soil cohesion

E_c = Pseudo-modulus of collapse

E_d = PMT modulus within the zone of influence of the deviatoric tensor

E_D = Modulus of collapsible soil, dry condition

E_{sp} = PMT modulus within the zone of influence of the spherical tensor

E_i = Initial tangent modulus

E_{PMT} = Pressuremeter soil modulus

E_s = Soil modulus

E_w = Modulus of collapsible soil, wet condition

e = Void ratio

e_o = Initial void ratio

G = Shear Modulus

H = Height of soil layer of interest

H_a = Thickness of collapsible layer or the depth of wetting, whichever is less

H_b = Equivalent embankment height

H_c = Height of collapse zone

H_E = Actual embankment height

I_F = Influence factor

I_z = Schmertmann's influence factor, varies with modulus

K_d = Bearing capacity factor, function of foundation width and embedment depth

K_{UR} = Non-dimensional modulus number

N = SPT blow count

P = Pressuremeter pressure

P_l = PMT limit pressure

P_D = Pressure on dry PMT curve at which moisture is added

P_W = Pressure on wet PMT curve occurring at ε_{H_2O} strain level

P_D* = Yield pressure of dry PMT curve

P_W* = Yield pressure of wet PMT curve

P_{lD} = Limit pressure of collapsible soil, dry condition

P_{lW} = Limit pressure of collapsible soil, wet condition

PLR = P_{lW}/ P_{lD} ratio

p_a = Atmospheric pressure

q = net bearing pressure

q_{all} = allowable bearing pressure

q_c = Cone penetrometer tip resistance

q_o = overburden pressure

R = PMT probe radius

R_o = PMT initial probe radius

S1 = Settlement attributed to w_1

S2 = Collapse settlement attributed to w_2

S3 = Settlement attributed to creep

S_{AV} = Average collapse settlement (S2)

s = settlement

V_o = initial PMT probe volume

v_{30} = PMT volume reading at 30 seconds

v_{60} = PMT volume reading at 60 seconds

w_1 = applied stress to dry soil

w_2 = applied stress during moisture infiltration

w_3 = applied stress following collapse

z = depth

α = rheologic modulus correction factor in PMT settlement equations

ε_c = Radial collapse strain

ε_c' = Vertical collapse strain

$\varepsilon_{\mathcal{D}}$ = Strain at yield on dry PMT curve

ε_{H_2O} = Strain occurring on the dry PMT curve at the time moisture is introduced

ε_W = Strain at yield on wet PMT curve

ε_V = Vertical strain

ϕ = soil internal angle of friction

γ = soil unit weight

γ_C = unit weight of collapsible soil

γ_E = unit weight of embankment soil

λ_c = Shape factor for spherical term in PMT settlement equation

λ_d = Shape factor for deviatoric term in PMT settlement equation

μ = Poisson's ratio

μ_c = Collapse soil strain ratio

σ_3 = Minor principal stress

1.0 INTRODUCTION

1.1 Characterization

Collapsible soils undergo significant settlement, or hydroconsolidation, due to a volume reduction after the addition of water and/or additional load. These soils, which may be quite competent in their natural state, frequently lose much of their load carrying capacity and may settle when wetted, oftentimes under overburden pressure alone. The collapse settlements that occur are not classical consolidation type settlements since no water is being forced out, but in fact the soil may be absorbing additional water and progressively losing strength (Dudley, 1970).

These soils usually occur in dry arid regions (see Figure 1.1) and may be aeloian, subaerial, colluvial, mudflow, alluvial, residual, or man-made fills. Due to the diversity of the deposition sources, classification of the adverse soil is oftentimes difficult. However, it appears that at least two factors must be present for a soil to exhibit potential to collapse: A loose soil structure (large void ratio) and a natural moisture content less than saturation. It has been suggested that a collapsible soil may have a bulk density such that upon saturation the moisture content exceeds the liquid limit (Spangler and Handy, 1982).

In order for collapse to occur the soil must have a preconsolidation pressure less than the existing overburden pressure, as determined by the consolidation test. Soil exhibiting this characteristic is either currently undergoing consolidation under its own weight, or it is being held back from consolidation by temporary restraints such as

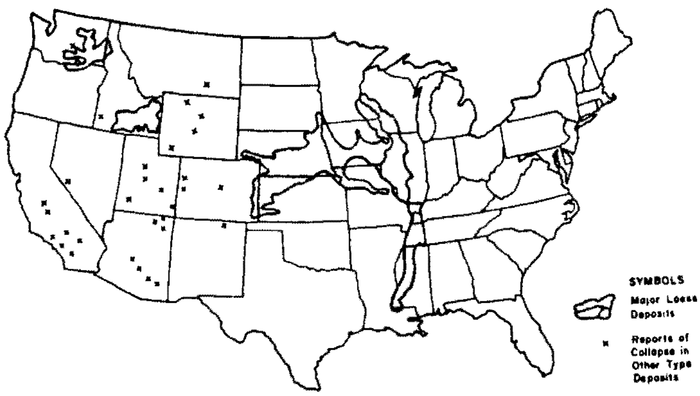

FIG. 1.1.--Collapsible Soils in the U.S. (after Dudley, 1970)

capillarity at intergranular contacts. This apparent anomaly creates a soil structure of a honeycomb nature that lends itself to the collapse phenomenon. The honeycomb structure is most commonly bulky shaped grains that are held in place by some material or force that is susceptible to destabilization by the addition of water (Figure 1.2). When the material or force support is removed the grains are able to slide on one another, moving in the vacant spaces (Dudley, 1970) resulting in significant settlements.

UNSATURATED SOILS

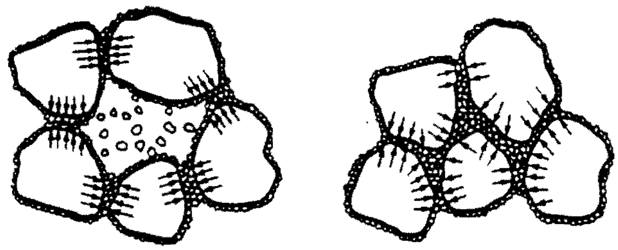

FIG. 1.2.--Collapsible Soil Structure (after Houston, 1993)

1.1.1 Loess

The most widely studied collapsible soils are the collapsible loess. These soils are formed primarily by aeolian processes and are characteristically silts with varying amounts of sand, clay, minerals, and other materials arranged in an open cohesive fabric. Dry densities may range from 70 pcf (11 kN/m^3) to 110 pcf (17.3 kN/m^3). These soils have an open structural arrangement that creates considerable strength in the undisturbed, naturally occurring low moisture content condition. In collapsible loess, the introduction of a sufficient volume of water results in a significant reduction in soil volume and a concurrent loss of strength. Since not all loess is collapsible, lab tests utilizing the consolidometer have been developed but have met with varying degrees of success in determining the collapse potential and field settlements of these soils.

1.1.2 Collapsible Alluvial Deposits

Collapsing alluvial soils are widely distributed over the arid and semi-arid regions of the western United States. As with other collapsible soils, these deposits exhibit considerable strength and stiffness in their dry, natural state, but lose strength and collapse when the become wet.

The formation of these deposits is primarily by alluvial fan, mudflow, and debris flow processes (Hansen et al. 1989) and they may consist of sands and silts with varying amounts of gravel and clay. Both laboratory and field evidence suggests that the fine-grained matrix of these deposits is responsible for the collapse phenomenon and that as little as 5% - 20% passing the #200 sieve may be sufficient to produce collapse behavior (Rollins et al. 1994) and that collapse can occur in soils with as much as 55% gravel. It has also been shown that collapse can occur at moisture contents considerably less than saturation, typically between 60% - 80% of saturation.

Since obtaining undisturbed soil samples of these coarse grained deposits is difficult, if not impossible, in situ testing is gaining popularity as a likely tool to predict the collapse potential and field settlements of these soils.

1.2 Settlement Problems

With the availability of water to support development in arid regions, that were previously areas of considerable desiccation, comes the risk of producing soil collapse. Before the ability to supply water to these regions was economically feasible the incentive to study the collapse phenomenon was minimal. Structures in these areas were usually small and the high strength of the dry soil provided adequate support. However, since the advent of readily available water, development has increased and many of these areas have reported instances of extreme settlements resulting in prohibitive remediation costs.

Identification of collapsible soils prior to construction can lead to their removal or treatment, eliminating a major portion of the potential settlement. While this may be a costly venture, the future savings in remediation costs are far beyond the initial cost of such precautionary methods. Should construction occur on these soils in their untreated state subsequent differential settlements induced by water infiltration can be severe. But there is a significant lack of knowledge and research to develop design methods to accommodate these soils. Only within the last ten years has there been any work appearing that addresses the testing and prediction of collapse settlement (Houston et al.,1988, 1989, 1993, 1995; Rollins and Rogers, 1991, 1994; Rollins et al., 1994; Moumoud et al., 1995; Lutenegger, 1988; and Smith and Rollins, 1997.)

1.2.1 Magnitude of Settlements

Reported settlements due to collapsible soils have been as much as 15 ft (4.6 m) for an irrigation canal in the west central part of the San Joaquin Valley in California (Dudley, 1970), nearly 3 ft. (0.9 m) at a cement plant in central Utah, and 2 ft. (0.6 m) at a man-made reservoir in Nevada (Rollins et al. 1994). Clearly, settlements of this magnitude warrant attention.

1.2.2 Estimate of Financial Repercussions

Information regarding the costs incurred due to collapsible soils is limited. However, with extreme differential settlements remediation may require extensive measures and even abandonment or demolition of structures may be necessary. One example of the cost of remediation is at the cement plant in Utah, which included underpinning by means of piles, hand-dug caissons, and compaction grouting and totaled nearly $20 million. It seems clear the associated costs of similar remediation measures could be enormous. This example illustrates the financial impact that can result from collapsible soils and underscores the value of identification and settlement predictions for these deposits.

1.3 Review of Settlement Methods

The prediction of both settlement magnitude and rate is of prime concern when designing foundations for structures. Methods for these predictions vary widely in their approach as well as the soil properties to be measured. A study of existing settlement methods can provide guidance for studying and predicting the magnitude of collapse in metastable soils. An overview of typical methods now in use and their possible contribution to collapse prediction follows.

1.3.1 Settlement Methods in Sandy Soils:

In free draining sandy soils immediate settlement dominates the total settlement. Primary consolidation is minimal but secondary consolidation, or creep, may be a significant, and is a poorly understood component of settlement in most soils.

When predicting settlement in these deposits, the theory of linear elasticity is frequently used. Making use of the soil modulus, (E_s), Poisson's ratio, (μ), stress increase, (q_o) foundation dimensions (B) and a variety of influence factors, (I_F), using the equation:

$$s = \frac{Bq_o}{E_s}(1-\mu^2)I_F \quad \text{(Timoshenko and Goodier, 1951)} \qquad (1.1)$$

produces an estimate of settlement for a given soil. There appears to be a wide variance of opinion as to what the influence factor should be, but it is essentially a correction factor based on foundation geometry, flexibility of the foundation, and/or embedment depth.

While equally applicable in both cohesive and non-cohesive soils this method is frequently used to predict the expected settlements in sands. However, caution should be exercised in the application of linear elasticity methods, for soils in general are highly non-linear and inelastic over the major range of loading conditions and significant errors can be made with this approach, particularly at loads nearing the yield stress of the soil.

As an alternative to purely elastic methods Schmertmann and Hartman (1978) devised the settlement equation:

$$\boxed{s = C_1 C_2 \Delta q \sum_{0}^{z} \frac{I_z}{E_s} \Delta z} \tag{1.2}$$

where: Δq = the average stress increase in layer Δ z and

z = the depth of influence

for use in sandy soils. The semi-empirical influence factor I_z varies with modulus, and should be obtained for each layer within the 2B influence zone (twice the foundation width). In collapsible soils it would appear that a suitable influence factor could be determined. However, the theory of elasticity does not hold during collapse and the determination of an appropriate soil modulus is difficult. Correction factors C_1 and C_2 are an attempt to address foundation embedment depth and creep effects. The correction factor for creep, C_2, does not make use of any soil properties, it is strictly a function of time. Assuming that no two soils creep alike, plus the complexity of the creep phenomenon itself, this seems an unrealistic and oversimplified approach to try to accommodate the secondary component of consolidation settlement. As many soils, including silty sands, silty gravels, silty/sandy gravels, etc., are of a particulate nature they can oftentimes experience considerable settlement attributable to the plastic adjustment of soil fabrics. This, as well as the issue of time, needs to be considered in the formation of a correction factor.

To accommodate the difficulty, or near impossibility, of sampling sandy soils, correlation's to allowable bearing capacity based on Standard Penetration Test (SPT) blow counts have been developed by Terzaghi and Peck (1967), Meyerhof (1956, 1974) and others. Assuming one inch (2.54 cm) acceptable settlement, the following was presented by Bowles (1988):

$$q_{all} = \frac{N}{F_1} K_d \qquad \text{when } B < F_4 \qquad (1.3)$$

$$q_{all} = \frac{N}{F_2} \left(\frac{B + F_3}{B}\right)^2 K_d \qquad \text{when } B > F_4 \qquad (1.4)$$

N = statistical average blow count from 0.5B - 2B

$$K_d = 1 + 0.33(D/B) \leq 1.33 \qquad (1.5)$$

Table 1.1: Factors for SPT Correlation to Settlement

	SI	**fps**
$\mathbf{F_1}$	0.05	2.5
$\mathbf{F_2}$	0.08	4.0
$\mathbf{F_3}$	0.30	1.0
$\mathbf{F_4}$	1.2	4.0

Bowles (1988) implies that the settlement versus allowable load relationship is a linear one. Thus, from the foundation width and the average blow count over the zone of interest yields the computation for q_{all} for the one inch (2.54 cm) settlement. Application of the ratio $\frac{q_{actual}}{q_{all}}$ gives the predicted settlement. While this is a simple approach, again, caution should be used—soils do not generally behave in a linear fashion, and coupling this with an empirically based correlation could prove to be hazardous, particularly when actual loads exceed allowable loads for settlements greater than one inch. In the case of a collapsible deposit, blow counts could be established for dry conditions and wet conditions. But no real information specific to the collapse phase, when the soil is undergoing changes in properties, can be obtained from the SPT. Additionally, no information regarding foundation size is provided and it may be questioned whether one inch (2.54 cm) settlement for a 12 inch (30.5 cm) foundation is "acceptable settlement" or if it is a bearing capacity failure.

Similar correlation between the Cone Penetrometer Test (CPT) tip capacity, q_c, and allowable bearing capacity for one inch acceptable settlement have been suggested. The merits and limitations of this approach would remain unchanged from those discussed regarding the SPT, with the further complication of no physical sample being a further limitation of the CPT.

In summary, for sandy soils, several methods are available for predicting the immediate component of settlement. These methods, for the most part, prove to be adequate but lack the complexity to properly address the properties of a real soil, in particular the collapsible soils under study. The methods that attempt to address the effects of creep are oversimplified and should only be used as an indication of possible settlements attributable to the phenomenon.

1.3.2 Settlement Methods in Clay:

In cohesive materials the opportunity for an increase in pore water pressure due to low coefficients of permeability is present, shifting a considerable amount of the total settlement to the consolidation phase. During this phase the pore water pressures dissipate over time, resulting in a time dependent settlement. This creates a need to inspect not only magnitude of settlement, but the rate of settlement as well.

In determining settlement magnitude Timoshenko and Goodier's linear elastic relationship is often revisited for wide foundations over thin clay layers. Drained properties (Young's Modulus and Poisson's ratio) are used to calculate the total settlement, while undrained properties are used to determine the immediate portion of settlement. Creep is neglected. To make use of the more easily obtained undrained properties an assumed drained Poisson's ratio of 0.25 is used and the consolidation settlements can be approximated as one and a half times the immediate settlement. Total settlement, therefore, becomes

$$2.5\Delta qB\left(\frac{1-\mu}{E_s}\right)I_F \quad \text{(Atkinson, 1993)} \qquad (1.6)$$

A modification of the elasticity equation was made by Janbu et al. (1956) for specific use with saturated clays. In their equation:

$$s = A_1 A_2 \frac{q_o B}{E_s} \tag{1.7}$$

A_1 and A_2 are functions of geometry and/or embedment depth.

Again, the same caution should be exercised when applying a linear elastic theory to a non-linear inelastic material.

By far the most accepted consolidation settlement method for cohesive materials is based on the one-dimensional consolidometer test (ASTM D-2435). Through incremental loading of an undisturbed soil sample and measurements of displacement, the test data can be manipulated to construct a void ratio vs. log pressure plot. From this plot, strain values and settlements can be obtained for the design increase in stress where:

$$\varepsilon_v = \frac{\Delta e}{1 + e_0} \quad \text{and} \quad s = \varepsilon_v H \tag{1.8}$$

While the laboratory consolidation test gives a stress versus strain relationship that can be used to evaluate strain at a given stress, the theory of linear elasticity is still relied upon to establish the increase in stress level within the influence zone that is used to obtain the associated strain levels. However, this method has been shown to be effective for soils that are relatively easy to sample and it gives a fair prediction of actual consolidation settlements. Further, a determination can be made regarding the rate of settlement, a critical area of interest for consolidation settlement. Although well accepted methods have not yet been developed, it is possible that the consolidation test could also provide information regarding the magnitude and time rate of secondary consolidation, or creep. A modified consolidometer test, or double consolidometer test, for collapsible soils has met with a degree of success when careful sampling techniques can provide an adequate sample. To perform the test a soil sample is tested under natural water content and a second soil sample obtained from the same location is flooded and tested at the same pressure levels. The results are plotted together to give an indication of collapse.

For stable cohesive soils, where laboratory testing is more practical, settlement predictions based on consolidometer results have been successful. This is particularly true when there is a thin compressible layer and plane strain conditions exist, and consolidation induced settlements dominate. However, consolidation tests are time intensive and expensive and are not always an option when budgets are limited, and even minor sample disturbance can influence the results. In addition, it is not an adequate representation of soil behavior under shear, the condition that dominates under small footings or at extended depths in the plane strain condition. Because of these limitations, practitioners may frequently rely on the elasticity methods.

1.3.3 Settlement Predictions in Collapsible Soils Based on Field Load Tests and Pressuremeter Results:

The less widely used field load (plate load) tests and pressuremeter tests are gaining some in popularity, primarily due to the ability to obtain data under field conditions.

The plate load test (ASTM D-1194) is an in situ method of determining vertical modulus and bearing capacity. As in the SPT and CPT it is again assumed that one inch settlements are acceptable. A small steel test plate of diameter 12 in. - 30 in (30.5 cm - 76.2 cm) or 1 ft. - 2 ft. (0.3 m - 0.6 m) square, which can be thought of as a model foundation, is placed at the foundation level and load increments of approximately one fifth the estimated bearing capacity are applied. Settlements are recorded until one inch settlement is reached, or until the capacity of the testing apparatus is exceeded. This load versus displacement data can be manipulated using the area of the plate and the full influence depth of 4B to create a 'stress-strain' curve for the soil. A major drawback to this method is the introduction of size effects, which are magnified when the soil profile is non-homogeneous. The influence zone, which is a function of the foundation geometry, is considerably more shallow for the small steel plate than for an actual foundation, and as foundation sizes increase this size effect becomes more pronounced. Also, with increases in confining pressures with depth the soil generally becomes stiffer and as the foundation width increases there is a non-linear increase in bearing capacity. To move from the one inch (2.54 cm) settlement based bearing capacity information to settlement vs. load information again requires an assumption of linearity, which for soils is not the case.

An effective way to obtain the prediction of settlement from the shear stress-strain relationship for the more general case of a soil is with the pressuremeter. The shear modulus, G, is a direct result of the test and in elasticity E is a function of G:

$$E_{PMT} = 2G(1 + \mu) \tag{1.9}$$

and defines the pressuremeter modulus value. This pressuremeter modulus is typically less than the compressive soil modulus since some stress changes are in tension, and thus a rheologic modulus correction factor α is applied :

$$E_s = \frac{E_{PMT}}{\alpha} \, and \, \frac{1}{3} < \alpha < \frac{1}{2} \tag{1.10}$$

Menard and Rousseau (1962) developed the two part settlement equation that addresses in its two terms the deviatoric (immediate) and the spherical (consolidation) components of settlement, respectively.:

$$s = \frac{2}{9E_d} qB_o(\lambda_d \frac{B}{B_o})^{\alpha} + \frac{\alpha}{E_{sp}} q\lambda_c B \tag{1.11}$$

where: E_d = PMT modulus within the zone of influence of the deviatoric tensor

E_{sp} = PMT modulus within the zone of influence of the spherical tensor

q = net bearing pressure

B_o = PMT reference width of 2 ft. (60 cm)

B = Foundation width

λ_d = Shape factor for deviatoric term in PMT settlement equtaion

λ_c = Shape factor for spherical term in PMT settlement equation

Separate modulus values for near surface layers and deep layers are now possible in this approach, and since the pressuremeter is a purely deviatoric test, it is suggested that its best application are for conditions where there are deep fairly uniform deposits and where deviatoric strains predominate. In cases of wide footings on thin compressible, but stable, layers, spherical strains predominate and consolidation test data is suggested.

1.3.4 Commentary on Settlement Methods:

Of the several settlement methods available to practitioners few address the special difficulties of coarse-grained deposits. Not only is the sampling of such deposits nearly an impossibility, but the mechanics of particulate material is not well understood. There is presently little particulate mechanics theory to draw upon to explain the very complex nature of soils and their behavior, especially for the unstable structures in collapsible soils.

In addition, science and design methodology typically advance by expanding the existing boundaries of well established and accepted theories and practices. When the original scientific basis is sound and universally applicable this approach has served the engineering community well. However, in the case of soil mechanics the foundation on which the basis of the material behavior is defined is rooted in solid mechanics elasticity theory. It was recognized by the pioneering geotechnical engineers that this was, at best, an approximation for particulate substances such as soils, but the theory appeared to adequately describe much of the soil behavior encountered. However, when theory was inadequate, empirical and semi-empirical relationships were developed to define and predict soil behavior. It should be recalled that the initial foundation for soil mechanics was indeed an approximation, and that the scientific tradition of building upon an existing knowledge base in such instances bears scrutiny and may lead to frustration when developing methods to predict complex soil behavior, such as settlement on collapsible soils.

1.4 Settlement Prediction in Collapsible Soils:

Of the settlement procedures discussed above none address the factors and components particular to the hydroconsolidation of collapsible soils. These deposits are unique in their large scale settlements induced by a combination of factors, and a reliable method of predicting the magnitude of such settlements has not yet been perfected. As the development in the western United States proliferates there is a need for evaluating the collapse potential of soils within the dry arid regions. When these metastable deposits are encountered there is a further need to be able to estimate the magnitude of collapse so appropriate recommendations can be provided.

1.4.1 Factors Contributing to the Collapse Phenomenon

There are three primary factors that, in any combination, can contribute to the settlement of collapsible soils: existing overburden, additional load, and additional moisture. To more easily visualize these factors consider the three circle representation of Figure 1.3. The combination of existing overburden and additional load is the condition considered in standard settlement methods and remains unchanged for collapsible soils. However, the remaining three combinations all involve the critical

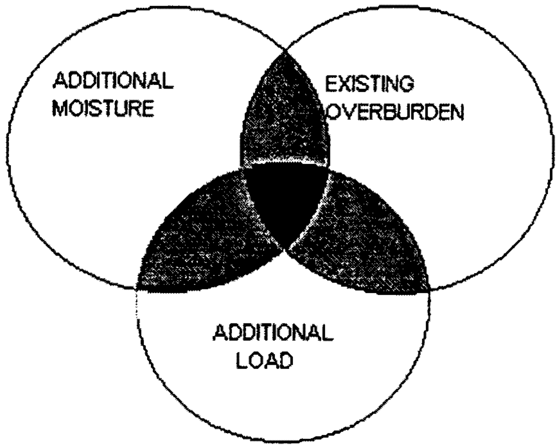

FIG. 1.3. -- Factors Contributing to Collapse

component of the collapse phenomena: additional moisture. It can be seen that additional moisture, in combination with additional load and/or existing overburden can also induce significant settlement. For existing settlement methods for use on stable soils moisture is not a factor in non-collapse settlement and does not enter into the calculations. Therefore, there is an inability to predict the magnitude of collapse settlement with available settlement methods since the single critical variable, moisture, is missing.

1.4.2 Components of Settlement in Collapsible Soils

The settlement that collapsible soils experience can be broken into three parts or components (Figure 1.4).

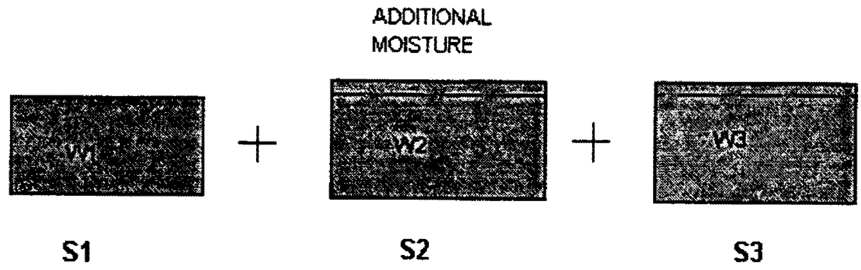

FIG. 1.4. -- Components of Collapse Settlement

First, settlement can occur with the addition of a new load, w_1, quite consistent with settlements occurring in any other stable soil type. Available settlement methods can predict this component of settlement, S1, with equal accuracy (or inaccuracy) to that obtained in stable soils, applied without modification if the pertinent properties can be measured.

With the addition of moisture the second component of collapse settlement, S2, or the true "collapse" may occur. This can be of large magnitude and typically occurs over a relatively short time period. It is in this phase of the settlement when the intergranular bonds weaken or unstable silt buttresses dissolve resulting in large changes in volume. This component of the collapse settlement is not well understood but may likely account for the vast majority of the overall settlement which, hence, is poorly predicted. Factors such as saturation level, moisture composition, and existing loading conditions are all suspected to be elements effecting the magnitude of the collapse. Little research exists to ascertain how critical these factors are. It is during this collapse phase that the realignment of the soil structure can significantly alter the soil properties. This may lead to the necessity of examining whether or not this "collapse" might actually be due to a *bearing capacity failure* of the wetted soil rather than true settlement.

Finally, settlements of the now wetted soil under load can occur. This component, S3, will most likely be secondary consolidation or creep type settlement associated with the new properties of the wetted soil. This has long been recognized as a difficult to predict component of settlement and it is frequently neglected. However, in some soils, particularly those with significant percentages of silt (such as the Willamette Silts), this secondary consolidation may be considerable and care should be exercised when choosing to neglect its contribution to total settlement. If additional stress is applied to the wetted soil, further settlements can be predicted using the wet soil properties and traditional settlement methods.

Some of the above components of collapse settlement are depicted in Figure 1.4 and may occur in isolation or together with other components. w_1, w_2, and w_3 are stresses applied on dry soil, after water has infiltrated, and after collapse respectively. For example, if

$$w_1 = w_2 = 0$$

he condition of collapse under additional moisture and overburden alone prevails. In this case there would be no contribution of S1, and likely very little from S3, towards total settlement. In contrast if

$$w_1 > 0 \text{ and } w_2 > 0$$

there may well be significant settlement from all three components. Table 1.2 summarizes the components of settlement that may be significant under different loading conditions.

Table 1.2: Settlement Components of Selected Loading Conditions

Loading Condition	**S1**	**S2**	**S3**
Overburden $w_1 = w_2 = w_3 = 0$	N/A	X	N/A
Fills $w_1 > 0$ $w_2 > 0$ $w_3 = 0$	N/A	X	N/A
Existing Structures $w_1 > 0$ $w_2 > 0$	N/A	X	X
New Structures $w_1 = 0$ $w_2 > 0$	X	X	X

1.4.3 Current Methods in Use for Collapse Prediction

A modified consolidometer approach to measuring strains and predicting settlements in collapsible soils with significant fines has met with modest success

(Beckwith, 1989). The method has the undisturbed soil sample at the natural water content loaded to the estimated foundation stress. This load is held constant while the sample is flooded and the resulting strain recorded. This gives an indication of collapse potential and a change in strain value that can be used in collapse settlement calculations.

While this method has been somewhat successful, it does not lend itself well to the difficult-to-sample coarse-grained deposits. For these types of soils we typically look to in situ tests to provide necessary data.

Houston et al. (1995) proposed the use of a down-hole collapse test system which is essentially a plate load test coupled with the release of controlled water volumes conducted at the bottom of a borehole. While this procedure has merit, it also has all the same difficulties associated with the traditional plate load test, most notably the influence of size effects. An additional distinct disadvantage is the necessity of specialized testing equipment not readily available. These issues aside, it would appear that some success in predicting collapse magnitude was achieved by the authors. However, this was not a Class A study and certain "correction factors" in the proposed settlement equation were not well explained. It has not yet been shown if the proposed method works well outside the study limits.

In an attempt to gain in situ strain information analogous to that produced in the modified laboratory consolidometer test, a testing procedure involving the pressuremeter (PMT) has been developed (Smith and Rollins, 1997). This procedure requires a simple attachment for the PMT probe which would allow for the introduction of water to the test hole. When the pressuremeter is inflated to a predetermined stress level, a volume of water is introduced and allowed to penetrate the soil surrounding the probe while the pressure is held constant. A measure of the volume expansion is made when the wetting is complete. This change in volume can be reduced to radial strain and stress-strain data for the soil can be obtained. Although the PMT measurements are from shear, the measured change in strain from the PMT test is somewhat similar to the change in void ratio obtained in the one dimensional consolidometer. Utilizing this information to develop a method of settlement prediction is considered in this study.

1.4.4 Case Study

A recent study regarding the treatment methods for collapsible soils was conducted at a site of known collapsible soils near Nephi, Utah (Rollins and Rogers, 1994). Settlement measurements were made that correspond to the components of settlement, S1, S2 and S3, introduced previously. This was done by loading a 4.9 ft. (1.5 m) square foundation to 1775 psf (85 kPa) and measuring the resulting settlement. The area around the footing was then flooded until the soil was wetted to a depth of 11.5 ft. (3.5 m) and the settlement was again recorded. A final measurement of settlement was performed six months after the wetting procedure. This is represented in Figure 1.4 with $w_1 = 1775$ psf (85 kPa), $w_2 = 0$ and measurable settlement for all three settlement components, S1, S2 and S3 was evident.

Considerable testing was also performed at the site to determine soil properties and to capture information regarding the propensity for the soil to collapse. This well documented study is unique and will be referred to frequently as it contains both field measurements of actual settlements and applicable test data, information essential to this study.

2.0 PRESSUREMETER TESTING METHODS IN COLLAPSIBLE SOILS

The pressuremeter test is a down-hole controlled expansion of a cylindrical probe for the measurement of a stress/strain relationship. Of the in situ tests currently available, the pressuremeter test (PMT) appears to hold much promise for obtaining reliable information regarding the hydroconsolidation of collapsible soils. The ability to test the soil undisturbed, under both dry and wet conditions, can help to determine changes in limit pressure and modulus values that occur when moisture contents are increased. If a relationship between these properties and the change in shear strain could be inferred from the resulting dry and wet PMT curves this may provide an important link to better predict the magnitude of collapse that could be expected in these soils.

2.1 Pressuremeter Testing

In 1987 pressuremeter testing was standardized by the American Society for Testing and Materials. The apparatus and procedures, briefly discussed below, are fully described by ASTM D4719-87.

2.1.1 PMT Apparatus

Although other PMT devices and insertion methods exist, the most common type of pressuremeter testing device in the U.S. is the preboring pressuremeter (PBPMT) which consists primarily of four components: the rubber membrane probe, protected by metal strips, the volume/pressure control unit, associated tubing and pressure gauges (Figure 2.1).

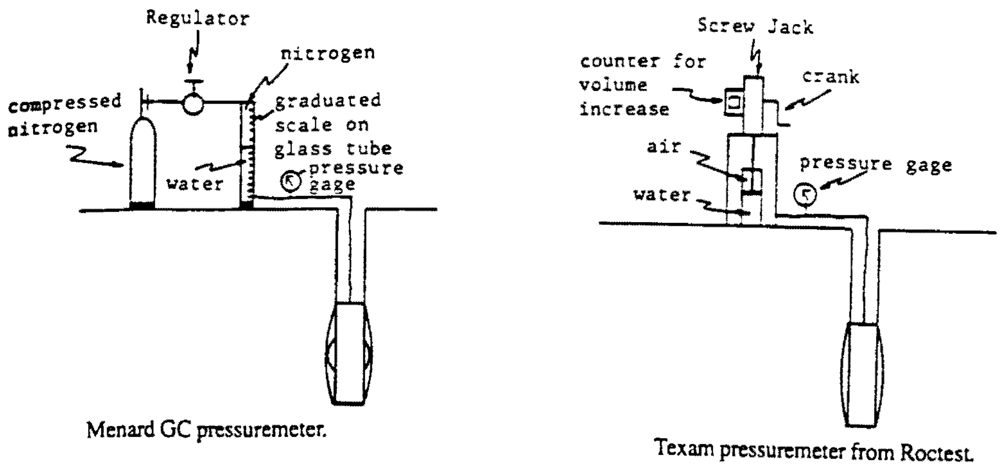

FIG. 2.1.--Schematic of Menard and TEXAM Pressuremeter Devices (after Briaud, 1992)

Probes may vary among different PBPMT devices, but in each case end effects resulting from probe expansion are accommodated by adopting a minimum length to diameter ratio in equipment design. In earlier Menard pneumatic units this is done by including extra "guard cells" at either end of the main expansion cell of the probe. In later hydraulic units end effects are accommodated by increasing the overall probe length such that end effects are negligible and one cell can be used.

The pressure/volume control unit controls the volume of fluid, usually a water antifreeze solution, used for probe expansion. The pressure source may be either bottled nitrogen for pressure controlled units or a screw jack and piston. In hydraulic units volume expansion utilizing bottled nitrogen may be measured electronically by feeler cantilever arms that follow the expansion of the membrane wall. The travel of the piston is used to measure volume changes in the screw jack and piston device for strain controlled units such as the TEXAM (1982). Approximately 164 ft. (50 m) of tubing for the testing apparatus is sufficient for all commercial site investigation work with the PMT.

2.1.2 Existing Testing Procedures

There are two testing methods for performing the PMT suggested in ASTM D4719-87, Method A and Method B. These two methods can be used as the basis of PMT testing of collapsible soils, and can be used either exclusive of one another, or in combination to determine collapse strain.

Method A is a stress controlled test and requires the probe to be inflated in 7 - 14 equal pressure increments to failure. The pressure increment is established as one-tenth of the anticipated limit pressure. Volume readings are taken at 30 seconds, v_{30}, and 60 seconds, v_{60}, for each pressure increment and a plot of pressure vs. v_{60} corrected to radial strain is constructed. The difference, v_{60} - v_{30}, is a measure of creep as a function of pressure (Figure 2.2).

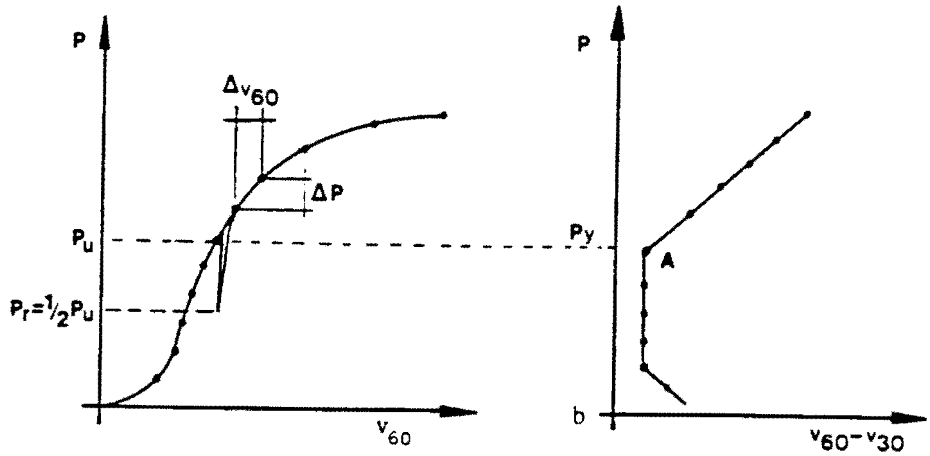

FIG. 2.2 -- Pressure Controlled PMT Procedure (after Briaud, 1992)

In the U.S. the strain controlled hydraulic PMT applies Method B and is the most frequently applied method, primarily to remain consistent with the majority of laboratory testing done under strain controlled conditions. With this method the probe, of initial volume V_o, is expanded in 40 equal volume increments of $V_o/40$, to twice its initial volume. Pressure readings are taken at equal time intervals and a pressure vs. radial strain plot is constructed (Figure 2.3). Available computer software corrects the readings for equipment calibration and readily converts volume increases to radial strain.

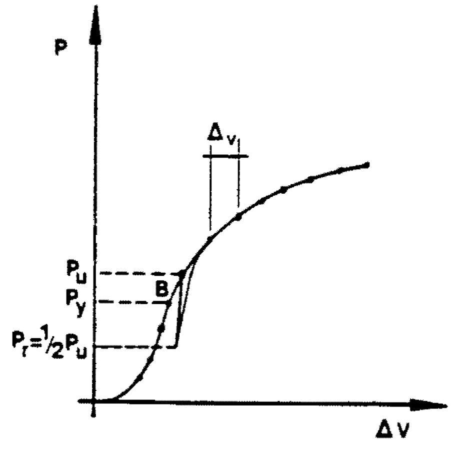

FIG. 2.3 -- Volume Controlled PMT Procedure (after Briaud, 1992)

Many soils possess the time dependent properties of consolidation and creep that are a function of stress. To reduce the impact of these properties on strain controlled PMT results pressure readings are made for each strain level at consistent time intervals. A curve resulting from 15 second pressure readings may differ considerably from a curve plotted from 20 minute pressure readings that may be considerably reduced as consolidation and creep occur, validating the need for consistent time readings.

However, by using a special testing procedure that makes use of the pressure change, the creep, relaxation and consolidation properties of the soil can be studied. It is recommended for the hydraulic units that at a predetermined pressure during the standard test, the pressure be held constant while readings of volume increase vs. time are made (Figure 2.4). Several such pressure steps at varying levels can be performed to evaluate

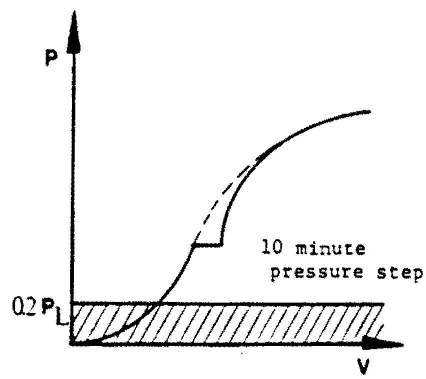

FIG. 2.4. -- Creep Consolidation Test (after Briaud, 1992)

the time dependent properties of the soil as a function of stress. Relaxation properties of the soil can be studied by similarly holding the volume of the probe constant and taking the decaying pressure readings over time (Briaud, 1992).

In stable soils this time dependent soil strain can be attributable to either consolidation of the soil or creep, or a combination of the two. Plotting the radial strain against the logarithm of time can provide some insight into the cause of the soil strain properties has been developed by Smith and Rollins (1997). It is recommended that volume expansion measurements, as opposed to electronic radial point expansion measurements be used to accommodate the non-homogeneity of most collapsible soils. The suggested procedures are based on test results obtained with the TEXAM (Roctest 1983) design PMT in dry test holes prepared with air circulation. Three separate test procedures as outlined by Smith and Rollins are as follows:

2.2.1 Procedure 1: Dry

Following dry hole preparation inflate the probe with either Method A or B of ASTM D419-87. Reduce the data to allow for normal membrane resistance, volume loss and hydrostatic head in hydraulic equipment. Report the dry modulus and limit pressure, E_D and P_{lD}, respectively. This test should be required for all possible collapse sites as a reference test.

2.2.2 Procedure 2: Wet

Following the same dry hole preparation as for Procedure 1, place the probe in position at the bottom of the dry borehole and inflate to a seating pressure of $P_{lD}/20$. Rapidly pour 8 gallons (30 liters) of water down the open support rod internal annulus into the probe and take volume readings every 5 minutes for pressure holding period of 30 minutes. Probe expansion then recommences after this period and is continued up to the limit pressure, either by that the soil experiences, but unless the resulting plot is linear (indicating the presence of creep only) the relative amounts of consolidation and creep remains unknown. In collapsible, or metastable soils, there is the additional uncertainty in defining soil behavior during the collapse phase.

2.2 Proposed PMT Methods for Use in Collapsible Soils

Collapsible soil behavior differs significantly from stable soils for which the current PMT testing procedures were developed. In stable soils a modulus value can be determined through PMT data analysis that can be used to adequately predict settlement behavior, particularly in deep fairly uniform deposits where deviatoric strains predominate. Methods for settlement prediction are rooted in the solid mechanics based theory of elasticity. While this works reasonably well for stable fine grained cohesive soils, significant difficulties arise in its application to metastable coarse grained deposits. During collapse when large strains occur under constant load, a modulus value ceases to have meaning and solid mechanics theory does a poor job of explaining the real behavior of the soil. Also, dilatancy and contractive effects of coarse grained deposits are not adequately addressed through elasticity. However, alternative theoretically sound methods of settlement prediction in metastable soils are as yet undeveloped, so it remains necessary to rely on the existing theory of elasticity, acknowledging its deficiencies, in the study of settlement prediction in collapsible soils.

Recognizing the versatility of the PMT, with both strain controlled and stress controlled expansion possible, application for its standard use in collapsible soil

Method A or B of ASTM D419-87. Reduce the data and report the wet modulus and wet limit pressure, E_w and P_{lw},, respectively.

2.2.3 Procedure 3: Dry-Wet

Borehole preparation is identical to Procedures 1 and 2 and the probe placed in position in the borehole. Inflation takes place under ASTM D419-87, either Method A or B, to a pre-determined stress level. This should not exceed the yield pressure (end of linear range) measured under procedure 1. Then follow the recommendations given in Procedure 2 using 8 gallons (30 liters) of water and the 30 minute holding period. Data reduction should identify three modulus values: dry, wet, and collapse, (E_D, E_w, and E_C respectively) which are defined in Figure 2.5, after Smith and Rollins (1997).

Recognizing that some routine pressuremeter testing includes unload-reload cycles, these may be added to any of the procedures described above. However, these unload-reload cycles should only be conducted *after* any water addition in Procedures 2 and 3 (Figure 2.5).

After completion of the test and withdrawal of the pressuremeter probe drilling with air can recommence. Any procedure can be adopted in the same hole after a suitable minimum distance is obtained below the previous test position. For commercial NX size probes 4.6 ft. (1.4 m) is sufficient, and this should prevent moisture contamination from the 8 gallons (30 liters) of water used in the previous test if Procedure 2 or Procedure 3 was used.

FIG. 2.5. -- PMT Modulus Values (after Smith, 1997)

The modulus values, E_D, E_W, and E_C, obtained from the procedures outlined above can be made use of in a method of predicting collapse potential that will be discussed in Chapter 4. E_D may be used in settlement calculations that occur under dry conditions, and E_W may provide information on post-collapse settlements that occur in wetted soils. Further discussion of the application of these values in settlement calculations is presented in Chapter 5.

3.0 NUMERICAL FINITE ELEMENT MODELING OF PMT

Soil mechanics problems have only recently begun to be treated numerically because of the complexity and difficulty in devising realistic constitutive models and generating an adequate understanding of the subsurface conditions on which to apply the soil models. However, with the progress of ongoing soil mechanics research and developments in the computer industry, the numerical treatment of soil mechanics problems is becoming a more popular analytical technique. It remains important to recognize the difficulties associated with each step that is taken beyond the current level of computing technology and it is essential to acknowledge the limitations.

In collapsible soil applications it is difficult to devise an appropriate constitutive model that recognizes the zero, or near zero, modulus of the soil in tension, the collapse phenomena, the anisotropy and the change in soil properties that result from the addition of water. Additionally, the frictional behavior of coarse-grained materials and dilatancy effects are poorly understood and constitutive models that effectively represent this behavior have yet to be developed. However, by making use of current soil models and features specific to geotechnical computer codes, coupled with collapsible soil field data, some information regarding the basic behavior of metastable soils can be gained.

3.1 Introduction to CAMFE and FENAIL Computer Codes

Two separate finite element computer codes were used in this study to help analyze PMT field results obtained from tests conducted near Nephi, Utah (Rollins and Rogers, 1994), an area of known collapsible sandy silts and silty sands.

CAMFE (Carter, 1978) is a computer program, written in FORTRAN, for the analysis of a cylindrical cavity expansion in soil. It was developed at Cambridge University (CAMbridge 1-D Finite Element program) to predict the behavior of driven piles in clay. This program, however, also quite readily models the cylindrical expansion of the pressuremeter probe. The program performs the necessary computations for the cylindrical cavity expansion in a two-phase soil, modeling the strength of the soil before and after pore water pressures dissipate. The expansion of the cavity is modeled one dimensionally as a point on the radius of the cavity wall. The soil elements are modeled as a series of nodes extending beyond the cavity wall. There are three different soil models within the program available to the user: isotropic, linear elastic; Mohr Coulomb (Tresca when phi=0); and modified Cam-clay. When specifying isotropic, linear elastic material types only the drained Young's modulus and the drained Poisson's ratio, along with the unit weight of the pore fluid and soil permeability need to be declared. Additionally, for the Mohr-Coulomb criteria, drained cohesion and friction angle values need to be specified. If modeling the modified Cam-clay soil type, a volumetric hardening, elasto-plastic material based on critical state concepts, seven parameters are required, all of which are defined in the CAMFE manual. The one-dimensional finite element calculations are performed within one main routine and thirteen sub-routines. Although there are no iterations, the equations governing each time step are non-linear. Hence, load and time steps should be kept small enough so that the initial values can be considered appropriate for the entire step. This is in keeping with the "tangent stiffness" method. Inner and outer cylindrical surfaces static and hydraulic boundary conditions along with initial stress conditions must be specified in the input file. At each boundary a known traction or displacement condition as well as either a fixed pore water pressure or impermeable interface must be declared. The capabilities of the program, its verification and the accuracy obtainable have been reported in some detail by Carter (1978).

A more vigorous analysis of collapsible soils was performed with the aid of a second finite element computer program, FENAIL (Finite Element analysis of soil NAILS, Smith, 1993), which was derived from a finite element code called Soil Analysis Code (SAC) written at the University of California, Davis (Hermann and Kaliakin, 1986) for soil consolidation analysis. The FENAIL program can be used to perform two-dimensional static and quasi-static stress analysis of earth structures, including excavation and construction effects. This excavation and construction feature also allows for a change of material type to occur during the execution of the program. Constitutive soil models available to the user include linear-elastic no tension, anisotropic elastic, and a hyperbolic plasticity model which makes use of material cohesion and internal friction behavior. An acceptable tolerance value and a maximum number of iterations allowed per step are stipulated in the input file, for the iterative process in non-linear solutions. FENAIL is written in FORTRAN 77 and input files are free-form. Output is provided in standard finite element line printer element and nodal output files and also in individual load/time step files for post processing via the proprietary graphics post processing package, TECPLOT (Amtec Engineering, 1988-1992).

3.2 Limitations of Numerical Finite Element Codes

Both CAMFE and FENAIL can be used to adequately model the expansion of the PMT, but each has particular strengths and weaknesses. CAMFE, which is a one-dimensional representation, is by far the easier program to execute. The "mesh" is limited to a string of adjacent nodes horizontal to the cavity wall. In the case of the PMT, information regarding a single point along the borehole cavity and the soil conditions at that depth are all that is modeled. When restricted to one dimension and run axisymetrically, the complete geometry of the actual problem cannot be adequately modeled. Since length and/or depth is not modeled no information regarding the influence of soil above and below, or end effects of the probe can be obtained. What is essentially modeled is an infinite horizontal layer of soil with uniform conditions along an infinitely long cavity wall. Also, the pressuremeter test measures changes in the volume of the probe, which can be reduced to changes in radius only under uniform expansion conditions. When probe expansion occurs in non-homogeneous coarse-grained deposits, primarily gravels and cobbles, the radial expansion along the length of the probe may be non-uniform, and average radial expansion may be the best approximation for computer applications. However, if a careful selection of a representative point is made and the possibility of non-homogeneous soil conditions is accounted for, useful information of stress and strain conditions can still be obtained.

The ability to model a more complete representation of the PMT is possible by utilizing the more complex two-dimensional finite element code, FENAIL. The free-form input file is substantial, but with close attention to detail and available manuals and sample files, an adequate representation of existing conditions can be created. By establishing the dimensions of a mesh such that it captures the zones of influence, and setting appropriate boundary conditions, the user is able to study the effects of a myriad of conditions. When modeling the PMT, the axisymmetric option, as opposed to plane strain, is selected. This allows for the representation of the real three-dimensional problem in two-dimensional space, a significant advantage over the one-dimensional mode. However, this, too, is limited to the condition of homogeneous layering of soils about the borehole wall. This assumption is considered valid in most cases involving the modeling of the PMT, which has a radius, R, of 1.45 in. (3.7 cm) and a horizontal influence zone believed to be less than or equal to 10R.

A significant limitation of finite element programs, in general, that are used for geotechnical applications is the difficulty in modeling frictional materials. This is a direct result of the current lack of understanding of particulate mechanics and the weakness of elasticity to capture dilatancy and contractive effects. Without a fundamental basis in the theory of frictional behavior, any attempt to model a coarse grained soil is speculative at best. Being aware that the constitutive soil models make use of piece-wise elasticity, and the collapse phenomenon is poorly defined by elasticity theory, all results based on such models should be interpreted in a manner that acknowledges the deficiencies of the soil model.

3.3 Overview of Nephi, Utah Field Testing

Damage of structures from collapsing alluvial debris fan soils can be substantial and the extent of cumulative repair costs has prompted a study by Rollins and Rogers (1994) to evaluate the cost and effectiveness of various mitigation measures. The study included comprehensive field testing to determine soil properties and settlement behavior under various treatment procedures. Instrumentation of the influence zone below six full-scale footings provided information on measured settlements, both pre-treatment and post-treatment.

The test site was located in Nephi, Utah on the lower end of an alluvial fan where annual rainfall is less than 14 in./yr. (350 mm/yr.). Houses, roadways, and commercial buildings have experienced significant settlement as a result of accidental wetting the collapsible soils. Prior to any treatment the subsurface profile to a depth of 16.4 ft. (5 m) was defined at each of the six test cells within the site. The soil profile was relatively consistent across the site and was primarily a clayey sandy silt, classifying as a CL-ML, or ML type material according to the Unified Classification System. Typical soil gradation was 30% sand, 60% silt and 10% clay.

Pressuremeter tests were performed using a TEXAM pressuremeter with a BX probe in holes that were prepared with a rotary drill rig using compressed air to remove the cuttings. All three PMT testing procedures recommended by Smith and Rollins (1997) were performed at the site.

Each test cell consisted of a 13 ft. x 13 ft. (4 m x 4 m) area. Full-scale footings, 4.9 ft. (1.5 m) square and placed at the surface, and bearing pressures of 1775 psf (85 kPa) were used in the study. To determine the distribution of settlements below the footings settlement monitors were located a depths of 3.3 ft. (1 m), 6.6 ft. (2 m) and 9.8 ft. (3 m) below foundation level at the corners of each footing. These monitors consisted of auger sections with a riser pipe that was isolated from the surrounding ground. Settlement of the footings and monitors under a 1775 psf (85 kPa) load was measured with a survey level prior to wetting. Small dikes were than built around the perimeter of the test cell and the area within the cell flooded until the moisture front penetrated to an approximate depth of three meters. Settlement measurements were made as the front advanced and again after six months to evaluate creep. A typical set of results is given in Figure 3.1.

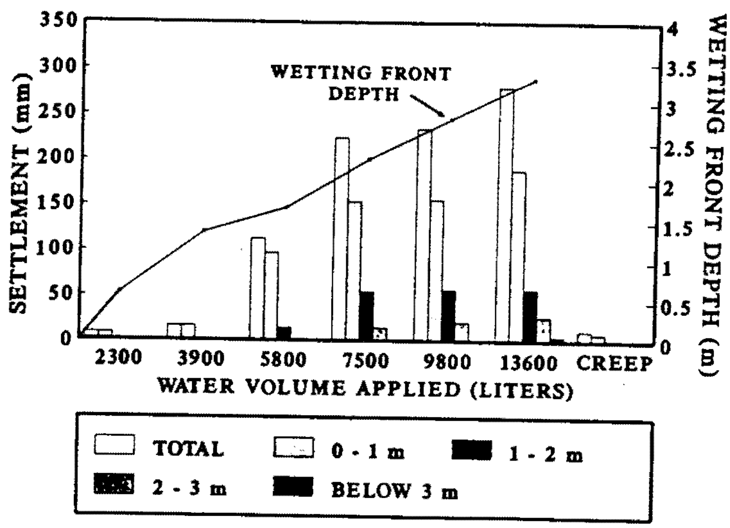

FIG. 3.1 - Summary of Typical Settlement Monitor Data (after Rollins and Rogers, 1994)

Of particular interest to this research is the reference test, or no treatment test cell, and the water treatment cell performed at the Utah site. The settlements under load measured on untreated soil before the wetting procedure, immediately following the wetting procedure, and six months later will be used in the development of a settlement estimation procedure for these conditions addressed in Chapter 5. The measurements made at the water treated cell will be used for the development of an estimation procedure for collapse under overburden and collapse for new construction on wetted collapsible soils.

3.4 Strategy for Matching Field Test Results at Nephi

In the development of a settlement method for collapsible soils it is fundamental to establish the basic behavior of the soil, whether it behaves in a cohesive or non-cohesive manner. In the case of a stable, saturated cohesive silt subjected to additional load excess pore water pressures develop and, as these pore water pressures dissipate, consolidation of the soil mass within the influence zone occurs. This consolidation process can be approximated in the laboratory consolidation test and field settlements can be predicted with reasonable reliability. In coarse-grained, non-cohesive soils that readily drain, excess pore water pressures do not develop and elasticity methods can be used to predict settlements. In partially saturated collapsible silts it is uncertain whether the cohesion of the silt will allow for the development of pore water pressures, or if the absorption of water into the soil fabric will result in no excess pore water pressures indicating frictional behavior. Determining whether the basic behavior is cohesive or frictional can help in the development of collapsible soil settlement predictions.

To establish whether the collapsible soils encountered at the Nephi, Utah site were behaving cohesive or non-cohesive a non-dimensional plot, P/P_l vs R/R_o, of the PMT test results was constructed (Figures 3.2 and 3.3) and a FEM match was attempted. Initial

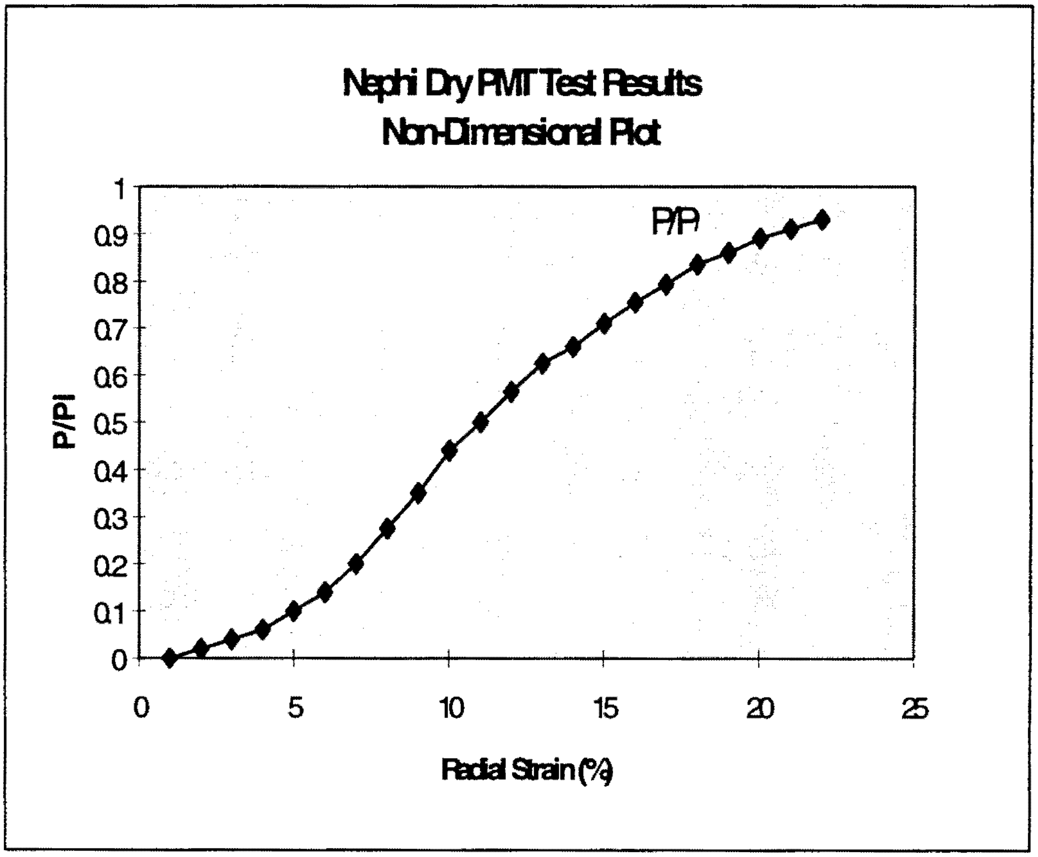

FIG. 3.2. -- Non-dimensional PMT Plot From Nephi, Utah: Dry Condition

execution of both CAMFE and FENAIL made use of the linear elastic solution using PMT modulus values and an assumed Poisson's ratio of 0.4. The FEM output was compared to the closed form solution for linear elasticity to verify the accuracy of the model. Table 3.1 reports, at small strains (less than 10%), FEM analyses predicted stresses at the midpoint of the probe along the borehole wall that were consistent with the closed form solution calculations.

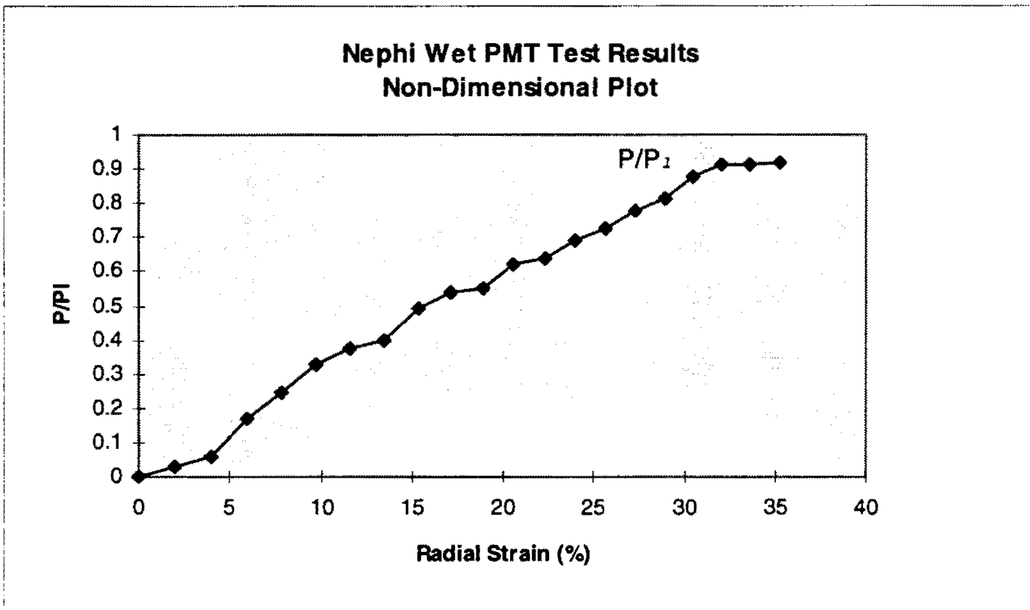

FIG. 3.3. -- Non-dimensional PMT Plot From Nephi, Utah: Wet Condition

Table 3.1: Stress Along Borehole Wall at 6.67% Radial Strain

Closed Form (psi)	**FENAIL (% Error)** (psi)	**CAMFE (% Error)** (psi)
34.8	36.9 (6%)	32.2 (7%)
Note: 1 psi = 6.895 kPa		

The CAMFE mesh was based on an increasing element ratio of 1.025. The two dimensional FEM mesh used to model the PMT test in FENAIL was composed of 96 quadrilateral elements and 119 nodes. Figure 3.4 is a schematic of the FENAIL mesh used throughout this analysis.

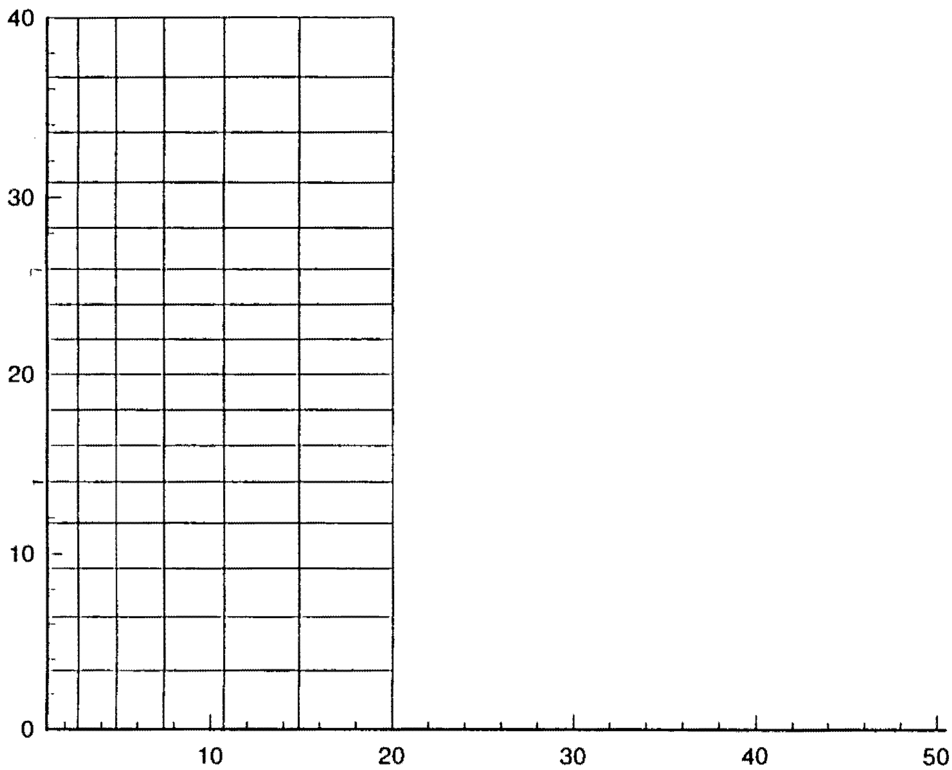

FIG. 3.4. -- FENAIL FEM Mesh

3.4.1 Use of Field Limit Pressures and Modulus Values in the Model

To evaluate whether the soil behavior could be determined as being primarily cohesive in nature or granular, under both dry and wet states, the two dimensional FENAIL code was executed using varying soil properties. This required the input for a full nonlinear modeling of field limit pressures and the dimensionless modulus number, K_{ur} values, calculated from the field test results. Assumptions were made regarding Poisson's ratio and n, the exponent determining the rate of variation of $\frac{E_i}{\sigma_3}$, where E_i is the initial tangent modulus and σ_3 is the minor principle stress, that reflected the silty sand/sandy silt soil type at the site (Duncan and Chang, 1970). Modification to c and ϕ values were made until a non-dimensional plot of the FEM solution closely matched the non-dimensional plot of the field results. Plotting the results in a non-dimensional form emphasized the shape of the curve for comparison purposes.

This procedure was done for both the dry and wet test conditions. For the dry conditions a uniform soil model was used throughout the mesh. To model the wet test the mesh elements within a 10R distance from the borehole wall were modeled using the wet PMT soil properties and varying the associated c and ϕ values. The soil properties for the elements outside this zone remained the same as those determined from the final (best match) dry test simulation.

3.4.2 Discussion of Match

The intent of matching the curve was to determine cohesion and internal friction angle values of the collapsible soils by utilizing PMT data and generating a FEM curve to match the field generated curve.

Under dry conditions the non-dimensional PMT field curve had a gentle slope, characteristic of a frictional material (Figure 3.5). The finite element analyses with varying c and ϕ values substantiated this assumption, with the final "best match" being a purely frictional material with an internal friction angle of 42.3° (Figure 3.6).

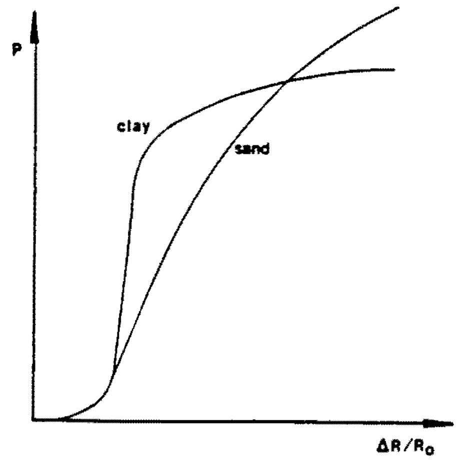

FIG. 3.5. -- Characteristic PMT Curves (after Briaud, 1992)

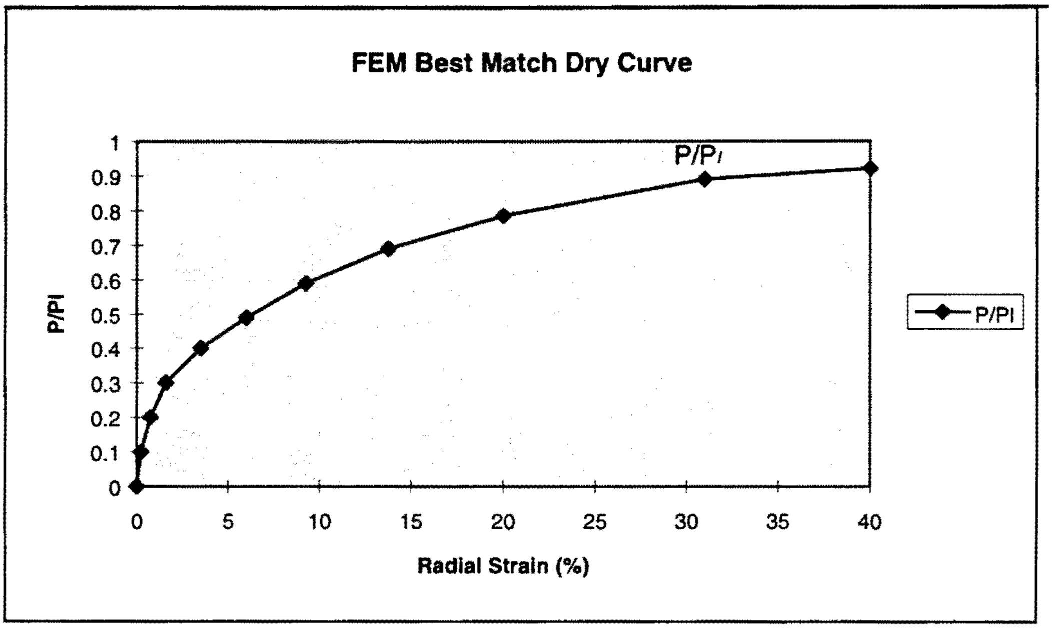

FIG. 3.6 -- FEM Best Match: Dry Curve

Because of the frictional nature of the soil, confining pressures in the model were, by necessity, inconsistent with field conditions, however, limit pressures and non-dimensional modulus number, K_{ur} values, were based on field results. PMT tests were conducted at a depth of 3.67 ft. (1.1 m) and FEM confining pressures necessary for execution of the code reflect a depth of 13 ft. (4 m). The limit pressure of 140 psi (965 kPa) was a result of the field pressuremeter test and K_{ur} , based on n=0.6, $\alpha = \frac{1}{25}$, atmospheric pressure p_a = 14.7 psi (101 kPa)and γ = 90 pcf (14.1 kPa), was calculated from the initial PMT modulus value as

$$K_{ur} = \frac{E_i}{[p_a(\frac{\sigma_3}{p_a})^n]} = \frac{2015}{14.7(\frac{2.29}{14.7})^{0.6}} = 418 \qquad (3.1)$$

rounded up to 500. This is consistent with values for frictional materials reported by Duncan, et.al. 1980. The FEM "best match" plot did not capture borehole effects, but for strains outside this region, up to 15% strain on the FEM output, a reasonable match was achieved. Figure 3.7 shows the field results corrected for borehole effects plotted with the FEM results. The model is particularly sensitive to changes in Poisson's ratio, υ, as well as c and ϕ. The Poisson's ratio used in this analysis was 0.4969. Other values

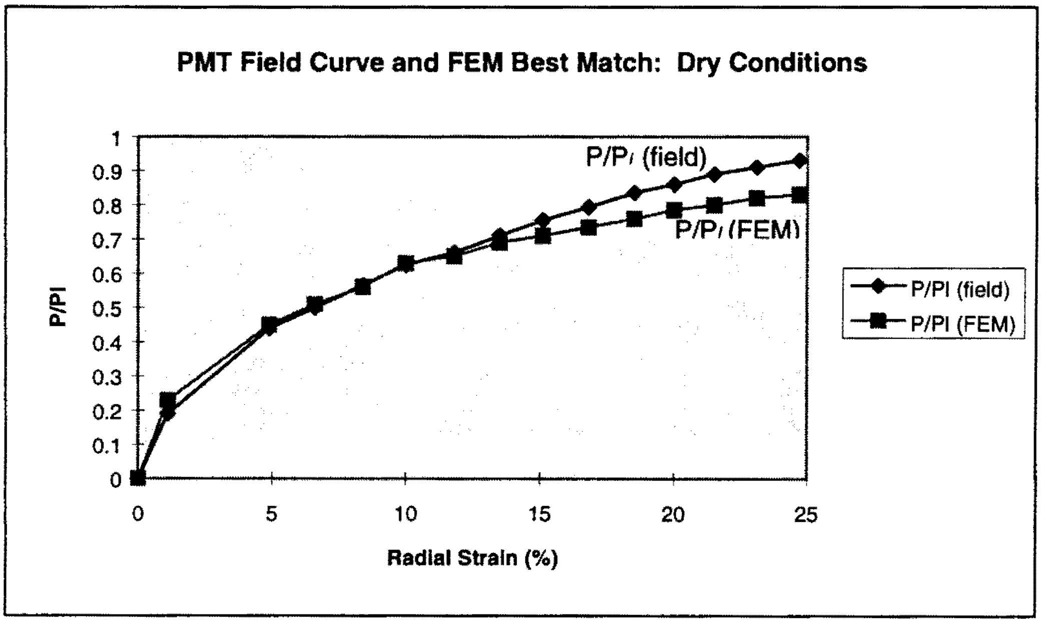

FIG. 3.7. -- PMT Field Curve and FEM Best Match: Dry Conditions

resulted in a significant change in friction angle, however, regardless of the combination of υ and ϕ obtained, any contribution of cohesion to the model greatly increased the initial slope and rendered the result a bad match. The FEM analysis, the non-dimensional plot of field data, and the field based K_{ur} value all substantiate that the collapsible soil in the dry state is behaving as a *frictional* material.

A similar approach was taken to model the wet conditions. The non-dimensional plot of the Nephi measured field results exhibited an interesting feature, undetected in the direct pressure vs. radial strain plot. In the non-dimensional graphic (Figure 3.3) there appears to be an initial modulus, followed by an "intermediate limit pressure" where upon the initial modulus is reestablished followed by a subsequent "intermediate limit pressure". This pattern continues until the overall limit pressure is reached . To confirm that this was not an isolated condition, or an erroneous test result, two other non-dimensional plots of PMT results in wetted collapsible soils were made. Figures 3.8 and 3.9 indicate this

feature is present at a second test site at Nephi and at a test location in Provo, Utah.

The presence of this feature made creating a good match with FEM analysis beyond the ability of the code, so a match of the initial slope only of the wet Nephi field plot corrected for borehole effects was attempted. Again, the limit pressure of 25 psi (172 kPa) was a direct result of the pressuremeter and K_{ur} was calculated from the initial PMT modulus value as K_{ur} = 72.6. The calculated K_{ur} compared well with values given

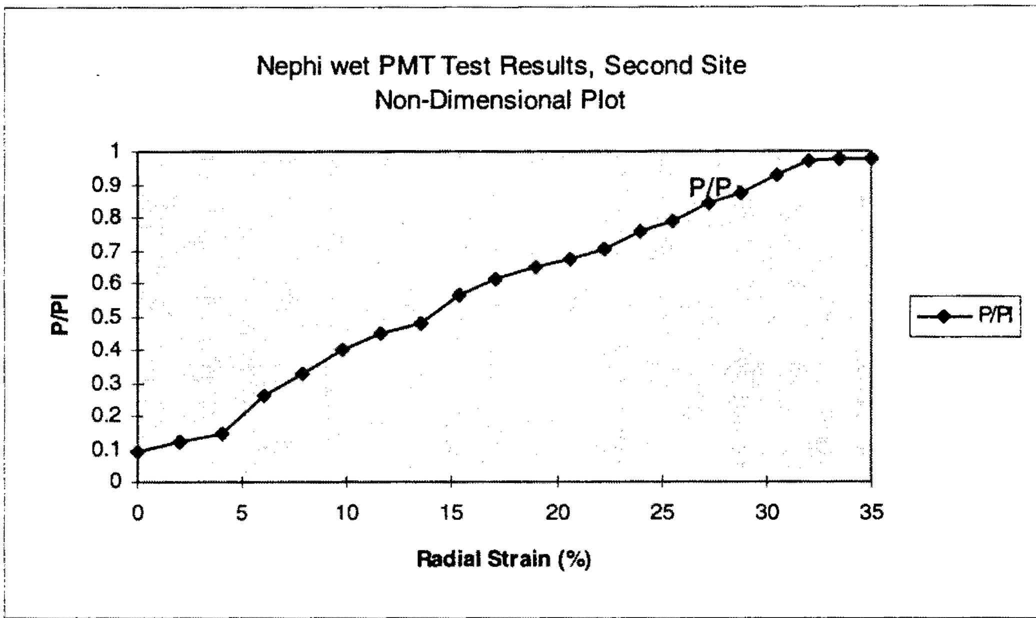

FIG. 3.8 -- Non-dimensional Plot From Second Nephi Site: Wet Condition

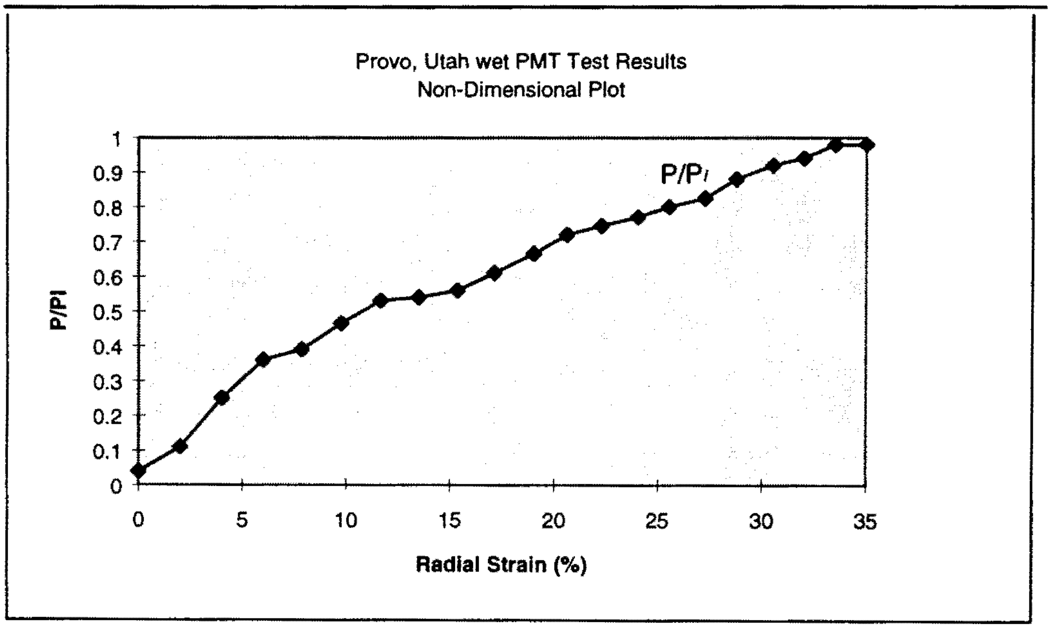

FIG. 3.9. -- Non-dimensional Plot From Provo, Utah site: Wet Condition

by Duncan for Sandy silty clay, so a component of cohesion was entered into the FEM model. The general shape of the wet field curve was somewhat steeper that the dry curve and possessed a distinct final limit pressure, also indicating the presence of some cohesion (Figure 3.5). Non-dimensionally the wet curve was similar to the dry curve beyond 15% strain.

Varying c and ϕ in the wetted zone, an area modeled in the FEM analysis as approximately ten times the pressuremeter radius (10R), and keeping the remaining area of the mesh under dry state conditions, a best match was produced at an assumed Poisson's ratio of 0.46, cohesion of 3.7 psf (0.6 kPa) and an internal angle of friction of 40° (Figure 3.10). Again, sensitivity to Poisson's ratio, an assumed value, effects the resulting c and ϕ values, but it remains that the material in its wet condition is primarily *frictional* with now a minor contribution from cohesion. Using c and ϕ values from the "best match" FENAIL analysis, a one-dimensional analysis was performed using CAMFE (Figure 3.11). The non-dimensional plot of this execution resulted in a more gradual slope, and it also predicted that the limit pressure of 25 psi (172 kPa) occurred at approximately the same strain level predicted by the two-dimensional code and the actual strain measured in the field. This could suggest that the one-dimensional model may be sufficient for limit pressure applications.

With dry and wet properties established the construction of a model of the dry/ wet test (procedure 3) using the time history function and the excavation/construction feature of the FENAIL code was attempted. However, while this technique is possible, it is not sufficiently developed and the soil models are not advanced enough to adequately represent the frictional behavior of the soil in a quantitative sense.

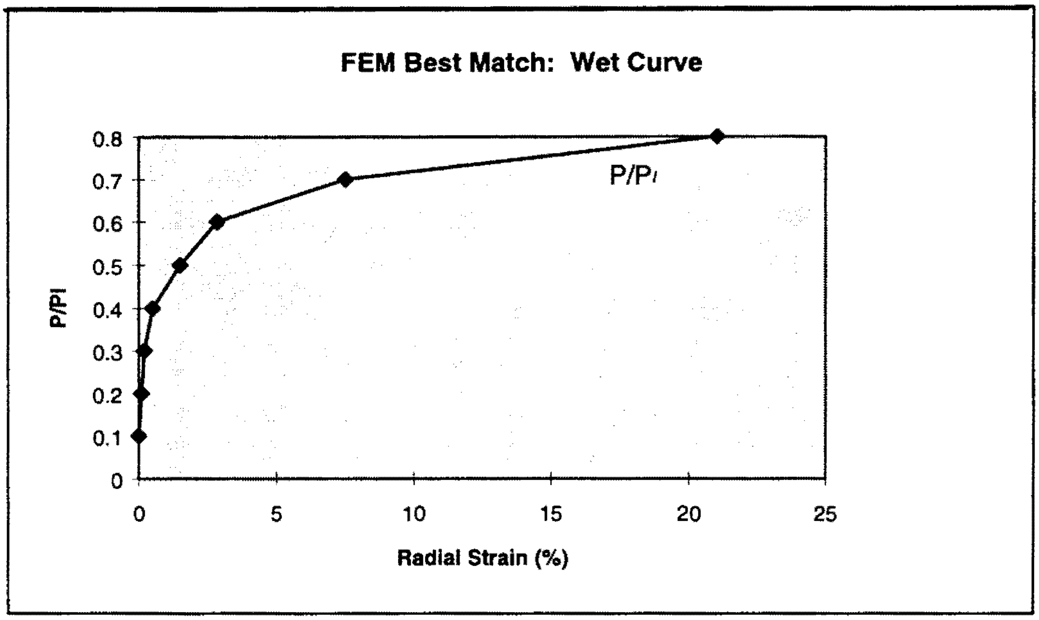

FIG. 3.10. -- FEM Best Match: Wet Curve

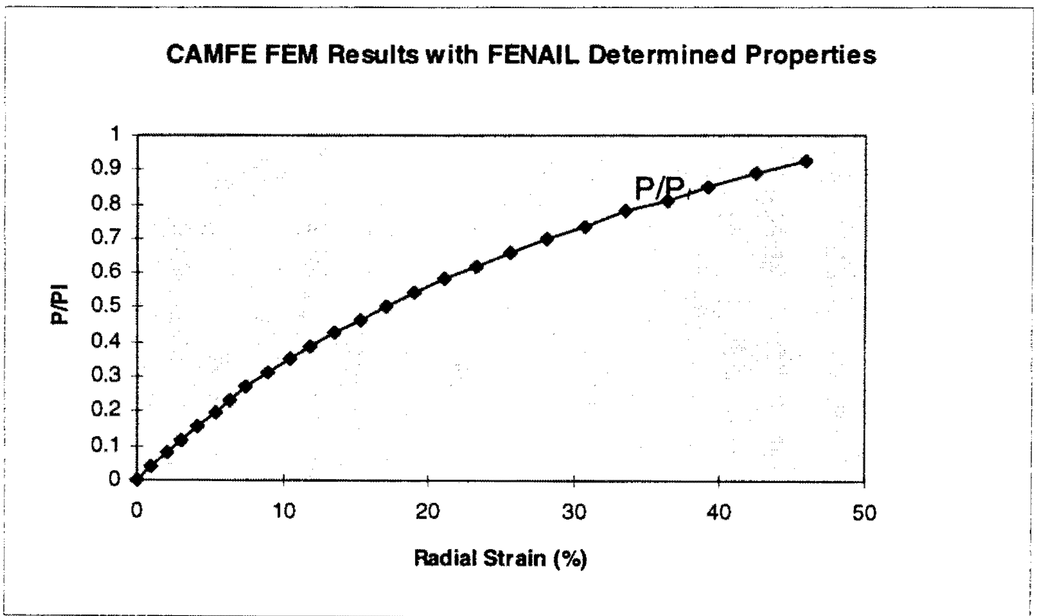

FIG 3.11. -- CAMFE FEM Results with FENAIL Determined Soil Properties

3.5 Effects of Cohesion and Internal Friction Angle

By executing the FENAIL FEM repeatedly with varying cohesion values from 0 psi (0 kPa) - 25 psi (172 kPa) and internal friction angle values from 0° - 45° it became apparent that the FENAIL code works most effectively for cohesive soil models under small strain. With limit pressure as input and working towards that pressure occurring at approximately 30% - 40% strain (consistent with field results) a match was achieved primarily by varying Poisson's ratio and the angle of internal friction value. The slope of the resulting curve was controlled by changes in cohesion. However, an increase in c to increase slope necessitated a drop in ϕ and, likewise, an increase in ϕ to decrease strain values at the limit pressure required a concurrent drop in c, with Poisson's ratio remaining constant. This iterative process was continued until satisfactory results were obtained.

3.6 Determining the Zone of Influence

To verify that a distance of ten times the probe radius from the borehole wall fully encompassed the zone of influence around the probe two approaches were taken; one making use of the CAMFE code and the other, more extensive approach, using the FENAIL code.

The CAMFE analysis involved setting up two separate meshes, one of very large proportions (11,280R) and one fixed at 10R. Each file was executed at a set strain value and all soil properties were consistent in the two input files. The resulting horizontal stress at the borehole wall for the 10R mesh was within 1.7% of the stress value at the same location of the large mesh, which was deemed acceptable.

The FENAIL code was then used to substantiate the CAMFE findings and to study the effects of incomplete wetting of the 10R zone. To determine that the 10R distance encompasses the influence zone, two separate executions of the FENAIL code were performed, one with the boundary of the wetted zone at 14R and one at 10R. Figure 3.12 shows the results plotted together. This plot indicates that little error is experienced between the 14R and 10R executions, justifying the CAMFE results.

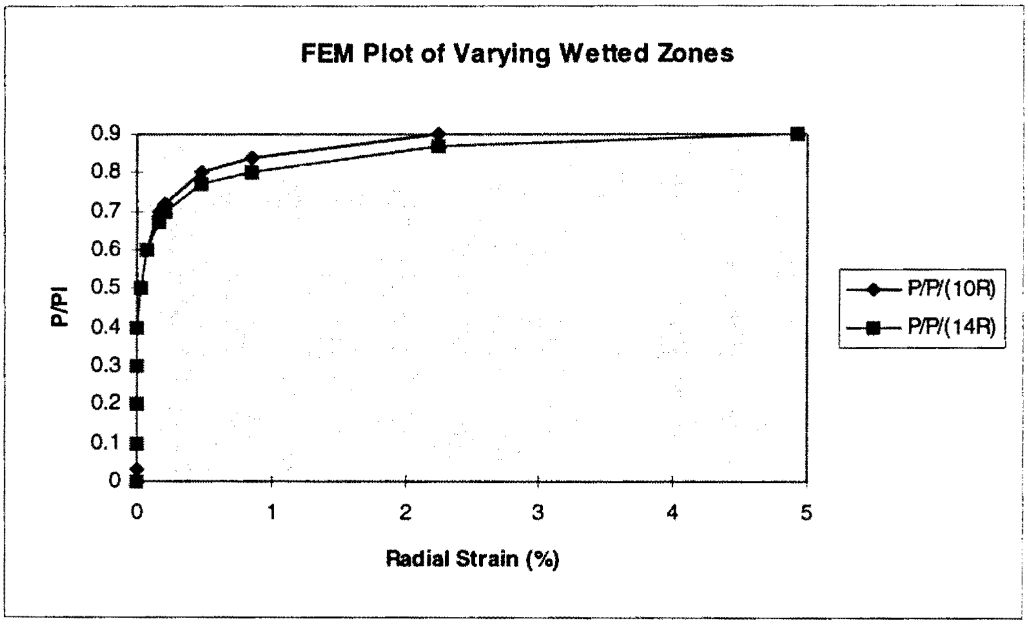

FIG. 3.12. -- Verification of Influence Zone

When performing the wet PMT test or the dry/wet PMT test (procedure 2 or Procedure 3) water is introduced to the soil through collar slots at the top of the probe and an aperture in a modified shoe at the bottom (Figure 3.13). The volume of water and the wait time determined by Smith and Rollins (1997) are intended to be such that the two separate moisture fronts indicated on the schematic will coalesce and extend to a distance of 10R beyond the probe. A FEM study of the effects of a reduced volume of water and of non-coalescence was made to account for soil variability. Moisture patterns for cases 1C - 1F are depicted in Figure 3.14, and the FEM stress/strain results plotted in Figures 3.15 and 3.16. Selected stress/strain results were plotted with the aid of TECPLOT and are shown in Figures 3.17 (a) and (b). All results clearly indicate that coalescence is essential for a valid test. Should there be any doubt, extra water and/or time should be provided when performing the test.

3.7 Commentary on Time History Effects

Procedure 3, a dry/wet test sequence, was also performed at Nephi and an attempt was made to model the field results making use of the FENAIL time history function and the excavation/construction feature. Using the pre-determined soil properties from the dry and wet FEM models the new model was created where by, at a selected pressure level, the dry soil in the 10R influence zone was "excavated" while, at the same time, this zone was "constructed" with the wet soil. The pressure was held constant for two steps of the execution, the first giving results with the dry soil in place and the second giving the results after the wet soil had replaced the dry soil in the influence zone.

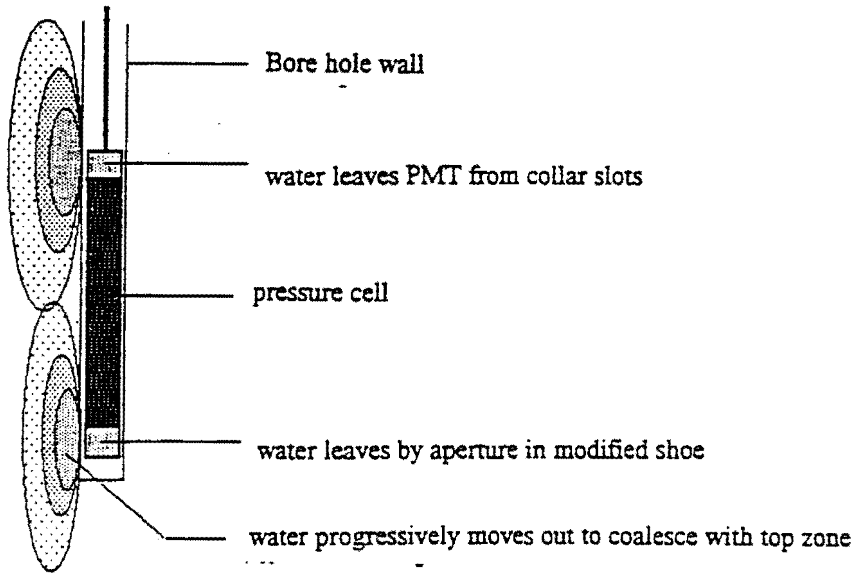

FIG. 3.13. -- Schematic of Axisymmetric Moisture Migration (after Smith and Rollins, 1997)

XXX
XXXX
XXXX
XXX
XXX
XXXX
XXXX
XXX

Condition 1C
(decreased volume)

XXX
XXXX
XXX
XX
XX
XXX
XXXX
XXX

Condition 1D
(decreased volume)

XXXX
XXXX
XXXX
XXX

XXXX
XXXX
XXX

Condition 1E
(decreased volume)

X
XXXX
XXXXX
XXXXX
XXX

XXX
XXXXX
XXXX
X

Condition 1F
(equal volume)

FIG. 3.14. -- Moisture Patterns Used In FEM Coalescence Analysis

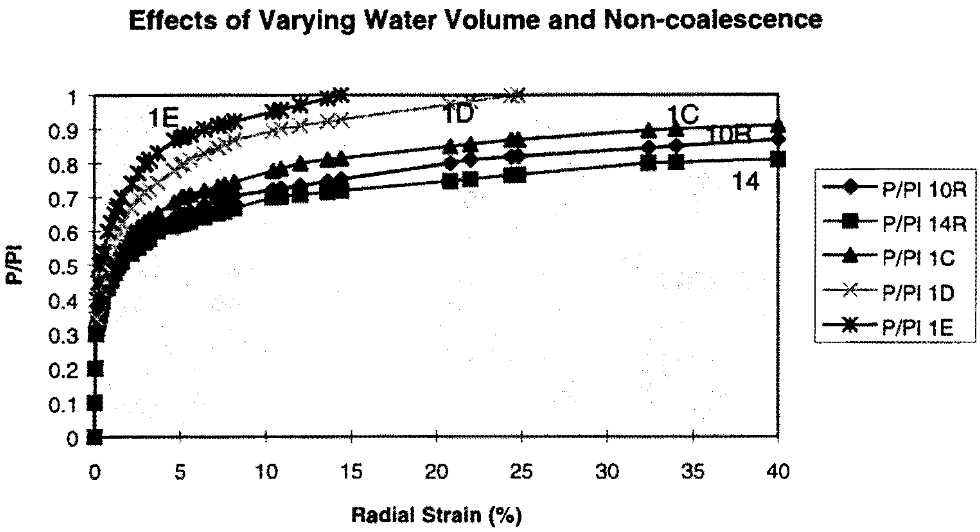

FIG. 3.15. -- Effects of Varying Water Volume and Non-coalescence

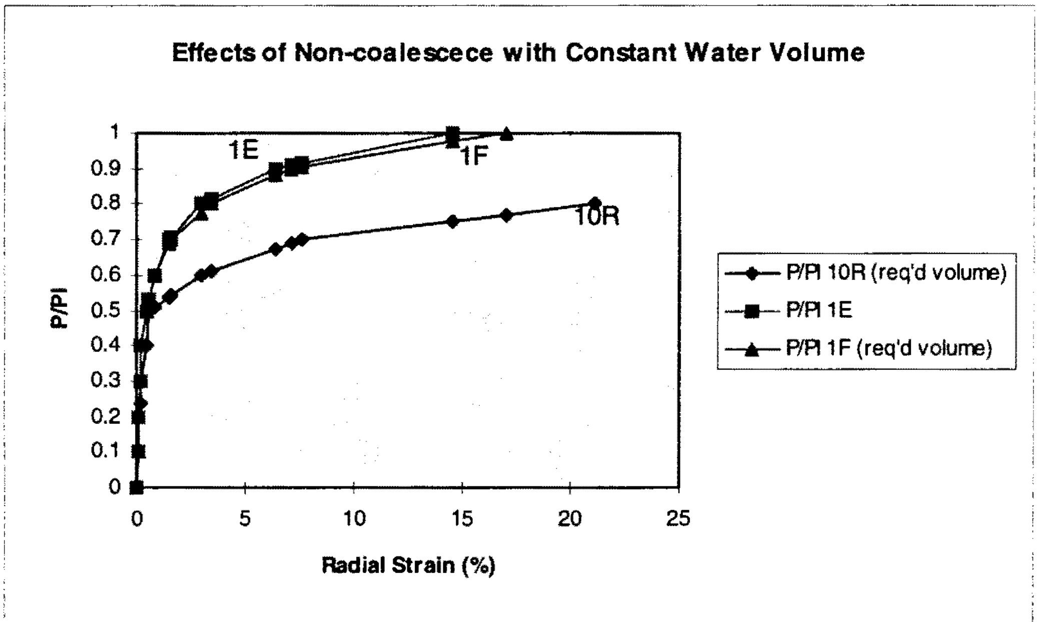

FIG. 3.16. -- Effect on Non-coalescence With Constant Water Volume

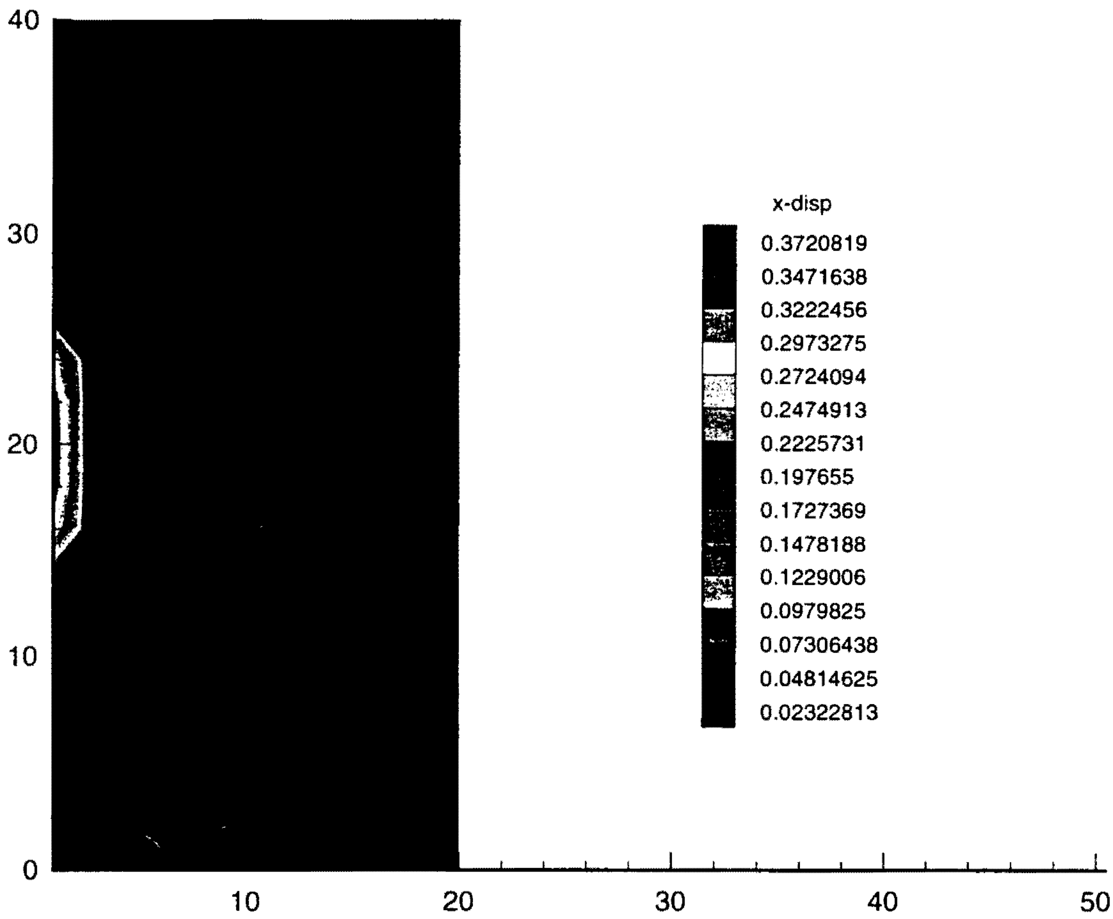

FIG. 3.17 (a). -- TECPLOT Plot Of Horizontal Displacement for Moisture Pattern 1D

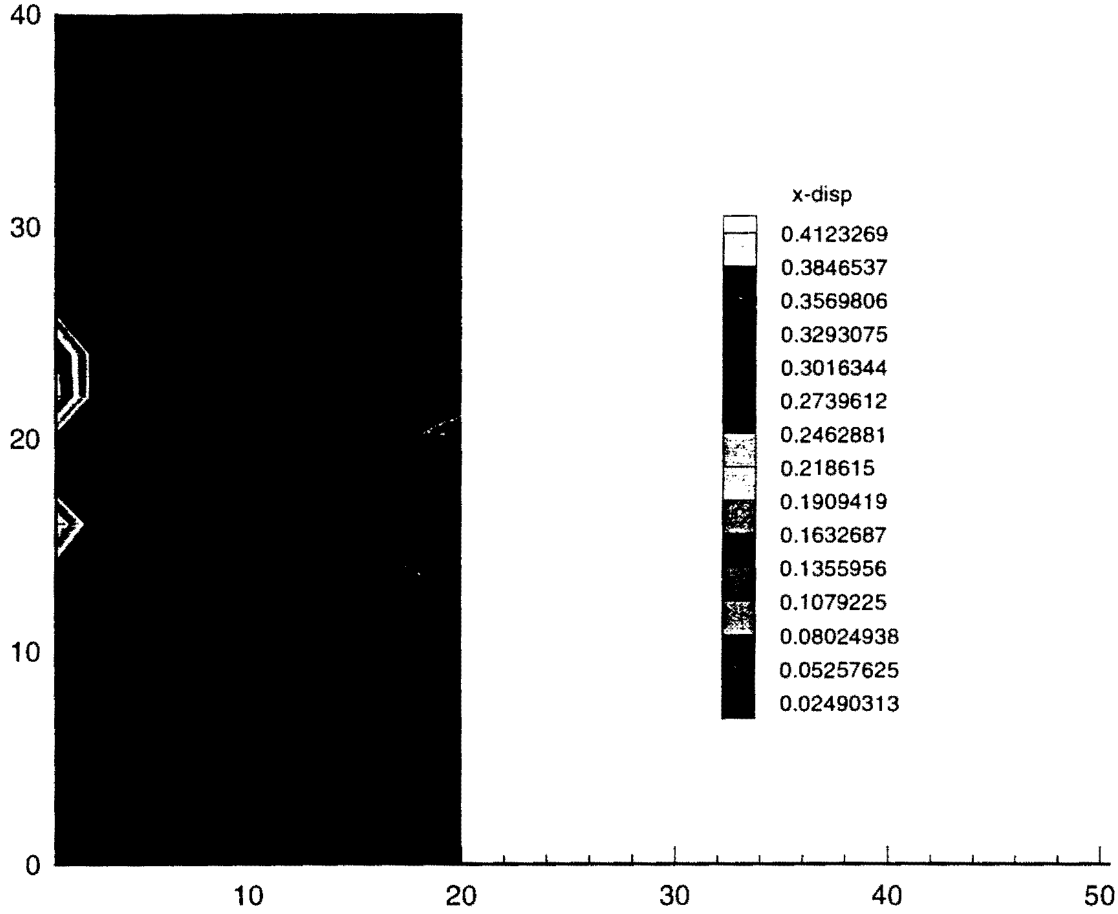

FIG. 3.17 (b). -- TECPLOT Plot Of Horizontal Displacement for Moisture Pattern 1E

While the code has the capacity to execute such time history sequences, the hyperbolic soil model was unable to accommodate the frictional behavior and the significant changes in properties of the collapsible soil. No reasonable results could be obtained for the Nephi case, but the FEM procedure itself was validated by using two less dissimilar soil types and increasing the allowable tolerance level. A generic plot of the result is shown in Figure 3.18.

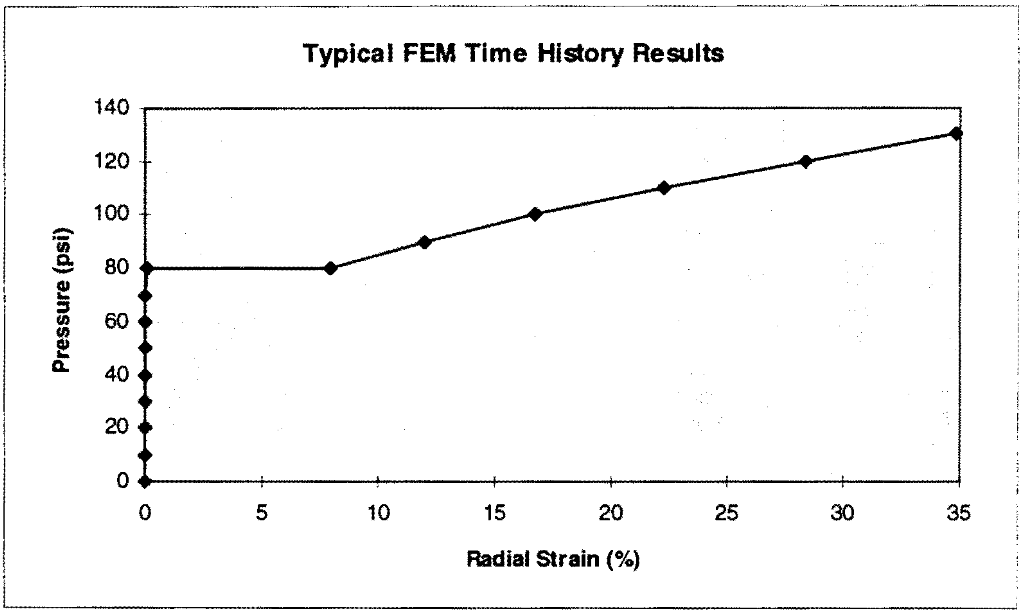

FIG. 3.18.-- Typical FEM Time History Results

With continuing improvements to the soil models of the code it should be possible to effectively model the procedure 3 PMT test sequence in the future.

3.8 Summary

Through finite element modeling it is seen that the collapsible silt under study at Nephi behaved as a *frictional* material when dry and exhibited some cohesion, but remained primarily *frictional*, when wet. This is contrary to the present assumption that silts behave cohesively, with grain sizes that do not exhibit strong frictional characteristics. However, it is supported by other mainly observational evidence from Utah (Rollins, personal communication, 1996). It seems clear that the absorption of water into the soil fabric does not allow for the development of excess pore water pressures. The PMT test may be somewhat analogous to the double consolidometer test. In each case a separate test is performed under dry and wet conditions and the results plotted together. At a given pressure the change in void ratio in the case of the consolidometer and the change in radial strain for the PMT define a collapse window particular to that stress level. Non-dimensional plots of PMT field results under wet conditions exhibited evidence of a series of "intermediate limit pressures" and "mini-collapses" before the final limit pressure was reached. Such behavior is believed to be attributable, at least in part, to the frictional nature of the soil, but a complete examination of the phenomenon is beyond the scope of this study. It was further determined, through used of the finite element models, that the zone of influence about a PMT probe is approximately ten times the radius of the probe.

Models of procedure 3, the dry/wet PMT test, met with limited success, but were insufficient to add significantly to this study. The FEM code modeled an instantaneous change in soil properties and was not capable of modeling the actual wait time during which the soil was flooded. Intermediate soil properties remain uncertain and an accurate model was not possible.

4.0 COLLAPSE POTENTIAL

4.1 Introduction

To determine the possibility and/or severity of potential collapse a number of observations regarding the soil's properties can be made. Potential for collapse may be evident during site reconnaissance if sink holes, open structured deposits, or if excess settlement of existing structures are noted. Typical subsurface investigations making use of SPT blow counts have shown to be an unreliable source for determining collapse potential, since, under dry conditions, blow counts may be quite high indicating a dense, competent soil layer. However, comparisons of soil characteristics determined by in situ tests, particularly the PMT test, can provide a qualitative and quantitative measure of the collapse potential of a soil.

The dry/wet test results shown in Figure 4.1 (after Smith and Rollins, 1997)

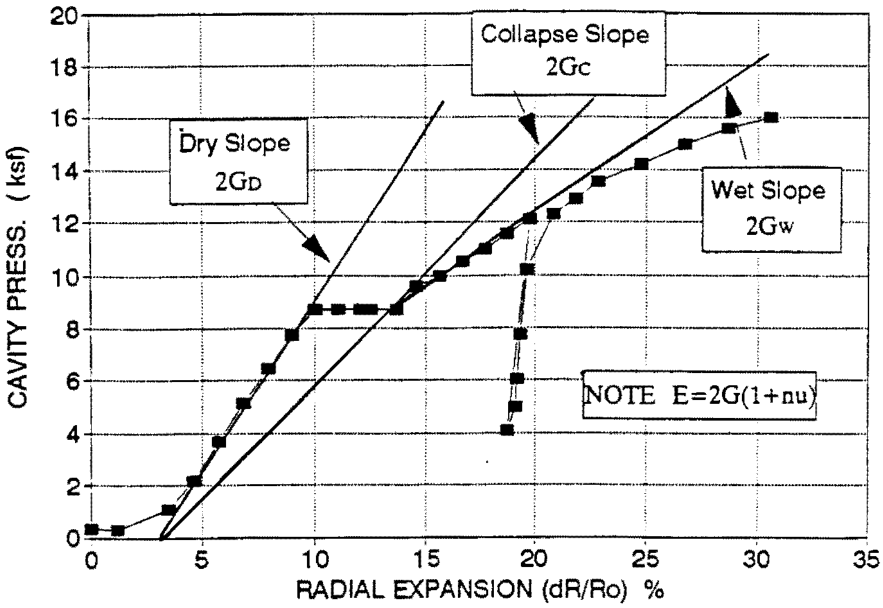

FIG. 4.1. -- PMT Test Curve Of Collapse Strain and Modulus Values (after Smith, 1997)

provide a quantitative measure of collapse strain, as well as information regarding dry, wet, and collapse modulus values. Ratios of these modulus values may give insight into the elastic collapse potential of the soil. Recall that a true elastic modulus during collapse is undefined and it has not been determined whether the collapse is settlement based ,or failure based. Thus, the appropriateness of this method may be questionable. However, since no other theory is available, it is necessary to rely on an "elasticity" based approach.

Separate PMT tests performed under dry and wet conditions will: verify the dry and wet modulus values obtained in Procedure 3, establish a dry limit pressure (P_{lD}), and wet limit pressure (P_{lW}). Again, a ratio of the wet limit pressure to the dry limit pressure may give an indication of collapsibility, but it is more likely to be related to bearing capacity issues. A large data base of dry and wet PMT test pairs has been compiled for alluvial collapsible soils found in the western United States (Duquette, 1995). The data base of moisture sensitive soils includes six major soil types: clays (CL), silty sands (SM), silty gravel (GM), silt (ML), and sand (SC and SP). Information regarding pressuremeter modulus, limit pressure and modulus/limit pressure ratio is catalogued according to soil type and site location. This data base represents a valuable resource for the development and validation of any settlement method.

4.2 Modulus Ratios

PMT modulus values are obtained under conditions of compressive increases in radial stresses and tensile increases (compressive reduction) in circumferential stresses.

In elasticity the decrease in circumferential stress (tension being negative) below the horizontal stress due to overburden, (p_{OH}), is equal to the increase in radial stress above p_{OH} up to yield. It is unlikely that many soils can resist any tensile stress, yet this is the theory used to obtain the PMT Modulus, E_o. An elastic approach studying the influence of having a modulus in the circumferential direction different from the modulus in the radial direction is carried out by Briaud (1992). It was previously shown that the PMT modulus needs to be multiplied by a correction factor, $\frac{1}{\alpha}$, in order to obtain a 'compressive' soil modulus E_s. $\frac{1}{\alpha}$ appears to vary from 1 for overconsolidated clays to 4 for very dense sands.

Soil modulus values have long been considered a key to understanding and predicting the elastic settlement of soils, and this is the basis for its value in interpreting collapsible soil behavior. In stable soils the introduction of water does not effect a change in soil properties and mud rotary drilling is required in the ASTM Standards for PMT testing. This is true for stable clays, as the permeability of clay is such that the water does not infiltrate and the wet mud in the borehole is displaced by the probe. For sands the moisture in the borehole rapidly drains to the phreatic surface, and again there is no change in soil properties. Thus, should a PMT test be performed under dry conditions, the soil properties, including the modulus, would be identical to those obtained with mud rotary drilling, so for stabilization of the borehole wall, ease in removing cuttings, and minimizing borehole wall disturbance mud rotary drilling is demanded.

However, in collapsible soil there is normally a significant change in modulus values between dry and wet conditions and, at the time of collapse, a point where a modulus value is ill-defined. The PMT helps to illustrate the change in elastic properties a collapsible soil undergoes when wetted. Figure 4.1 defines the dry and wet PMT modulus values, E_D and E_W, respectively, and a collapse PMT pseudo-modulus, E_C. The magnitude of the change and the potential severity of collapse is most easily represented by the use of the modulus ratios suggested by Smith and Rollins (1997) and with reference to Figure 4.1 where if:

<u>Condition 1:</u> $\frac{E_C}{E_D} \approx 1$ and $\frac{E_W}{E_D} \approx 1$ The soil is Not Collapsible

<u>Condition 2:</u> $\frac{E_C}{E_D} < 1$ and $\frac{E_W}{E_D} \approx 1$ The soil possesses moderate collapse potential

<u>Condition 3:</u> $\frac{E_C}{E_D} \approx 1$ and $\frac{E_W}{E_D} < 1$ The soil possesses moderate collapse potential

<u>Condition 4:</u> $\frac{E_C}{E_D} < 1$ and $\frac{E_W}{E_D} < 1$ The soil possesses severe collapse potential

Criterion for differentiating between approximately equal to 1 and less than 1 has not yet been established, and should be based on the judgment of the practitioner. However, it is likely that in most potentially collapsible soils $\frac{E_W}{E_D}$ will be somewhat less than $\frac{E_C}{E_D}$, so the same criteria should not be applied to both ratios.

Assuming collapse is at least partially due to failure, soils that undergo a large reduction in limit pressure when wetted will also exhibit a collapse modulus considerably less than the dry modulus. Based on the reduction in modulus a qualitative assertion of collapse potential related to soil type can be attempted, making use of the PMT test pair data base. Using only high quality PMT test results Table 4.1 has been constructed. For the limit pressure ratios, LPR = $\frac{P_{lW}}{P_{lD}}$, significantly less than 1, it is inferred that the collapse modulus, E_C will be much less than the dry modulus, E_D, and the ratio, $\frac{E_C}{E_D}$, will be less than 1. The PMT test results shown in Table 4.1 are from collapsible soils located at various sites in the western United States, including Utah, California, and Arizona.

Table 4.1: Collapse Potential by Soil Type

Location	Soil Type	LPR	Ec/Ed	Ew/Ed	Collapse Potential
	USCS		(inferred)		
Fredonia	SM	0.59	<1	0.19	Possible Severe
	SM	0.56	<1	0.84	Moderate
	SM	0.79	<1	0.84	Moderate
Ferron, UT	ML/CL	0.75	<1	0.93	Moderate
Greenslake Dam,CA	CL	0.25	<<1	0.18	Severe
	CL	0.31	<<1	0.08	Severe
	ML	0.41	<<1	0.25	Severe
Magma, AZ	ML	0.84	=1	0.39	Moderate
	ML	0.4	<<1	0.91	Severe
	ML	0.76	<1	0.6	Moderate to severe
McCoy Wash, CA	SW/SM	0.7	<1	0.37	Moderate to severe
Shavano 2	SP/SM	0.7	<1	0.59	Moderate to severe
	SC	0.91	=1	0.84	slight
	GM	0.87	=1	0.39	Moderate
White Tanks, AZ	GM	0.41	<<1	0.13	Severe
	GM	0.55	<1	0.44	Severe
	GM	0.95	=1	0.79	Moderate

Table 4.1 indicates that for the silts and sandy silts tested the collapse potential could be classified as moderate to severe. This is substantiated by visual observations made at the Fredonia test sites (Deal, personal communication, 1996) and reported by Smith, (1991). Greenslake Debris Dam suffered subsidence on the order of 2 ft. (0.6 m), (Smith, 1991) and the clays tested showed predicted collapse potential was severe. In the coarse grained deposits moderate to severe collapse was predicted. Since the data

base consists primarily of tests performed in known collapsible deposits, the modulusratio method of predicting collapse potential appears to correctly identify such potential.

Table 4.1 also indicates that when moderate collapse is predicted, the silty sand deposits exhibit Condition 2 properties, where the $\frac{E_C}{E_D}$ ratio determines collapsibility. The silts and gravels tested exhibit Condition 3 characteristics in that the $\frac{E_W}{E_D}$ determines the potential for collapse. Average modulus values based on soil type (Duquette, 1995) and their ratios suggest that most moderately collapsible soils, regardless of composition, exhibit Condition 3 characteristics.

4.3 Limit Pressure Ratios

The limit pressure of a soil is representative of soil failure in shear and, hence, the bearing capacity. This soil property is not overly sensitive to disturbance and is usually obtained from PMT results with a reasonably high degree of reliability. Making use of the notable change of limit pressures observed from dry to wet conditions, a relationship of limit pressure to collapse potential is desirable.

In collapsible soils the limit pressure can be reduced by as much as an order of magnitude when water is introduced, but in non-collapsible soils the limit pressure remains essentially unchanged. This would imply that as the ratio $\frac{P_{lW}}{P_{lD}}$ becomes small, indicating a decrease in bearing capacity, the potential for collapse increases. A soil of reducing shear strength will exhibit collapse settlements due to plastic strain. Field evidence substantiates that a reduction in limit pressure is normally present in wetted collapsible soils, however, whether the relative reduction is proportional to the degree of collapse is not certain. Limit pressures define the point where expansion of the probe continues with no additional pressure. Because it is assumed that infinite strain is occurring at this limit pressure of both the dry and wet tests, a distinction between the resulting strains and degree of collapse cannot be made. The plastic failure defined by the limit pressure is a measure of bearing capacity, not strain related settlements, which emphasizes the notion of collapse being a two part phenomenon involving both bearing failure due to loss of strength and collapse due to a reduction in modulus.

4.4 Collapse Strain

Another measure of collapse potential is the PMT collapse strain. PMT collapse strain can be considered to be the increase in strain that occurs with the addition of water while pressure remains constant at a predetermined level before yield. Figure 4.1 is a generic plot of a dry/wet PMT test with the collapse strain portion of the plot indicated. The pressure level at which water is added may be chosen as the yield point (where the curve goes non-linear) of the dry PMT reference test curve to obtain the maximum collapse strain, or at a stress level to match design loads.

It is not clear whether the collapse that occurs during the wetting of the soil is due to elastic settlement issues, shear failure, or, quite possibly, a combination of the two. Figure 4.2 is a schematic of the collapse zones that may be present during the wet portion of a PMT test, which includes a zone near the probe where plastic deformation may occur, a zone under "trigger conditions" where the moisture and load conditions are at the exact combination to induce collapse, and a portion at the periphery where pressures are reduced and strains are elastic.

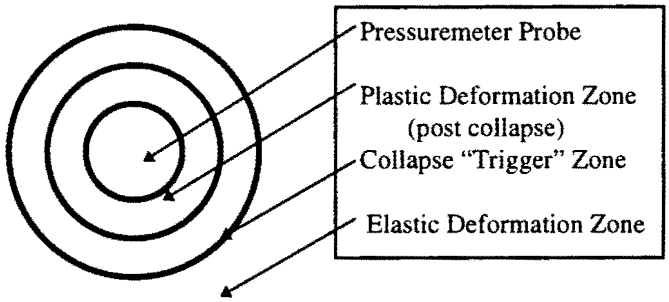

FIG. 4.2. -- Schematic of Zones That May be Present During Wet PMT Test in Collapsing Soils

The majority of research in settlement prediction of collapsible soils thus far has been consolidation based settlement due to overburden and/or loading and has made use of consolidation test results. Those who have studied this avenue include Andrei and Manea (1980), Lawton et al. (1989), Ismael (1991), Rollins et al. (1994), Menard, Inc. (1981), Hansen et al. (1989). Limited studies have been done with plate load tests by Houston et al. (1995) and Mahmoud et al. (1995). However there is very little in the literature on shear stress (failure) induced collapse. PMT testing may provide some of the best data for studying this component of collapse, but published information is limited, Smith and Rollins (1997), Rollins et al. (1994), and Varaksin (1987), and in most cases actual settlements have not been reported. Studies have been reported using the PMT to measure swell potential in sensitive clays (Flavigny et al. 1995). No known information regarding bearing capacity related collapse is available.

The recommended test sequences for the modified consolidometer and the PMT do not allow for the distinction of the nature of the collapse, only that collapse does occur. Other testing methods under study, also, do not appear to make reference to the possible dual nature of the collapse. Current settlement methods address only the settlement portion of the collapse, whereas bearing capacity issues, which may play a significant role, have not entered into the analyses. It would appear that the limit pressure may give an indication of the bearing capacity, or failure, component due to increased load. Consolidation related settlements may be best predicted by use of the soil modulus.

4.4.1 PMT Strain vs. Consolidometer Strain

Laboratory testing for hydroconsolidation potential involves a modification of the standard consolidation test, ASTM D 2435, 1987. Samples obtained with the ring sampler are transferred to the consolidometer ring with as little disturbance as possible. At natural moisture content the sample is loaded to a predetermined level. With this load held constant the sample is allowed to flood with water for four hours and the resulting strain increase recorded in order to determine the magnitude of the hydroconsolidation potential (Romani and Hick, 1989). Figure 4.3 indicates typical stress/strain test results using the modified consolidation test procedure with the hydroconsolidation potential of the sample at nearly 0.04 in/in (4%).

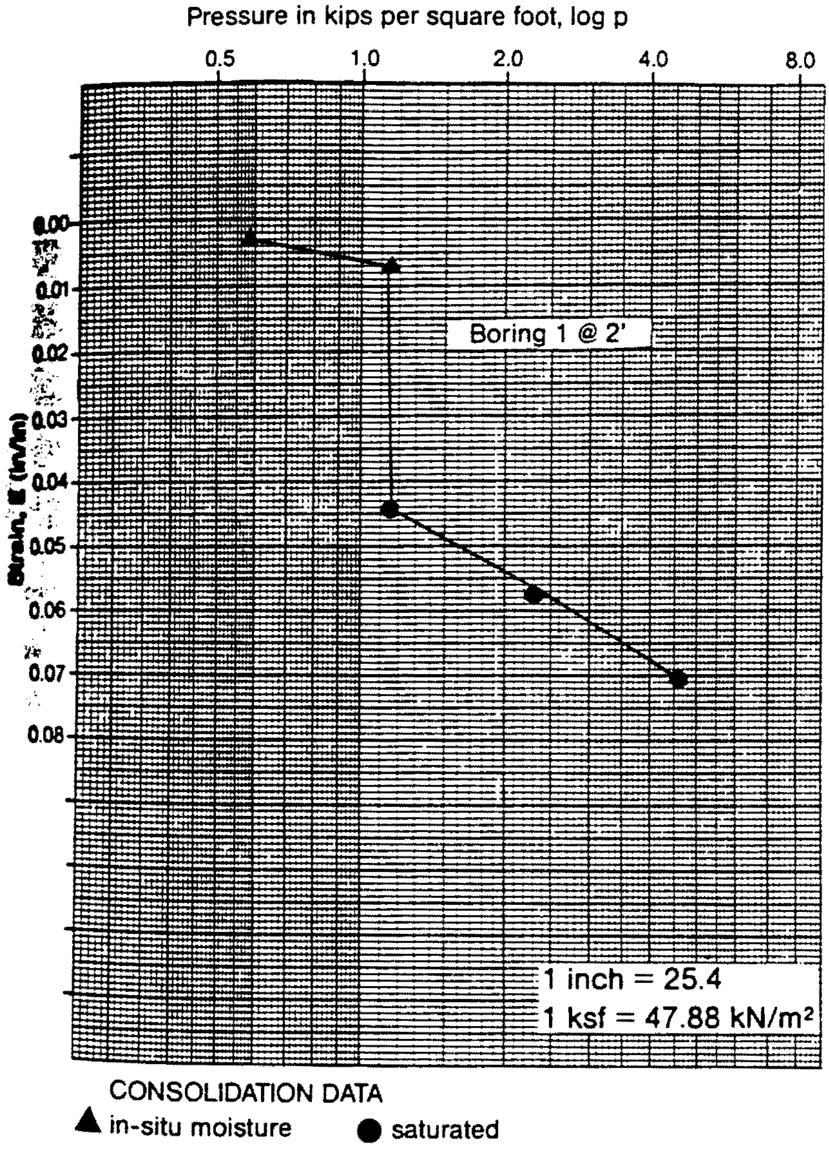

FIG. 4.3. -- Consolidation Test Results to Determine Hydroconsolidation Potential (after Beckwith and Hansen, 1989)

While this modified consolidation test is performed under a vertical orientation of the sample, with loads applied and displacements read in the vertical direction, the basic premise of the test is similar to that of the dry/wet (Procedure 3) PMT test recommended by Smith and Rollins (1997). However, for the PMT the test induces by *shear stress*, and the end result is a measure of the *radial strain* that occursunder a constant pressure when the soil is flooded. The resulting change in radial strain from dry conditions to wet conditions gives a measure of collapse strain, as does the hydroconsolidation potential of the modified consolidometer test. A distinct advantage of the PMT is the reduction of sample disturbance, particularly in coarse-grained deposits, however, the laboratory test affords the ability to verify complete sample flooding, which may be difficult to discern in the PMT test sequence. Both test procedures provide a quantitative measure of the collapse potential of a collapsing soil but under quite different stress paths (see Figure 4.8). The PMT test, being a purely shear test with the decrease in circumferential stress (tension being negative) below the horizontal stress due to overburden, (p_{OH}), equal to the increase in radial stress above $p_{OH,}$ takes the soil directly to shear. Whereas the consolidation test does not produce failure, but rather the increase in vertical stress induces an increase in horizontal stress and the resulting stress path defines the K_0 line.

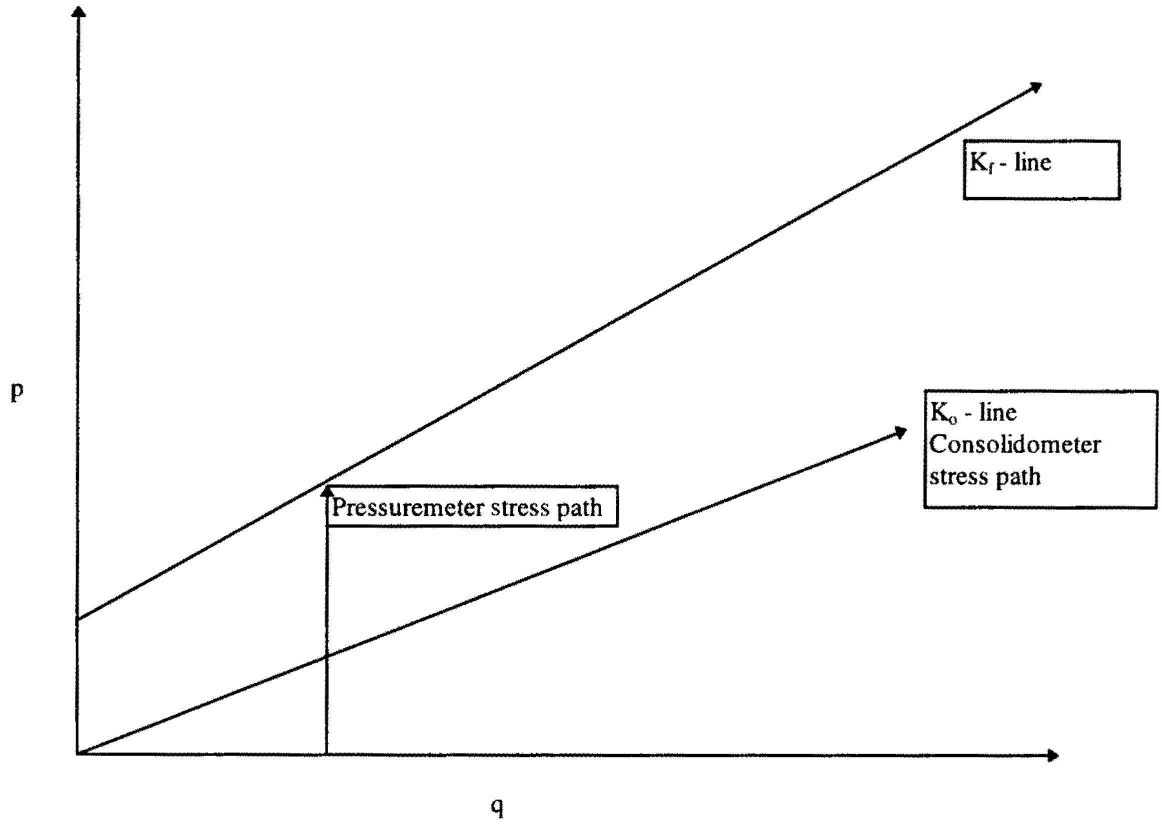

FIG. 4.8. -- Pressuremeter and Consolidometer Stress Paths

4.4.2 Alternate Method for Determining PMT Collapse Strain in Silty Collapsible Soils

Using the PMT test Procedure 3 the value of the PMT collapse strain can be obtained directly from the reduced PMT data. The collapse strain is the strain increase from water addition when the predetermined pressure is held constant for 30 minutes. This is the collapse strain applicable at the selected pressure only, and may differ for other pressure levels. This collapse strain that occurs under constant pressure is defined on Figure 4.1.

Alternatively, a collapse strain for silty soils can be inferred from a dry PMT test result coupled with a wet PMT curve from the same location. Based on the data from two separate sites in Utah (see Chapter 3) the following semi-empirical relationship is proposed to define the collapse window between the two curves and the collapse strain ε_c.

$$\varepsilon_c = C\left(\frac{P_{lD}}{P_{lW}}\right)\left(\frac{P_D - P_W}{P_D * - P_W *}\right)(\varepsilon_D) \tag{4.1}$$

where:

C = collapse constant

ε_c = collapse strain

P_{lD} = limit pressure of dry curve

P_{lW} = limit pressure of wet curve

P_D = pressure on dry curve at which water is added (at corresponding strain, ε_{H_2O})

P_W = pressure on the wet curve which corresponds to ε_{H_2O}

$P_D *$ = yield pressure of dry curve

$P_W *$ = yield pressure of wet curve

ε_D = strain at yield on dry curve less the strain value at the intersection of the projection of the linear portion of the curve and the horizontal axis.

The variables are indicated for a typical dry/wet PMT pair in Figure 4.4.

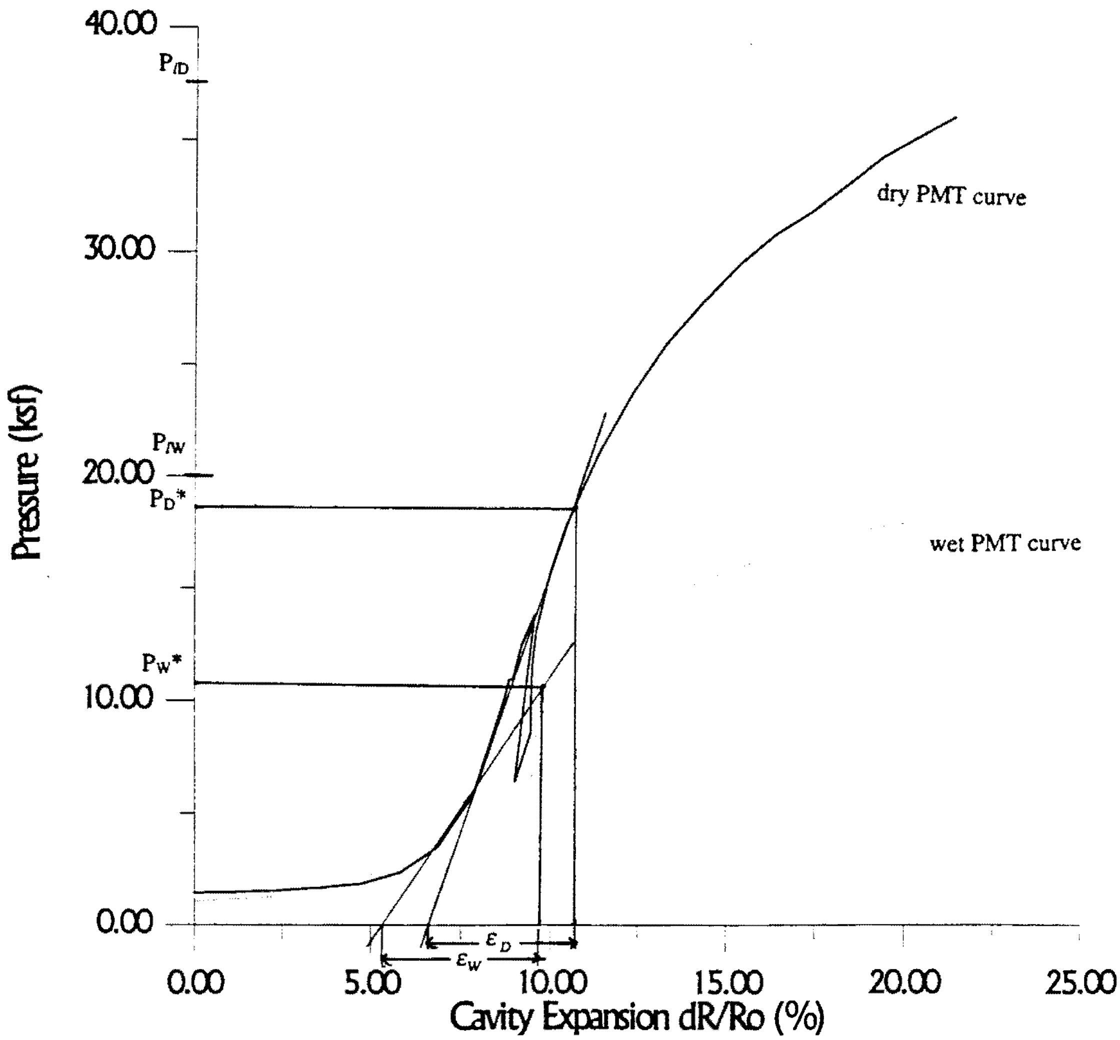

FIG. 4.4. -- Illustration of Collapse Variables For Dry/Wet PMT Test Pair

The constant, C, has been determined to be 0.115 by using three separate PMT Procedure 3 tests in silty soils with known collapse strain, limit pressures, strain at yield for the dry curve and pressures where the addition of water was known to occur. In all cases the soils were primarily silts, and it is suspected that this constant would vary according to soil type, likely decreasing as the percent gravel increased, but further testing is required to validate this assumption. This relationship was further verified using PMT lab data on laboratory fabricated collapsible diatomite silt reported by Denham (1992). Figures 4.5, 4.6, and 4.7(a), (b), and (c) show the PMT results used in the development of equation 4.1. The constant, C, has not been identified for soils other than the silty soils at the Nephi and Provo, Utah sites and the laboratory soil due to the limited number of Procedure 3 test results available. For silty soils $C = 0.115$ could be used in the absence of site specific data.

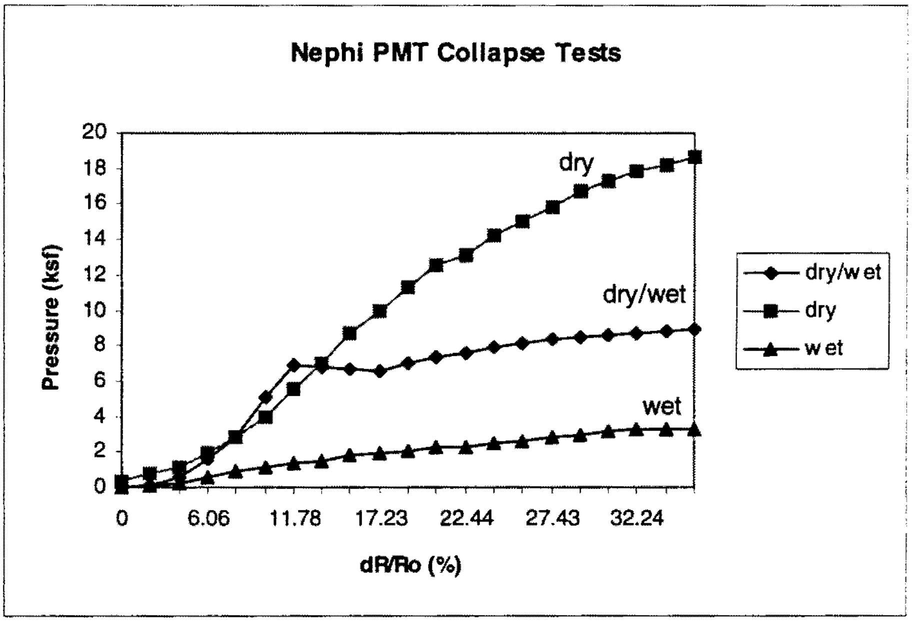

FIG. 4.5. -- Nephi PMT Results

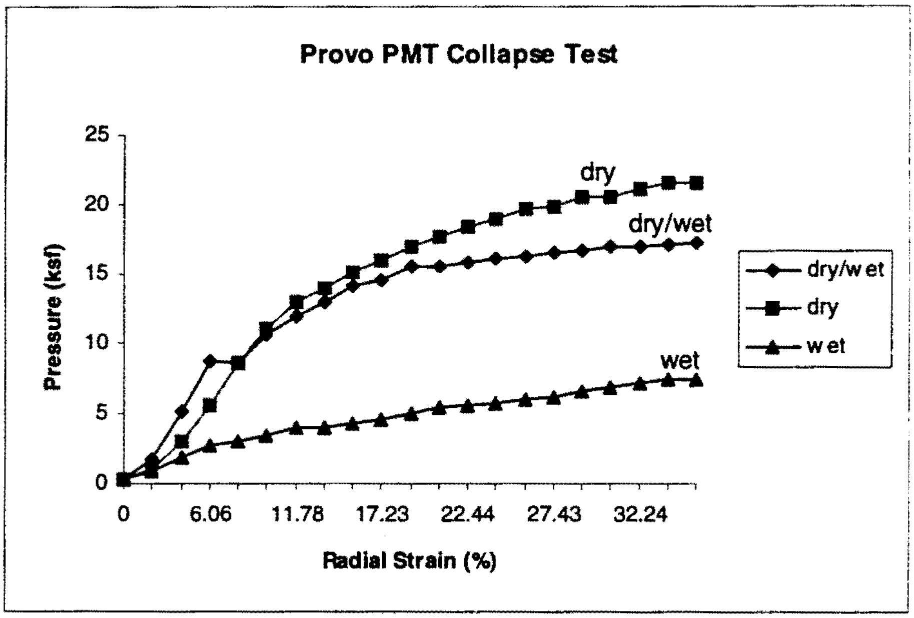

FIG. 4.6. -- Provo PMT Test Results

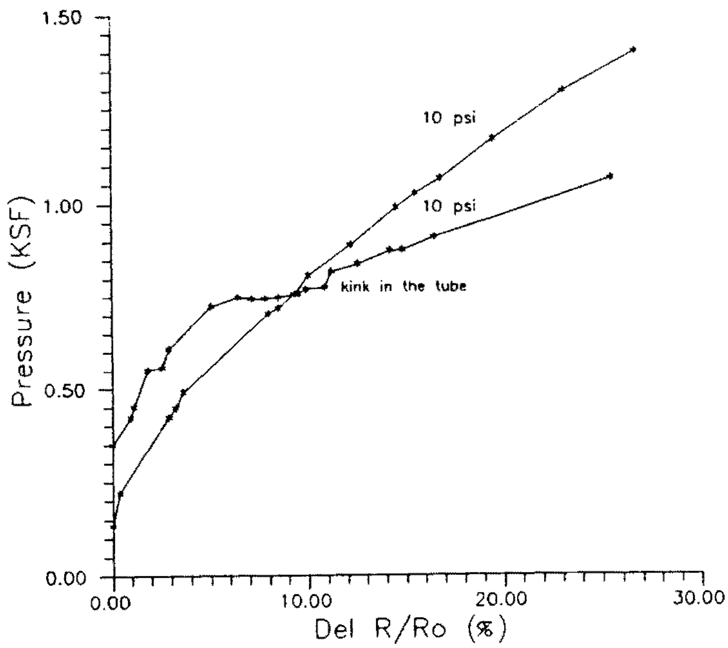

FIG. 4.7 (a). -- Laboratory Diatomite Silt PMT Results: Dry Condition, Procedure 1 (after Denham, 1992)

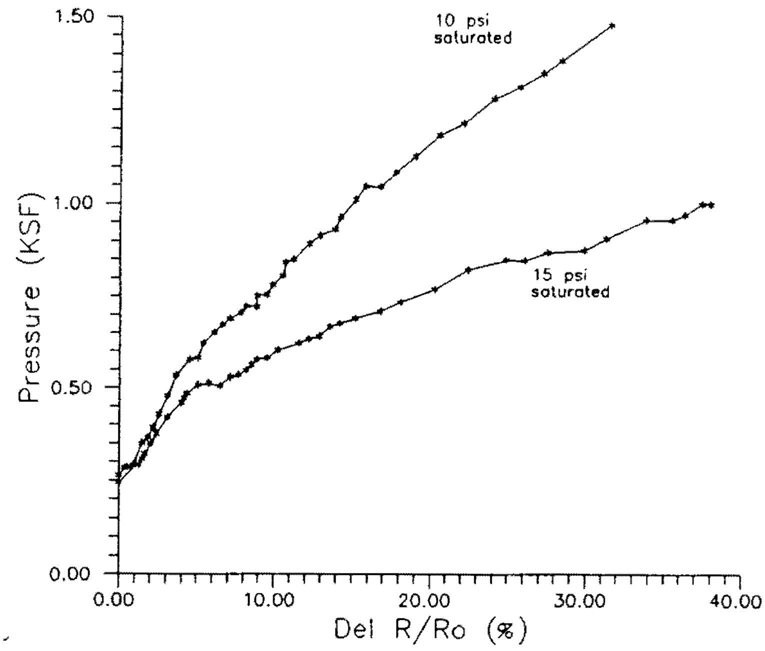

FIG. 4.7(b). -- Laboratory Diatomite Silt PMT Results: Wet Condition, Procedure 2 (after Denham, 1992)

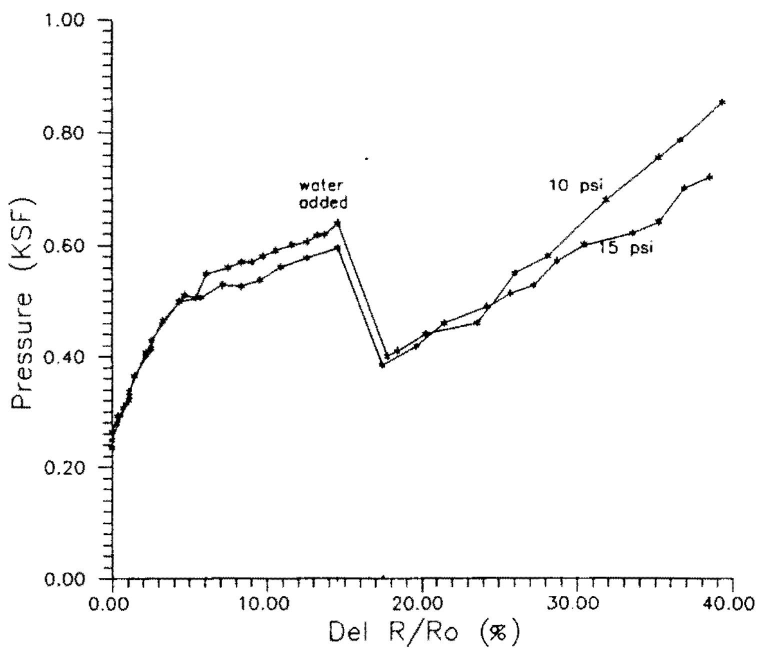

FIG. 4.7 (c). -- Laboratory Diatomite Silt PMT Results: Dry/Wet Condition, Procedure 3 (after Denham, 1992)

It is suspected that as the percentages of coarse grained material increases, inhibiting the symmetrical expansion of the probe, the value of C may also increase but there is insufficient data to test this hypothesis.

4.5 Summary

It is evident that collapse potential is not adequately predicted by use of SPT data from a typical subsurface investigation. To provide a means of predicting the collapse potential of a soil several PMT methods have been proposed which can be viewed as qualitative measures of the phenomena. The difficulty in ascertaining whether the soil collapse produced is elastic based settlement, bearing failure, or a combination of the two currently limits the theoretical usefulness of PMT collapse strain values and soil modulus and limit pressure ratios beyond establishing that the potential for collapse exists. However, the above measures may prove to be useful in the development of an empirical relationship for the prediction of collapse settlements.

5.0 SETTLEMENT ISSUES FOR COLLAPSIBLE SOILS

5.1 Introduction

Any procedure to be developed for the prediction of collapse settlement should include the variables that are particular to collapse. Difficulty arises with collapsible soils and the critical variable of moisture. There is presently no testing method that accurately measures the soil saturation level at which collapse occurs, although it is suspected that as little as 60% saturation can induce collapse (Rollins et al., 1994). Current testing methods assume 100% saturation, but no conclusive studies have been conducted that relate degree of collapse to percentage saturation. There is some indication that a "trigger" water content/load combination induces collapse and that additional moisture will not appreciably effect the settlement, but further research is necessary to ascertain moisture effects. This research would necessarily include not only the study of the moisture versus collapse relationship but also the development of a testing method that could measure the saturation level during collapse.

One observable and measurable variable is location within the metastable layer where collapse occurs. Study of test results in silty collapsible deposits (Rollins and Rogers, 1994) indicate that the depth below the ground surface where collapse occurs varies depending on the loading and wetting conditions present.

Collapse due to overburden alone, a condition where there is wetting of the soil with no additional load, occurs primarily at depths where overburden pressures become sufficient to induce collapse. In the Rollins study 80% of a 3 in. (80 mm) settlement, or approximately 2.5 in. (65 mm), was experienced at a level beyond 9.8 ft. (3 m) in a wetted zone of 14.8 ft. (4.5 m).

Later loading of a 4.9 ft. x 4.9 ft. (1.5 m x 1.5 m) full-scale model foundation on the wetted deposit to 1775 psf (85 kPa) caused an additional average settlement of 10 in. (250 mm), 80% of which occurred in the *upper* 3.3 ft. (1 m). Additionally, a similar foundation on an adjacent site that was loaded to 1775 psf (85 kPa) and then wetted to a depth of 14.8 ft. (4.5 m) experienced a collapse of 11 in. (280 mm), 75% of which occurred in the *top* 3.3 ft. (1 m). Only 0.4 in. (10 mm) of settlement was recorded beyond a depth of 9.8 ft. (3 m), significantly less that the 2.5 in. (65 mm) recorded due to the overburden alone of the adjacent site.

Such test results indicate that certain initial stress conditions, moisture levels and stress increases other than simply increases in vertical stress, are necessary for collapse to occur. The stress influence of a loaded shallow foundation with depth can be shown for a semi-infinite, isotropic, homogeneous, weightless, elastic half-space using the Boussinesq equations. At near surface depths both horizontal and vertical stresses increase, but the horizontal stress increases decay more rapidly with depth than the vertical stress increases resulting in an increase in shear stress. Since, under foundation loading, collapse occurs near the surface, it could be concluded that the presence of elevated horizontal and vertical stress coupled with low induced shear stresses, or an increase in the spherical component of stress, are requisite for collapse. This condition is present at increasing depth under embankments, or overburden for a weakened soil. However, soils in general are not isotropic, and it appears that collapsible soils may actually become *more* anisotropic after collapse with vertical modulus values increasing without a comparable increase in horizontal modulus. Results of SAP 90 (Wilson and Habibullah, 1978-1990) FEM modeling of extreme anisotropic conditions verify the settlement patterns described above and the assumption that anisotropic conditions are present. Therefore, the stress distribution obtained from the Boussinesq method would not be an actual representation of the stress distribution in a collapsible soil, but may be thought of as a rough approximation. Because of this variation in depth where the collapse occurs, care needs to be exercised when selecting testing depths for the procurement of soil properties to use for collapse predictions. As a preliminary guideline, properties near the surface may govern collapse settlements for shallow foundations whereas properties near the final depth of expected wetting may be the most significant for embankment type settlement calculations.

Other factors essential for a settlement estimation are the depth and thickness of the collapsible layer, the maximum expected depth of the moisture front and the loading conditions when applicable. The depth and thickness of the collapsible layer can be determined during the site investigation through continuous sampling methods, test pits, or possibly by CPT soundings. The expected depth of the moisture front migration will need to be approximated based on anticipated volumes of water and soil permeability.

5.2 Factors Influencing Collapse Due to Overburden

Evidence of sinkholes in an area of collapsible deposits is a strong indication that collapse has occurred under overburden stresses alone. This type of collapse is made up entirely by the S2 component of collapse settlement. To estimate the magnitude of such collapse that may occur in nearby sites the following procedure is recommended. Similar collapse evidence is observable within some engineered debris dams, but the following addresses only the condition of naturally occurring collapsible deposits.

It is first necessary to determine the layer thickness of the collapsible deposit and to estimate the depth of wetting that may be possible. In the case of irrigation the depth of wetting can be approximated by assuming a depth of penetration of the moisture front from the surface due to extensive watering operations. The largest volume of water expected and the permeability of the soil should aid in this approximation. For underground utilities, the wetted zone should be estimated from the bottom of the trench to the estimated depth of penetration of the moisture front. Again, some estimation regarding the volumes of water and the permeability of the soil should be made.

Realizing collapse due to overburden will most likely be significant at depth, an evaluation of collapse strain should be made near the bottom of the collapsible layer or the bottom of the estimated moisture front, whichever is nearest the surface. Determining the collapse strain can be achieved by the PMT, as suggested in Chapter 4. Collapsible soils are extremely sensitive to disturbance, and it is recommended that a reference PMT test in a hand augured hole be performed if possible. Correlation can be made to an adjacent PMT test completed at an equivalent depth in a conventional air rotary borehole. Future PMT tests at the site can thus be corrected for borehole effects. Once the depth of the collapsible layer and the collapse strain have been determined an estimation of the magnitude of collapse can be made. If the three components of settlement defined in Chapter 1 are accepted it can be seen that the collapse component, S2, is the only significant contribution to the total settlement.

5.3 Factors Influencing Collapse Under Existing Load

In the case of existing structures on collapsible deposits, the possibility of future collapse due to additional moisture is a prime concern. The magnitude of such collapse may determine the economic feasibility of any proposed development. If collapse is occurring, an estimation of the maximum expected settlement may aid in decisions of remediation efforts.

5.3.1 Shallow Foundations

Because of the probable horizontal variability of collapsible soils it is suggested that testing for soil properties be conducted directly beneath the existing foundation. Since evidence shows that the majority of the collapse under loaded shallow foundations occurs near the surface, collapse strain should be determined in the upper portion of the influence zone. However, when using the PMT for collapse strain determination, care should be exercised to ensure that the probe is at a sufficient depth to avoid surface effects. This has previously been determined to be 40 - 60 probe diameters below the surface (Smith, 1983). The depth and thickness of the collapsible layer and existing foundation loads should be determined and the expected depth of the moisture front estimated. Measured settlements at the Nephi site suggest that the vast majority of the collapse will occur in the upper portion of the influence zone to a depth of approximately

1B below the foundation depth. With the collapse strain, depth and thickness of the collapsible layer, depth of the moisture front and foundation loads determined an estimation of the collapse settlement can be made. In this case it is likely that all three settlement components will be present. Traditional settlement methods can be used to determine S1, the settlement that occurs prior to wetting of the soil. The collapse portion S2, can be estimated by making use of the suggested procedures in Chapter 6. If creep is neglected the total settlement can be estimated as the sum of S1 and S2.

5.3.2 Embankments

Stress conditions for an embankment on a collapsible soil will be similar to that of overburden on a weakened soil. If the collapsible layer becomes wet, the most significant collapse will likely occur towards the bottom of the wetted zone or the bottom of the collapsible layer, whichever is closer to the surface. PMT testing to evaluate collapse strain should be directly below the embankment at a depth that reflects the most likely location of significant collapse as described. The testing procedure should be identical to that suggested for the estimation of overburden collapse. Prediction of collapse should be calculated with an equivalent height that adjusts the collapsible layer thickness to reflect the embankment. This can be done by taking the height of the embankment times the unit weight of the embankment material which is then divided by the unit weight of the collapsible layer, (see figure 5.1). The resulting height value should be added to the height of the collapsible layer when estimating the magnitude of collapse. In some instances when onsite soils are used collapse may occur within the fill itself. This phenomenon has not been thoroughly explored but is discussed in more detail in Section 5.5.

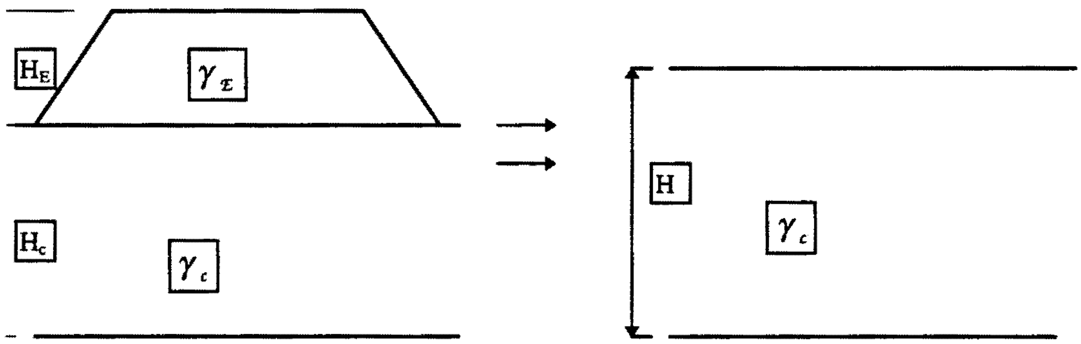

FIG. 5.1. -- Schematic of Equivalent Height

5.3.3 Remediation

When it appears that the magnitude of collapse under existing structures could be significant it may be necessary to implement measures to preserve the structure. By far the most obvious and least expensive alternative would be to install a drainage system that would insure that any water would be diverted away from the structure Without additional moisture, collapsible soil exhibits high strength and settlements are normally quite small.

However, if additional precautions are deemed appropriate, installing remedial micro-piles through the collapsible layer to competent material below can be an effective method of supporting the structure when collapse of the soil may either be underway or unavoidable. While this has been performed successfully and documented for a cement plant in Utah (Hepworth, 1988) with no interruption in the operation of the plant, it is a somewhat expensive alternative for existing structures, and a cost benefit analysis should be performed before the procedure is undertaken. Also, the effect of negative skin friction my play a significant role in the design of the piles.

Another option that is currently under study is the injection of a sodium silicate solution into the collapsible layer. This was studied at various sites in Utah and comparisons of PMT tests performed under dry, water treated and sodium silicate treated soils were made (Bleazard, 1992). The tests show mixed results with the sodium silicate treated soil properties showing improved from dry properties at some locations and worsening to conditions less than water treated soil in other locations. While this does not verify the effectiveness of the sodium silicate treatment, it does reemphasize the extreme sensitivity and variability of collapsible soils. Sodium silicate treatment was performed at the Nephi site with successful results (Rollins, 1994), but the treatment has not yet been applied to an actual existing structure to determine the practical success of the method.

5.4 Factors Influencing Collapse Under New Load

This section addresses the case of construction on a collapsible deposit that is currently wetted to a significant depth. For new construction on known collapsible deposits the magnitude of possible collapse is critical to the development of a foundation preparation procedure. When the potential for significant collapse exists it can be accommodated either by foundation design, or by preconstruction mitigation measures, thereby insuring the long-term integrity of the proposed structure.

5.4.1 Shallow Foundation

Load tests and measured settlements at Nephi show that the collapse that occurs when susceptible soils are initially prewetted *before* the load is applied are very similar to that which occurs when the wetting occurs *after* the loading. There was only a modest reduction in settlement when overburden collapse alone was allowed to precede the loading. The settlement due to external loading on the wetted soil amounted to nearly 90% (250 mm) of the settlement that occurred when the wetting of the soil took place after the foundation loading. The average overall collapse settlement due to the overburden collapse alone (80 mm) plus the external loading collapse (250 mm) *exceeds* the average collapse settlement of a foundation loaded to an equivalent stress level and subsequent wetting of the soil (280 mm). This is reasonable since the overburden collapse and the collapse due to foundation loading take place in different portions of the influence zone. This can be attributed to the stress conditions present at the time of wetting and/or loading and it illustrates the ineffectiveness of prewetting the soil for collapse control. It also suggests that the procedure for estimating collapse for new construction on wetted soils should be identical to that for estimating collapse for existing structures, except when an overlapping of the collapsing zones occurs.

5.4.2 Embankments

Stress conditions for overburden induced collapse and embankment collapse are similar. In both instances it is assumed that the most significant collapse occurs near the bottom of the wetted zone or the bottom of the collapsible layer, whichever is closest to the surface. Therefore, it can be assumed that the collapse under an embankment constructed on a wetted collapsible deposit could be estimated as the collapse induced by an embankment constructed on a dry collapsible deposit that is subsequently wetted less the collapse due to overburden. Since the depth of collapse is not shifting from the bottom of the collapsible layer to the top, as it seems to do under the newly constructed shallow foundations, the resulting collapse is the difference of the collapse due to the embankment and that due to overburden. This is in contrast to the shallow foundation where the overburden collapse was essentially neglected.

5.4.3 Mitigation Measures

Before construction commences in areas of known collapsible soils it may be prudent to assess the practicality of available mitigation measures. Typically, if the deposit is at the surface and of limited depth, it can be removed from the site and replaced with stable soil to subgrade level. For shallow foundations excavation of the collapsible soil in the influence zone may be all that is necessary.

However, should the collapsible layer and anticipated probable wetting zones be extensive alternative mitigation measures may be considered. Possible mitigation measures include the suggested remediation measures for existing structures: drainage control, deep foundations, and possibly sodium silicate treatment. In addition, dynamic compaction of the wetted soil has been shown to achieve favorable results in some studies (Rollins and Rogers, 1991). Pre-wetting of the soil before construction in isolation does little to control the magnitude of collapse and should not be viewed as a treatment method. However, prewetting the soil and applying a surcharge may be an effective way to preconsolidate the lower portions of the collapsible layer for embankment type applications.

5.5 Discussion of Collapse Within Fills and Deep Foundations

Observations of particular debris earth dams in the arid regions of the west have revealed that collapse can occur not only at the foundation level, but within the fill itself. Intra-fill collapse has been observed in fills of native materials that have been compacted and found to be in compliance with standard specifications, typically 95% of the maximum density and within 2% of the optimal water content as determined by the Standard Proctor. The collapse does not likely occur at the time of construction, but rather after a period of time after the completion of the project has elapsed. It is not certain what the underlying causes of this collapse may be, but the phenomenon should be recognized and careful long-term monitoring of projects making use of compacted collapsible soils considered. If it is determined that collapse is beginning to occur within a fill appropriate measures can be implemented and damage to the project arrested before it becomes severe.

Should deep foundations, or piles, be specified for either remediation purposes or for original design, careful attention should be given to the effects of negative skin friction that can occur during the collapse of a metastable layer. It has been determined that the collapsible soils analyzed thus far for this study behave in a frictional manner. It may also be suggested that, should the collapse be relatively instantaneous, there may be a dynamic effect due to the change in velocity of the soil. Given this and the oftentimes severe magnitude of collapse, the down-drag forces on piles within the collapsible medium may be considerable and should not be discounted in design.

6.0 SUGGESTED PROCEDURES FOR ESTIMATING COLLAPSE SETTLEMENT

Based on the knowledge of existing settlement methods and their applicability to collapsible soils, the versatility of the pressuremeter and the FEM analyses conducted in this study the following procedures for estimating collapse settlement are proposed. The proposed procedures reflect the frictional nature of the collapsible soil, the anisotropy of the soil, loading conditions at the time of collapse, and the consolidation type settlement that occurs during collapse.

To accommodate the anisotropic nature of the collapsible soil a pseudo-Poisson's ratio, or a collapse soil strain ratio, μ_c, is introduced to convert the radial collapse strain obtained from the pressuremeter to a vertical collapse strain that can be used for the estimation of the magnitude of collapse. Based on field PMT curves and measured settlements the collapse soil strain ratio at the Nephi site was estimated to be $\frac{\varepsilon_c}{\varepsilon_c'} = 0.67$

where: ε_c = the radial collape strain and

ε_c' = the vertical collapse strain

The loading conditions are considered when conducting the pressuremeter tests, with recommended probe locations reflecting the expected zone of collapse. Also, under overburden and embankment conditions, where increases in stresses are either non-existent or accounted for by other methods, equation 4.1 does not hold and a modified form of the equation making use of the strain at yield of the wet curve is substituted.

Depending on these conditions the average magnitude of settlement due to collapse can be estimated using the following suggested procedures. The recommended procedure estimates the *average* magnitude of collapse. The sensitivity of the soil, the moisture patterns, the eccentricity of loading, and other conditions contribute heavily to the magnitude of settlement, and maximum settlements may be significantly greater than the average value estimated.

The following steps outline the general procedure proposed for estimating the average magnitude of collapse that may be expected to occur in metastable silts. Specific modifications and discussion for each application follow the general outline.

1. Calculate The radial collapse strain, ε_c , using equation 4.1

$$\varepsilon_c = C(\frac{P_{lD}}{P_{lW}})(\frac{P_D - P_W}{P_D * - P_W *})(\varepsilon_D) \tag{4.1}$$

where:

C = collapse constant

P_{lD} = limit pressure of dry curve

P_{lW} = limit pressure of wet curve

P_D = pressure on dry curve at which water is added (at corresponding strain, ε_{H_2O})

P_W = pressure on the wet curve which corresponds to ε_{H_2O}

$P_D *$ = yield pressure of dry curve

$P_W *$ = yield pressure of wet curve

ε_D = strain at yield on dry curve less the strain value at the intersection of the projection of the linear portion of the curve and the horizontal axis.

or a modified form for overburden and embankment applications where:

$$\varepsilon_c = C(P_{lD}/P_{lW})\,\varepsilon_w \tag{6.1}$$

and ε_w = strain at yield on the wet PMT curve less the strain value at the intersection of the projection of the linear portion of the curve and the horizontal axis (see Figure 4.4).

2. Convert the radial collapse strain, ε_c , to vertical collapse strain, ε_c' , with a collapse soil strain ratio, μ_c , using the relationship:

$$\frac{\varepsilon_c}{\varepsilon_c'} = \mu_c \tag{6.2}$$

3. Calculate H_c, the height of the collapse zone and divide into layers, ΔH_c

4. Estimate the average collapse, S_{AV}, as:

$$S_{AV} = \sum_{0}^{H_c} \Delta H_c \varepsilon_c' \tag{6.3}$$

The following sections detail how the proposed general procedure can be applied to selected specific conditions.

6.1 Case 1: Average settlement due to overburden

1. Calculate ε_c using equation 6.1
2. Convert ε_c to ε_c' using equation 6.2
3. Calculate the height of the collapse zone as:

$$H_c = \frac{H_a(1 - \frac{P_{IW}}{P_{ID}})}{2.5} \tag{6.4}$$

where H_a is the depth of wetting or the collapsible layer thickness, whichever is less. This zone is located at the bottom of the collapsing layer.

4. Estimate the average collapse due to overburden with equation 6.3.

6. 2 Case 2: Average settlement due to embankments

1. Calculate ε_c using equation 6.1
2. Convert ε_c to ε_c' using equation 6.2
3. Calculate the height of the collapse zone as:

$$H_c = \frac{H\ (1 - \frac{P_{IW}}{P_{ID}})}{2.5} \tag{6.5}$$

where

$$H = H_a + H_b \tag{6.6}$$

and where H_b is an equivalent height of the embankment calculated as:

$$H_b = \frac{H_E \gamma_E}{\gamma_c} \tag{6.7}$$

and where

H_E = The actual embankment height

γ_E = The unit weight of the embankment material

γ_c = The unit weight of the collapsible layer

This zone is located at the bottom of the collapsing layer.

4. Estimate the average collapse due to embankments with equation 6.3.

6.3 Case 3: Average settlement for to existing structures from foundation soils wetting

1. Calculate ε_c using equation 4.1

2. Convert ε_c to ε_c' using equation 6.2

3. Calculate the height of the collapse zone as:

$$H_c = 2.5B \tag{6.8}$$

where:

B = the foundation width

or

$$H_c = H_a \tag{6.9}$$

whichever is the smaller. This zone is at the top of the collapsing layer.

4. Estimate the average collapse due to embankments with equation 6.3

6.4 Case 4: Average settlement for structures constructed on wet collapsible soil

1. Calculate ε_c using equation 4.1

2. Convert ε_c to ε_c' using equation 6.2

3. Calculate the height of the collapse zone using equation 6.8

This zone is at the top of the collapsing layer

Compare this value to $H_a - H_c$ where H_c is calculated for overburden (equation 6.4) since the collapse that has already occurred due to overburden at the bottom of the collapsing layer should be discounted if the two collapse zones overlap.

The smaller of the two values should be taken as H_c for new structures on wetted soil.

4. Estimate the average collapse due to embankments with equation 6.3

Table 6.1 provides a quick reference to obtain the proposed procedure for the above mentioned cases.

Table 6.1: Proposed Procedures for Estimating Collapse in Silts

CASE NUMBER	**1** (Overburden)	**2** (Embankments)	**3** (Existing Structures)	**4** (New Structures)
PMT PROBE LOCATION	Lower portion of collapse layer	Lower portion of collapse layer	Upper portion of collapse layer	Upper portion of collapse layer
ε_c	Eq. 6.1	Eq. 6.1	Eq. 4.1	Eq. 4.1
ε_c'	Eq. 6.2	Eq. 6.2	Eq. 6.2	Eq. 6.2
$\mathcal{H}_c$	Eq. 6.4	Eq. 6.5 Eq. 6.6 Eq. 6.7	The smaller of Eq. 6.8 or Eq. 6.9	The smaller of Eq. 6.8 or H_a - Eq. 6.4
S_{AV}	Eq. 6.3	Eq. 6.3	Eq. 6.3	Equation 6.3

To verify the suggested procedures for Case 1, Case 3, and Case 4 the average collapse was estimated for the site at Nephi, Utah and these values were compared to the actual measured values. Loading of the foundations was limited to 85 kPa. Maximum settlements that occurred were nearly 1.5 times the average settlement in some cases, which could be due to the propagation of the moisture front, non-homogeneity of the soil, eccentric loading, or other factors. This emphasizes the importance of recognizing that the proposed method is for *average* collapse, not *maximum* collapse, and that differential collapse settlements can be extensive. Case 2 type loading was not conducted at the site. The results of the computations and the average measured values are summarized in Table 6.2.

Table 6.2: Comparison of Estimated and Measured Collapse at Nephi Test Site

Case #	Estimated Collapse (mm)	Measured Collapse (mm)
1	80	80
3	281	280
4	225	250

Conversion: 1 mm = 0.0393 in.

From this table it can be seen that the proposed procedure works very well for the silts at the Nephi test site. However, further testing at additional sites is necessary to verify the validity of the proposed procedure for use in other soil types.

6.5 Sample Calculation

Assuming a 15 ksf average stress increase due to an existing loaded 5 foot square foundation on a 20 foot collapsible layer and using the dry and wet PMT test pair for the site, the average collapse settlement can be predicted using the proposed method outlined for a Case 3 type loading condition as follows:

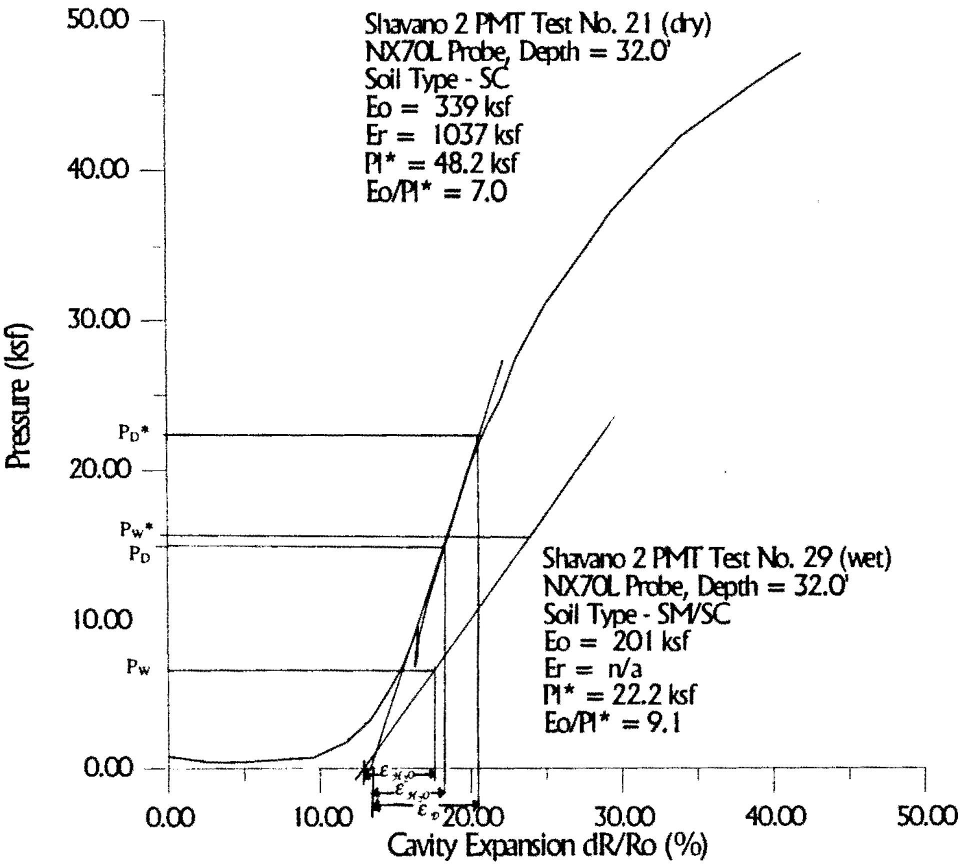

FIG. 6.1 -- Dry/Wet PMT Test Pair for Example Site

EXAMPLE

Assume: 15 ksf average increase in stress

5 foot square foundation

20 foot collapsible layer

$C = 0.115$

From Dry/Wet PMT test pair (Duquette data base)

$P_{ID} = 48.2$ ksf $\qquad P_D^* = 22$ ksf

$P_{IW} = 22.2$ ksf $\qquad P_W^* = 15.5$ ksf

$P_D = 15$ ksf

$P_W = 6.7$ ksf $\qquad \varepsilon_{\mathcal{D}} = 7.2\%$

Calculations:

$$\varepsilon_c = 0.115\left(\frac{48.2}{22.2}\right)\left(\frac{15.0 - 6.7}{22.0 - 15.5}\right)(7.2\%) = 2.3\% \qquad (4.1)$$

$$\varepsilon_c' = \frac{\varepsilon_c}{\mu_c} = \frac{2.3\%}{0.67} = 3.4\% \qquad (6.2)$$

$$\mathcal{H}_c = 2.5(5ft) = 12.5ft. \text{ (less than 20 ft.)} \qquad (6.8)$$

$$S_{AV} = 0.034(12.5\text{ft})\ (12\text{ in/ft}) = 5.1\text{ in.} \qquad (6.3)$$

7.0 CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

7.1 Conclusions

Observations of settlements in collapsible soils indicate that there is a component of settlement that is unique to these deposits, being neither immediate or consolidation type settlements that are associated with stable soils. For the collapse settlement to occur additional moisture must be introduced into the foundation soils. The collapse may take place under additional load or, in some instances, under overburden pressures alone. The collapse phenomenon requires an additional settlement component to be considered when estimating overall settlement in collapse prone soils. This has ramifications regarding testing methods, modeling and design approaches. Use of current methodology for developing approaches specific to collapsible soils can provide a basis on which to build.

7.1.1 Finite Element Modeling Analysis

Characterization of granular or cohesive behavior in collapsible deposits was not clear in reported tests. Thus, an analysis of a finite element model of an expanding probe was conducted to determine whether the collapsible silts at the Nephi site were behaving as a cohesive or frictional material. Non-dimensional numerical curve fitting of field tests by CAMFE and FENAIL revealed that the collapsible silts tested behaved primarily as a *frictional* material, under both dry and wet conditions. Non-dimensional plots of the field test results and other, primarily observational, evidence substantiate this finding.

One dimensional and two dimensional FENAIL FEM analyses was conducted to determine the zone of influence about the expanding probe. This analysis verified the that the influence zone extended a distance of approximately ten probe radii from the borehole wall, under small strain conditions. These results support the assumptions made in the recommended PMT testing procedures for wet conditions, where a wait time of 30 minutes is prescribed to allow for the saturation of a body of soil at least ten radii about the probe.

The assumption is that the two moisture fronts will coalesce for full saturation to ten radii along the full length of the probe. A FEM study was conducted to evaluate the effects of possible non-coalescence of the two moisture fronts, both from an inadequate volume of water and from non-standard moisture patterns. The results of this study emphasized the importance of assuring full coalescence of the moisture fronts. Substantially higher modulus values on the order of 3.5 times that of the fully coalesced moisture fronts were obtained for non-coalescence cases giving an inaccurate estimate of the wet properties of the soil.

7.1.2 Testing Methods

Existing testing and prediction methods for stable soils have been evaluated with respect to their applicability to collapsible deposits. It was concluded that the in-situ pressuremeter test, which creates a shear induced collapse, provided a reliable means for obtaining relevant soil properties for both dry and wet soil conditions. The PMT was particularly useful in testing the difficult to sample collapsible silts and coarser collapsible deposits that do not lend themselves well to traditional sampling and laboratory testing methods. Under pressure controlled testing conditions, a radial collapse strain can be obtained for a given pressure as moisture is added to the soil surrounding the pressuremeter probe. This collapse strain can also be identified from dry/wet PMT pairs and is an important property in the proposed collapse prediction method.

The suggested PMT testing method to obtain dry and wet test pairs and the interpretation is similar in concept to the double oedometer test that has been used with some success in fine grained deposits. The PMT method for obtaining radial collapse strain is also similar in concept to the modified consolidometer tests that have been attempted on collapsible soils. In both cases the PMT is found to be the superior method because the disturbance to the sensitive soils is minimized. The introduction of water about a partially expanded probe is made possible by a modification to the standard probe which allows water to pass to the surrounding soil from the top of the probe and from the bottom of the shoe. A volume of approximately 8 gallons (30 liters) of fluid is introduced into the borehole after the probe is seated and allowed to permeate the surrounding soil for at least 30 minutes before the wet test is begun. For dry/wet testing the same volume of water is introduced at a predetermined stress level and allowed to permeate the surrounding soil for the same 30 minutes before the secondary, or wet, portion of the test resumes. Air rotary drilling and hand augured reference holes are recommended to further minimize disturbance.

7.1.3 Settlement Approach

A basis for a collapse prediction method was selected from review of existing settlement methods and observation of settlement patterns measured at the Nephi test site. The basis of the method, which is founded in consolidation type settlement calculations, follows the observation that collapse occurred in zones that were subjected only to significant increases in spherical stresses. From settlement measurements recorded at the Nephi test site it appeared that zones of increased shear stresses did not undergo the same degree of collapse that the zones of increased spherical stresses experienced. This condition is closely represented by the consolidometer, so the basis of the proposed settlement method is similar in concept to the methods making use of consolidometer data, but adapted for the preferred method of in situ pressuremeter testing. Horizontal collapse strain measurements acquired from the PMT data and soil dependent constants are converted to vertical collapse strain values making use of a *soil strain ratio*. This is a redefinition of the classic mechanics Poisson's ratio. The vertical collapse strain is then used to calculate the collapse over the collapse zone. This method requires the determination of the collapse zone, as well as collapse soil constants that enable the estimation of the horizontal collapse strain from the dry/wet test pairs and the conversion of horizontal collapse strain to vertical collapse strain. The constants should be determined for each site, but generalizations may be made depending on soil type. The soil constants can be determined by performing a Procedure 3 type PMT test to determine the collapse strain at a known pressure level and using equation 4.1 to solve for the constant. For the collapsible silts at the Nephi site the proposed method

$$\varepsilon_c = C(\frac{P_{ID}}{P_{IW}})(\frac{P_D - P_W}{P_D * - P_W *})(\varepsilon_D) \quad (4.1)$$

predicted average collapse within 5%, but differential settlements may be extreme, and the method predicts *average* collapse only.

7.2 Recommendations for Further Research

7.2.1 Effects of Water Content and Foundation Geometry

It is acknowledged that the collapse phenomenon is dependent in part upon the degree of saturation, but how that effects the degree of collapse is unknown. It has been hypothesized that a "trigger" water content/pressure combination can induce collapse, but this has not been tested and the percentage of saturation and pressures necessary have not been measured. An alternate hypothesis suggests that the degree of collapse may be linearly dependent upon the saturation level, once collapse has begun. Presently all tests are conducted under assumed 100% saturation. To determine the effects of water content on collapse a method to test the soil and monitor water content simultaneously needs to be developed. For any FEM analysis, a constitutive model that contains the effects of water content also needs to be developed. Current constitutive models do not make use of any information regarding the saturation level of the soil.

Little has been done in the study of the effects of geometry and size of foundations on collapse. The observation of collapse occurring in zones of significant increases in spherical stresses should be verified with FEM analysis that vary the foundation parameters, and hence the zone of spherical stress increases.

7.2.2 Suggested Finite Element Constitutive Model Improvements

The constitutive soil models used by current codes are most applicable for cohesive soils that behave more closely to the solid mechanics based theory. For frictional materials, and especially the collapsible soils under study, the available soil models are inadequate. Models that incorporate the dilatancy and contractive effects that are particular to the frictional soils need to be developed. The changes in collapsible soil properties from the dry to the wet condition and the time dependency of such changes could be better represented by an improved time history function in the available FEM codes.

The present state of knowledge regarding the mechanics of particulate substances is an additional restriction to the development of a comprehensive constitutive model of frictional soils in general, and advancement in this arena is necessary before a truly representative model can be developed.

7.2.3 Behavior of Collapsible Soils in the Wet State

Non-dimensional plots of PMT data from the wetted collapsible silts showed an indication of repeated intermediate "mini-collapses" occurring as the pressure was increased up to limit pressure. This may be due to the frictional nature of the deposit, and with improved constitutive models it may be possible to study the behavior in depth.

Such study may give insight to the actual process of collapse and possible methods to control its magnitude.

7.2.4 Additional Field Studies

Verification of the proposed collapse settlement method should be achieved by conducting additional comprehensive field studies in varying soil types, in much the same manner as the study conducted at the Nephi site. Soil testing should include, at a minimum, a dry PMT test, and wet PMT test, and a dry/wet PMT test and soil sampling to establish the soil profile and for identification purposes. Soil constants should be calculated and settlements predicted at various pressures. Full scale foundations should then be loaded and settlements recorded with depth. Wetting of the soil should be done prior to loading at one location and after foundation loading at a second location. A third location should be wetted and settlements measured for information regarding settlements due to overburden. Additionally, settlement measurements under and within a constructed debris embankment on a wetted foundation should be made.

Several such tests performed in similar soil types may lead to soil dependent constants that can be used in the absence of site specific testing. Such constants for generalized soil types could be developed.

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