USE OF CENTURY CORN FOR MANUFACTURING  
TEXTILE AND FABRIC FIBER IN SUDAN

IV

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A Thesis submitted for the degree of  
M.Sc. (Eng.) in the University of Manchester.
The use of the static cone penetration test (S.P.T.) as a site investigation tool for classifying potentially expansive soil has been studied with the objective of presenting a field method for the classification of expansive soil.

The relationship between the S.P.T. parameters: cone resistance, $q_c$, skin friction, $f$, and friction ratio, $R_f$, as measured by a ten ton static cone penetration machine using an adhesion jacket cone and the small potential of the investigated soil have been analyzed. A method for classifying potentially expansive soil has been derived from the S.P.T. data. The method utilizes the parameters $q_c$, $f$, and $R_f$ for the proposed classification. Four small potential zones have been identified and proposed as a classification criterion. These are: non-expansive zone, low to medium expansive zone, high expansive zone, and very high expansive zone. Ranges of skin friction and friction ratio that define the boundaries between these zones were also identified.

In addition, the cone resistance was used in this study to define the consistency of the soil. Again ranges of the cone resistance, on the basis of a logarithmic correlation between the cone resistance and relative consistency of the soil were obtained to enable an estimate of the consistency of the soil to be calculated from a knowledge of the cone resistance, $q_c$. 
Chapter Four
Scope of work and site geology
4.1 Scope of work
4.1.1 Aim
4.1.2 Equipment used
4.1.2.1 Static penetrometer machine
4.1.2.2 Drilling rig used

4.2 Geology of the studied sites
4.2.1 Jungled coastal area
4.2.2 Harbour area

4.3 Field and laboratory tests
4.3.1 Field tests
4.3.2 Laboratory tests

Chapter Five
Results and Discussions
5.1 Classification of expansive soil from static CPT results
5.1.1 Introduction
5.1.2 Experimental results
5.1.3 Analysis of results
5.1.4 Discussion

5.2 Effect of moisture variation on the friction ratio

5.3 Relationship between consistency and cone resistance
5.3.1 Introduction
5.3.2 Field and laboratory test results
5.3.3 Analysis of results
5.3.4 Discussion of results

Chapter Six
Conclusions and recommendations
6.1 Conclusions
6.1.1 Classification of expansive soil
6.1.2 Variation of relative consistency with cone resistance
2.1 The Dutch pocket penetrometer
2.2 Original cone shape
2.3 Rustic cone
2.4 Adhesion jacket cone
2.5 10 ton Dutch static CPT machine
2.6 Ground anchor scanner
2.7 Ground anchoring weights
2.8 Connection of sounding tubes to the adhesion jacket cone
2.9 Mode of operation of the adhesion jacket cone
2.10 Typical observation form with computed $F_o(L/hr)$
2.11 Description of rows of Fig. 10
2.12 Graphical representation of Fig. 10
2.13 Relationship of cone bearing, local friction
2.14 Schmertmann's soil classification curves
2.15 Combined $e_o$, $W_s$ soil type plot
2.16 Variation of the constants $a$ and $b$ with PF & LL
3.1 Distribution of reported instances of heave
3.2 Mean annual rainfall in Sudan
3.3 Clay phase of soils.............................................
3.4 Determination of potential expansiveness of soils.............................................
3.5 South American expansive clay sediments classification chart..........................
3.6 Classification chart of swelling potential.............................................
3.7 Relation of volume change to soil content, plasticity index and shrinkage limit..........
3.8 Relationship of volume change to plasticity index.............................................
3.9 Percentage of sand against percentage of clay size.............................................
4.1 Rotary drilling rig used.............................................
4.2 Location of study area on country map.............................................
5.1 Applicability of a proposed chart for classification of 27 natural soils..................
5.2 Distribution of the experimental points in the classification chart..........................
5.3a Plot of \( a_1 \), \( a_2 \) with symbols showing the swell potential of the investigated soil.
5.3b Swell potential classification zones.............................................
5.4 Static OMC results before addition of water.............................................
5.5 Static CPT results after addition of water

5.6 Variation of friction ratio with depth for the static CPT's shown in Fig. 5.4 and Fig. 5.5

5.7 Distribution of Jangkei and Baradia soils in Casagrande plasticity chart

5.8 Variation of relative consistency with the core resistance
Table No.

2.1
Consistency of clays as a function of:
- Number of blows if on sampling spoon and
- Undrained shearing strength......................... 36

2.2
Estimates of consistency index from
cone resistance........................................... 37

3.1
Classification of expansive soils in terms
of shrinkage index...................................... 38

3.2
Data for making estimates of probable volume
changes for expansive soils............................ 54

3.3
Data for making estimates of probable volume
changes for expansive soils............................ 57

3.4
Soil potential in terms of soil sample
properties as predicted by soil author............. 58

3.5
Summary of expedient methods for identification
of expansive soils..................................... 61

3.6
Data for values are of the degree of soil
potential from static CPT results................... 32

5.1
Estimates of consistency from cone resistance
value.................................................... 100
... Activity
C Clay content
C.R. Relative consistency
C.P.T. Cone penetration test
f_s Total resistance
f_k Skin friction
I.M.C. Initial moisture content
L.L. Liquid limit
M.C. Moisture content
N Standard penetration resistance
P.I. Plasticity index
P.L. Plastic limit
r_c Cone resistance
R.D. Relative density
r_f Friction ratio
S.I. Shrinkage Index
S.L. Shrinkage Limit
S.P. Swell potential
 Specific gravity
S.P.E. Standard penetration test
W Water content
INTRODUCTION

1.1 Site Investigation:
Site Investigation means technical investigation by which the necessary information is obtained about geological, hydrological, and soil conditions; as well as information about geotechnical properties of soil at the prospective building site and the performance of the various soil types encountered when acted upon by structural and applied loads, water and temperature (Juminis, 1963). The field and laboratory investigations to obtain the required information are thus termed soil or site investigation. This information is necessary as a background upon which the design of a structure is based.

1.2 Extent of Site Investigation:
For a new structure the soil information needed should provide data to enable the assessment of the following:
1) Location of ground water level.
2) Bearing capacity of the soil.
3) Depth and/or type of foundation.
4) Amount and rate of settlement.
5) Other soil parameters and properties.
The extent of the work required to obtain the
above information depends upon the importance
and foundation arrangements of the structure as
well as the complexity of the soil conditions.

Subsoils were formed by geologic processes that
change at random in space and time with a decisive
influence on the sequence, shape and continuity of
the soil strata. Hence a standard spacing between
exploratory drillholes is not possible and the
spacing is still governed by convention and not
by rational consideration as stated by Terrazghi
and Peck (1967).

The first step in any subsoil exploration is the
investigation of the general character of the site
which permits a more efficient program for soil
exploration to be laid out. The second step is to
make a few exploratory drillholes that furnish more
specific information regarding the general character
of the soil profile. The number of drillholes
required increases with increasing complexity of the
soil profile and the size of job. For erratic soil
profiles the effort is usually concentrated on
obtaining reliable information regarding the struc-
tural pattern of the subsoil rather than obtaining
accurate data about the physical properties of
individual soil samples. The amount of money spent
in soil investigation usually ranges between 1 to
3% of the total cost of the project and a value of the order of 1% is usually accepted (S.C. CP (1991), 1997).

This amount of money is not to be considered wasted, but an intelligently laid out programme for soil exploration may reduce this cost to the minimum and this should be the aim of practicing soil engineers.

1.3 Methods of Site Investigation:

Every subsurface exploration should be preceded by a review of all available information concerning the geological and subsurface conditions at or near the site. In most cases these informations must be supplemented by the results of a more direct investigation. That will, of course, lead to direct contact with the soil at different depths to be made. Many methods to do this exist. These methods can generally be divided into two categories: the first one the soil is tested in situ, where the test equipments are transferred to the site. In the second one soil samples representing the soil profile at different depths are transported to soil laboratories where they are tested. In situ tests are usually considered more reliable since they measure the properties of soil in its natural conditions. The accuracy of laboratory results, on the other hand, depends on the degree of disturbance the
soil samples receive during sampling, transportation and extraction processes. Geophysical, vane shear, plate loading tests and some of the deep sounding methods are examples of insitu tests of soil where no soil samples are recovered. Auger boring, rotary drilling, percussion drilling and test pits are examples of exploration methods where disturbed and/or undisturbed soil samples are taken and transported to laboratories where they are tested and their properties assessed accordingly (Bowes, 1982).

1.4 Planning of Site Investigation Programme:

In view of the available methods of subsurface exploration and the useful information that can be derived from the site investigation, the soil engineer can lay out his programme. The programme generally includes none or all of the following steps:

1) Assessment of all available information on dimension, type and use of the structure and any special characteristics of the proposed building.

2) Reconnaissance of the area.

3) A preliminary site investigation in the form of few borings, test pits or in situ tests.
(3) A detailed site investigation for large projects or when the soil profile is erratic.

However, when planning such a programme, the engineer should be well acquainted with the current methods of subsurface exploration and have some idea of their limitations.

1.5 **The Static Cone Penetration Test (S.C.P.T.)**

The static cone penetration test is considered to be one of the deep sounding methods which with some adaptation can also be used to retrieve soil samples.

Recently the static S.C.P.T. has gained wide acceptance as a good and reliable in situ site investigation test giving a fairly idea about soil type and properties. These properties include strength, skin friction and settlement assessment. In a number of cases its data is used to decide on pile length, soil density and strength.

The Building and Roads Research Institute (BRRI) has introduced adequate experiences with the S.C.P.T. machine. Ooi (1983) has used the machine to assess soil type, density and shear strength. However, no experience with the use of the machine to assess expansive soil properties has been reported. In this study experiments made to study and evaluate some properties of expansive soil in Sudan are
The research is mainly dedicated to the possibility of finding tests with which one can identify potentially expansive soils and to estimate their degree of expansion. Part of the work described in this text is taken from the results of the site investigation reports of Jungiel Creek project carried out by Belfast Soil Mechanics Laboratory (B.S.M.L.) in collaboration with B.M.I. (1970). Arsenic reporting expansive soil damages in Newtons are also investigated and discussed in this study. The main study has been made by making static cone penetration tests. In addition classification tests were conducted on soil samples collected from near the C.E.P. locations. An Aber machine, model AB II rotary type drilling equipment was used to take disturbed and undisturbed soil samples for this purpose.
2.1 Introduction:

The origin of penetration testing goes back to 1946 when a method was developed by Collin in France. He used a needle of 1 mm diameter and weighing 1 kg to estimate the cohesion of different types of clay of varying consistency. Since then continuous development both in penetration equipments and refinements in measuring techniques have evolved.

There are mainly two types of penetrometers used in the field of soil exploration; these are the static and dynamic penetrometers. Each one is distinguished by the way in which the penetrating rods are driven into the soil. In the static cone penetration test (CPT) a cone is pushed into the soil at a constant rate of penetration, whereas in the dynamic cone the cone is driven by an impact of a standard hammer.

Large variations exist for both types with respect to the design and test procedure. Sagnier (1973) described about forty different penetrometers of the static, dynamic and static-dynamic types developed in various countries. In the following sections only a description of a recent variety of static penetrometers is given.
3.4 History and Development:

Static penetrometers are either hand operated or mechanized penetrometers. Both types are still used to evaluate soil properties.

Hand-operated penetrometers were firstly introduced in 1917 when the Swedish Railroad standardized a method of sounding. This method consists of pushing a metal rod, 19mm in dia., into the soil by applying loads of 5, 15, 25, 50, 75 and 100 kg. When refusal is encountered with a load of 100kg the rods are rotated either manually or by machine to advance the rods further (Bengtson, 1972).

In 1931, the Danish Railroad developed a pocket penetrometer, shown in fig. (2.1), and has been found to be satisfactory for evaluation of cohesion and allowable bearing pressure of soil.

In 1946 Heft Soil Mechanics Laboratory (D.S.M.L) in conjunction with Goudsche Machine-Fabriek manufactured a 2000kg hand-operated penetrometer.

Later in 1948 a 10000kg hand-operated penetrometer was made available by the same producers. Both of these penetrometers have been used since 1936 and 1935, respectively, by D.S.M.L. (Bengtson, 1974).
Fig. 2.1 THE DANISH POCKET PENETROMETER
However, hand-operated penetrometers which are pressed into the soil by weights or rotation are not suitable for testing soil at great depths as required. As a consequence, mechanized penetrometers emerged and appeared almost simultaneously and subsequently replaced the hand-operated ones.

Mechanized penetrometers are usually hydraulically operated and are supplied with an internal combustion engine to drive the hydraulic pumps.

Perhaps the first mechanized static penetration device was the 'Drum Ringer' sampler developed in 1933 to determine the characteristics of a glacial clay deposit (Sanger, 1972). The device consists of a central head, rods and casings pressed into the soil by means of hydraulic jack.

In 1939 geotechnical machine manufacturer in cooperation with C.G.E.L. introduced a motorized 10-ton capacity penetrometer. This penetrometer became most popular and was extensively used in various countries.

Again, in 1964 an extraneous penetrometer of 20-ton capacity was made available by the same manufacturer. This penetrometer can be rotated in a horizontal plane to ease penetration into highly resistant layers.
In the course of time a number of penetrometers have been developed, that differ in size, shape and mode of operation.

A cone whose projected area is ten square centimetres \( (10\text{cm}^2) \) and having an apex angle of sixty degrees \( (60^\circ) \) is generally accepted as standard. This standard has been specified in the European and American standards (ISRM 1977, ASTM 1971). A friction sleeve is often incorporated in the cone assembly. This is located above the conical tip, and has a standard area of one hundred and fifty square centimetres \( (150\text{cm}^2) \). When tests are made in the field, the cone resistance, \( q_c \), and the sleeve friction, \( q_s \), are measured. The ratio of \( q_s \) to \( q_c \) is termed the friction ratio, \( R_f \). This ratio is used to identify soil types (Segalman 1965, Schmertman 1975, Neubauer 1975, Eini 1982).

2.3 Mechanical Penetrometers:

The types of mechanical cones generally used are those originally developed in Holland. Brinell's plastic penetrometer appeared in Holland in 1943. He used a 60° apex angle, 10cm² base area cone in which the base diameter is the same as that of the casing, see Fig. (2.4). This cone is known as the original or simple cone. To prevent soil particles from entering in the annular space between the casing and the rod, the so-called "double cone" was
Mechanical penetrometers have been compared with electronic penetrometers (De Ruiter, 1982). It is observed that the friction penetrometer sleeve sleeve can have independently from the tip, provides a discontinuous measurement of cone resistance $q_c$ and sleeve friction $f_s$ by telescoping successively the conical tip and the sleeve ahead of the stationary sounding tube. The friction penetrometer as designed by Bagamon does not completely separate friction and end bearing in the measurement of $q_c$ and $f_s$. Depending on soil type and consistency, some friction can develop on the short tapered mantle behind the cone, whereas the lower edge of the friction sleeve can encounter cone end resistance. Furthermore, the calculation of friction ratio ($f_s/q_c$) suffers of a potential lack of precision, because the two measurements are
Fig. 2.2 ORIGINAL CONE SHAPE

Fig. 2.3 MANTLE CONE

Fig. 2.4 ADHESION JACKET CONE
and taken at exactly the same level in the soil. The difference in level is 30 cm, which can be
serious in layered deposits. So, in rather homogeneous-cohesive soils, without sharp variations
in cone resistance, the results can be fully adequate, provided that the equipment is properly
maintained and the operator has the required experience. Nevertheless the quality of the data
remains such that operator dependent than with an
electric penetrometer. In soft soils the accuracy
of the results is definitely inadequate for a quanti-
tative analysis of the soil properties. In highly
stratified materials even a satisfactory qualitative
interpretation is impossible (Joutter 1962).

However, these mechanical penetrometers find their
main advantage in the simplicity of the operation
and the low cost of tips and equipment. The main
criticism being in soft, highly stratified soils;
however, it remains reasonable to use the static
CPT in other types of soils. 
The use of this
type of penetrometer in Japanese soils is justified
by the fact that most Japanese soils are not soft,
and furthermore the depths of individual layers as
observed in borehole logs are of such sufficient
thicknesses as to permit reasonable interpretation
of static CPT results.
2.3 Objective of Test:

The static C.P.T. has the primary objective of measuring the resistance of the soil to the advancement of the cone. After the introduction of the adhesion jacket cone by Bygott, two independent measurements are made possible:

1) The cone resistance \( q_c \), which is defined as the mean vertical stress on a cone (usually 10cm² base area, 60° apex angle) that has to be exerted on this cone to push it into the soil at a constant rate of penetration (2 cm/sec).

2) The skin friction \( f_s \), which is defined as the mean frictional stress exerted by the surrounding soil on a cylindrical sleeve (10cm²) located above the tip of the cone when this sleeve is advanced into the soil at a constant rate (2cm/sec).

3) Friction ratio, computed from these two measurements, the friction ratio, \( R_f \) is defined by: \( R_f = \frac{f_s}{q_c} \times 100 \).
The rig used for pushing down the cone consists basically of a hydraulic jacking system. The thrust capacity of the machine for cone testing varies between 10 to 20 metric tons. 20 tons is about the maximum allowable thrust on the 37.7mm diameter high tensile steel push rods. Exceeding this load would result in buckling of rods (De Maiter, 1969). Screw anchors are used to develop the reaction required to balance the downward thrust. The most time consuming part of the test operation is the setting of these anchors to provide the reaction.

In 1969 Goodeve Macinefabrik in cooperation with B.S.M.I. introduced a 10 tons capacity penetrometer. This penetrometer has gained wide popularity and is being used in various countries. Different models of this penetrometer exist with the basic device that have the same function being identical (B.S.M.I. Catalogue 1969). This penetrometer has been used in the present study. In addition a model ADI Anchor auger drilling machine was used. The C.P.T. machine used has a two stroke, 7 h.p. engine and weighs about 750kg. Fig. (2.5) shows a view of the machine. The machine is usually supplied with the following replacement parts:

1. Pressure gauge:
   These are provided to measure either the cone resistance and skin friction if an adhesion
Fig(25) 10 Ton Dutch Static CPT Machine
(ii) **Sounding tubes**: 

In long, 36cm O.D. and 12cm I.D. These tubes are screwed together for making up the required length of penetration. They also act as a casing to ensure vertical penetration and to protect the push rods from soil friction.

(iii) **Pressure steel rods**: 

In long and 19mm diameter. These rods are used to push the cone while the sounding tubes are kept stationary. The rods stuck end-to-end inside the sounding tubes and bear directly on the base of the cone.

(iv) **Wetted cone**: 

See Fig. (2.3). 

(v) **Asbestos jacket collar**: 

See Fig. (2.4).
(vi) **Ground Ankered Anchor:**

The machine is also supplied with a reversible torque hydraulic ground anchor opener operated by the same engine. This opener is used to drive the anchoring auger into the ground and also to drive them out upon completion of the test. Fig. (2.5) shows a view of this apparatus.

(vii) **Ground Anchoring Auger:**

These augers are supplied with different blade diameters to suit different soil conditions. In all there are four ground augers used for normal ground conditions. In special cases core augers can be used, see Fig. (2.7).

### 2.7 Execution of the Test:

The procedure followed in performing the static C.P.I.L. using the 10tons capacity machine is often referred to as the mechanical or intermittent procedure. The machine is first centred over the testing location and then anchored to the ground by the anchoring augers. Either of the cone tips are normally used in the test; the mechanical mantle cone or the mechanical adhesion jacket cone developed by I.S.R.C. The adhesion jacket cone was used throughout this research programme and hence the test procedure concerned with this type only will be discussed here.
Fig(2.6) Ground Anchor Spanner
The top portion of the cone is inserted into the sounding tube such that the inner rods when dropped into the sounding tube will bear directly on the base of the cone. The moving hydraulic in a containing measuring gauges is lowered until it is possible to connect the sounding tubes to the head, see Fig. (2.0). The sequence of the test is then made in the following manner:

a) The sounding tubes and the cone are pushed to the depth at which the soil resistance is the advancement of the cone is required, Fig. (2.0 a). In this step the Bourdon gauges measure the total resistance of the soil to the movement of the sounding tubes, tip of the cone and the friction sleeve.

b) The tip of the cone only is then pushed through the inner rods a distance of 30 cm and the cone resistance is noted on the gauges, Fig. (2.0 b).

c) After the tip of the cone moves a distance of 30 cm, the friction sleeve is automatically engaged and the whole assembly is advanced another 40 cm, Fig. (2.0 c). This is done in one continuous operation showing a jump of the gauge pointer that the sleeve has engaged. The jump measures the resistance of the soil to sliding of the friction sleeve.
Fig. 2.8 CONNECTION OF SOUNDING TUBES TO THE ADHESION JACKET CONE
Fig. 2.9 MODE OF OPERATION OF THE ADHESION JACKET CONE
The tests are usually carried out at a constant rate of penetration of 2cm/sec and records of soil resistance are taken at 0.2m intervals.

2.8 Presentation of Results:

2.8.1 Registration form:

The tests would be made in a special tabular form as shown in Fig. (2.10). In this figure the observed data are recorded with depth in metres below the ground level. Three compartments a, b and c are constructed to register the readings of the bourdon gauges at each depth, see Fig. (2.11). These compartments give the values of the total resistance \( f_L \), (a) the cone resistance \( f_C \), (b) and the cone resistance plus the skin friction \( (f_L + f_C) \), (c) respectively. In the same observation form information about the type of cone used, the project name, place of test, date of test, and if known, the elevation of the ground surface with respect to a given datum are recorded.
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<td>30</td>
<td>4.0</td>
<td>7.0</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Fig. 2.10 TYPICAL OBSERVATION FORM: WITH COMPUTED $f_0 (kg/cm^2)$
<table>
<thead>
<tr>
<th>depth (m)</th>
<th>0.20</th>
<th>0.40</th>
<th>0.60</th>
<th>0.80</th>
<th>1.00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 2.11: DISCRIPTION OF ROW OF Fig. 2.10
2.6 Interpretation of C.P.T. Results:

The static C.P.T. was introduced in Holland in the early thirties and since then interest in this test is still on the increase. In recent years the static C.P.T. has gained popularity rapidly all over the world because of its added advantages over other contemporary field test used for similar purposes. The test is extremely useful in site investigation and gives qualitative and quantitative guidance to the in-situ engineering properties of soils. It has also been found to be of value for pile design. Important researches reported valuable...
Fig. 12: Graphical representation of data in Fig. 10.

Laboratorium voor Grondmechanica te Delft
information regarding the use of the C.P.I. results. A large number of publications are available. References are, Sanger and Smith, 1972, SCENR, 1975, SANG, 1982.

The following types of information could be derived from static C.P.I. results.

2.9.1 Soil classification.
2.9.2 Consistency of cohesive soils.
2.9.3 Relative density and friction angle.
2.9.4 Compressibility of clay.
2.9.5 Pile bearing capacity.

2.9.1 Soil Classification:

Soil classification based on grain size analysis are widely used by engineers, especially for preliminary or general description of soils. However, grain size analysis alone does not reflect a complete classification for fine grained soils. Hence classification of mixed soils containing both coarse and fine fractions, which is the case for most natural soils, have been based not only on grain sizes but with the help of other characteristics such as plasticity.
Soils are also described as cohesive or cohesionless with their consistency and relative density being measured respectively.

The cone penetration test was firstly used in cohesionless soils. However, during the last few years, the C.P.T. has increasingly been used to investigate cohesive soils as well.

It has been reported by many authors, e.g., Bagusmann (1962), Schmertmann (1969), Sanglerat (1972), (1973) that from the results of the C.P.T. & preliminary soil classification is possible if a special sleeve is provided above the base of the cone. The measurement of the resistance of soil received by this sleeve permits soil classification.

It was also found that the value of the cone resistance, $q_c$ of the sleeve friction, $f_s$, does not reflect the type of soil penetrated. It was therefore suggested that another parameter should be introduced. This parameter is the friction ratio defined by:

$$ R_f = \frac{f_s}{q_c} \times 100 $$

Using one of this parameter coupled with $q_c$ and $f_s$, Bagusmann (1963) proposed his classification chart shown in Fig. 2.13. Schmertmann (1969) also suggested the classification chart
shown in Eq. (2.14). This chart indicates the soil type and its consistency or density if the cone resistance \( q_c \) and the friction ratio \( R_f \) are available. Sundaram et al. (1975) found that values of friction ratio less than \( 2/3 \) are typical of cohesionless soils and those above \( 5/6 \) or \( 6/7 \) are typical of clays. Values between these values of friction ratio are typical of mixture of sand, clay and silt.

Zolin (1980) arrived at the same conclusions in his application of the static C.R.I. to cohesive soils. Using a statistical method known as discriminant analysis, he defined boundaries of soil type as shown in Eq. (2.15).

\[
R_f = \frac{E_i - H}{P.I.} \quad (\%)
\]

where:

2.3.2 Consistency of Cohesive Soils:

By consistency is meant the relative ease with which soil can be deformed (Funsho, 1983). It defines the firmness of the soil which may be termed as soft, stiff or hard.

Wadadeh (1996) defines the relative consistency by:

\[
\eta = \frac{E_i - H}{P.I.} \quad (\%)
\]

where:
Fig. 2.14: SCHMERTMANN'S SOIL CLASSIFICATION CURVES
Fig. 215 COMBINED $q_c - R_f$ SOIL TYPE PLOT
or a relative consistency
L.L. = Liquid limit
W = Water content
P.I. = Plasticity index

According to this definition the relative consistency of the soil equals to one if the water content equals to the plastic limit and it is equal to zero when the water content equals to the liquid limit.

Tomaghi and Becht (1957) used the unconfined compressive strength \( c_u \) and the standard penetrative test (S.P.T.) N value, to estimate the consistency of the soil. They were able to define ranges of the consistency in terms of \( c_u \) and N as shown in Table (2.1).

Table: 2.1

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Very soft</th>
<th>Soft</th>
<th>Medium</th>
<th>stiff</th>
<th>very stiff</th>
<th>hard</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N )</td>
<td>&lt; 2</td>
<td>2-4</td>
<td>4-6</td>
<td>6-15</td>
<td>15-30</td>
<td>&gt;30</td>
</tr>
<tr>
<td>( c_u, kN/m^2 )</td>
<td>&lt; 15</td>
<td>15-25</td>
<td>25-50</td>
<td>&gt;50</td>
<td>&gt;100</td>
<td>&gt;100</td>
</tr>
</tbody>
</table>

Some researchers studied the dependence between the soil consistency, soil plasticity and the static S.P.T. parameters.
Sanger et al. (1975) used the friction ratio to distinguish between cohesive soils of high and low plasticity.

Steinsoff and Bejkooff (1974) used the cone resistance to define ranges of consistency index \( I_c \) (\( I_c = \frac{L_1 - L_2}{L_3} \)). They found that the consistency of cohesive soils can be determined with the aid of Table 2.7 below.

<table>
<thead>
<tr>
<th>Cone Resistance ( q_c ) (kg/cm²)</th>
<th>Consistency Index ( I_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0 - 10</td>
<td>0.25 - 0.5</td>
</tr>
<tr>
<td>10 - 15</td>
<td>0.5 - 0.75</td>
</tr>
<tr>
<td>15 - 30</td>
<td>0.75 - 1.00</td>
</tr>
<tr>
<td>30 - 60</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>&gt; 1.00</td>
</tr>
</tbody>
</table>

In USSR, Ziminetskoy (1974) also used the liquidity index and correlated it to the cone resistance. Based on 907 experimental points, he found that the liquidity index \( L_1 \) can be written in terms of \( q_c \) as follows:

\[
L_1 = 0.65 + 0.013 \times q_c \tag{2.2}
\]
Thus the liquidity index can be determined with the help of the static cone resistance only.

Rocke (1972) correlated the cone resistance \( C_0 \) with the relative consistency \( Q_p \) of the soil as defined by Marschik in eq. (3.1). He found that relative consistency can be determined from the cone resistance using a mathematical equation in the form:

\[
\log Q_p = a + b \log C_0 \tag{3.2}
\]

where \( a \) and \( b \) are constants. This equation takes into account the type of cohesive soil being examined.

Furthermore he stated that a more general valid function can be established if \( a \) and \( b \) are taken as additional variables. He found that \( a \) and \( b \) vary with the plasticity index (P.I.) and liquid limit (L.L.) respectively, as shown in Eq. (3.16).

Eqs. 2.9.4, 2.9.5 and 2.9.6 are out of the scope of this research program and hence no revision is made here about them.
Fig. 2.16 VARIATION OF THE CONSTANTS a, b WITH P.I. & L.L.
3.1 Origin of expansive soils

Rendleman (1969) classified the parent materials that can be associated with expansive soils into two groups: the first group comprises the basic igneous rocks such as basalt, dolomite, and gabbro. The second group comprises the sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils. The montmorillonite was probably formed from either the products of weathering and erosion of the rocks in the highlands or volcanic eruptions, sending up clouds of ash fall on the plains and sea. These ashes were altered to montmorillonite.

Potentially expansive soils, accordingly can be found to exist almost anywhere in the world. Chen (1975). Fig 3.1 shows some of the areas where expansive soil exists.

3.2 Distribution of expansive soil in Cebu

Cebu is the largest island country situated in the tropical zone between latitudes 23°N and 10°S, thus occupying an area of about 3.6 million square kilometers. In going from north to south one passes
Fig. 3.1 DISTRIBUTION OF REPORTED INSTANCES OF HEAVE
EXTENDED FROM CHEN (1995)
through different climates, ranging from a totally desert climate in the north to an equatorial climate in the south, Omer (1981). Between these two extremes, all types of intermediate climates are observed : subdesertic, dry and subtropical humid climates with a long dry season. The dry time temperature is high during most of the year with an average daily temperature ranging between 10°C to 15°C in the north. The mean annual precipitation ranges between 25cm to 1500cm as shown in Fig. (1.3).

Potential evapotranspiration (PET) is very high throughout Sudan increasing from north to south and appreciably equals precipitation in the south indicating a potential for a net soil moisture deficit, (Omer and Charlie (1981)).

Darwin (1964) reported that Sudanese expansive soils cover one million square kilometers. This area includes most of the population centers and development projects. Areas reporting expansive soil changes in Sudan are shown in Fig. (1.1). The area called the clay plain of Sudan, in central Sudan, consists of alluvial soils containing montmorillonite, (Omer and Charlie (1984)). The northern clay plain soils, known as black cotton soils, are thought to be weathered soil mantle derived from the volcanic
Fig. 32 MEAN ANNUAL RAINFALL IN SUDAN

FROM MELSON, et al., 1973

ISOBARS IN MILLIMETERS
Fig. 3.3 CLAY PLAIN OF SUDAN
AFTER GOSNALL & CHARUE 1984

- Area reporting expansive soil damage
- The clay plain
Chilonea Highlands but the southern clay plain soils are of residual type. The clay plain generally exhibit expansive soil properties and contain about 40% montmorillonite, Oman and Charlie (1984).

World wide the amount of damage caused each year by expansive soils are well documented. The severity of this damage is most evident and plentiful in lightly loaded structures. In Oman a lot of damage occurs due to the expansion of soil. For example, a technical committee recommended the rebuilding of a 30 million pounds sugar factory which was built on expansive soil, (Oman & Charlie 1984).

The annual estimated damages exceed 6 million dollars (Oman and Charlie 1984). A survey made recently by Oman and Marmot (1984) on lightly loaded structures in Oman revealed that the improper drainage systems are the major cause of soil expansion. Cracks developed due to this expansion were found to be consistent with the classification made by ESC and that the cracked area increases with time.

3.3 Recognition of Expansive Soil:

Expansive soils apply to those soils which have the capacity to undergo volumetric changes when subjected to variations in water content. When the water content is increased, the soil will swell; likewise, a decrease in moisture content will facilitate shrinkage. This
type of soil has received more attention recently due to the many problems that it causes. The necessity of recognizing swelling soils has become a very important part of the soil engineer's work. It is important to detect the pressure of swelling soils and to obtain an idea of the amount of swelling that is to be expected from them. Many methods and techniques for expansive soil identification and classification have been proposed by researchers. These methods and techniques range from visual inspection of the soil in-situ to sophisticated methods of testing the soil in the laboratories.

Many researchers pointed out that the occurrence of blistering surfaces as well as surface cracks as noted in the field is an indication of expansive soils.

In his address lecture delivered in the First National Conference on the Science and Technology of Building held in Sudan, Burland (1964) directed soil engineers to visit the site and examine the soil upon which the structure is to be laid. He appreciated the signs that are given by nature for tackling expansive soil problem. He stated that during the dry spells the presence of prominent shrinkage cracks in the ground surface are clear indicators of active clay soil.
Adib (1983) pointed out that dissected or shattered clay profiles coupled with a low water table is a sure indication of potential expansiveness of the soil. This is field evidence which can be obtained by observation and simple site investigation.

Goss (1968) put a general characterization of expansive soil in Sudan based on visual inspection as well as laboratory tests. He defined expansive soils as being graphic to black soils with high clay content, high values of liquid limit (L.L.) and plasticity index (P.I.) (L.L. over 35% and P.I. over 25%), very low permeability, and with montmorillonite as a predominant mineral responsible for volume change.

Drisdell (1984) could identify expansive soils by their physical features. He showed that expansive soils have a smooth, greasy, plastic feel when wet, and a hard, brittle, cracked appearance when dry. He furthermore stated that to distinguish a soil with a very high clay content and with the same expansive clay minerals from another soil with much less potential expansiveness, may not be easy by visual inspection alone; a better prediction of the ability of a clay soil to change volume can be obtained from simple soil tests such as Atterberg limits and the quantity of clay particles in a soil.
Sample classification chart based on the relationship between the plasticity index of the whole sample and the percentage clay fraction was developed by Van Der Merwe in 1964. This chart is shown in Fig. (3.4). The chart was used successfully to classify highly decomposed soils in South Africa into very high, high, medium and low degree of potential expansiveness, Van Der Merwe (1964). This method, however, has been found to be less satisfactory when applied to other soils and climatic conditions both in South Africa and in other parts of the World (Williams and Donaldson, 1980). A further modification to Der Merwe's diagram was developed by Williams and Donaldson, catering for the sharp transitions from the low expansive category to the highly or very highly expansive categories on the classification diagram at higher clay contents. The modified classification diagram is shown in Fig. (3.5).

A rational method of predicting swelling potential for some of the expansives soils was proposed by Ranganathan and Jothimurugan (1965). The proposed method utilizes the shrinkage index (liquid limit - shrinkage limit) and swelling activity (change in shrinkage index/corresponding change in clay fraction) for such classification. The procedure adopted in this study is similar to that of Seed et al (1965).
Fig. 3.4 DETERMINATION OF POTENTIAL EXPANSIVENESS OF SOILS

After Van der Merwe, 1964
Fig. 3.5 SOUTH AFRICAN EXPANSIVE CLAY MODIFIED CLASSIFICATION CHART

AFTER WILLIAMS AND DONALDSON, 1940
with the plasticity index being replaced by the shrinkage index. A classification which is solely dependent on the shrinkage index was developed. This classification is shown in Table 3.1.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Shrinkage index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt; 20</td>
</tr>
<tr>
<td>Medium</td>
<td>20 - 30</td>
</tr>
<tr>
<td>High</td>
<td>30 - 60</td>
</tr>
<tr>
<td>Very high</td>
<td>&gt; 60</td>
</tr>
</tbody>
</table>

Dilworth and Earn (1973) used the plasticity chart of Casagrande as a basis for expansive soil classification. The liquid limit plotted along the x-axis and the plasticity index and the shrinkage index on the y-axis to the left and right hand side of the chart, were utilized in such classification. The chart was divided into six zones along the x-axis with their liquid limit as follows:

<table>
<thead>
<tr>
<th>Liquid limit</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 20</td>
<td>None swelling</td>
</tr>
<tr>
<td>20 - 35</td>
<td>Low swelling</td>
</tr>
<tr>
<td>35 - 40</td>
<td>Medium swelling</td>
</tr>
<tr>
<td>50 - 70</td>
<td>High swelling</td>
</tr>
<tr>
<td>70 - 90</td>
<td>Very high swelling</td>
</tr>
<tr>
<td>&gt; 90</td>
<td>Extra high swelling</td>
</tr>
</tbody>
</table>
Chen (1975) described three different methods for classifying potentially expansive soils. The first method is the micromorphological identification which is useful in research work to explore the basic properties of clays, but it is impractical for practicing engineers when dealing with natural soils. The second method uses indirect tests such as index properties, potential volume change (PVC) and activity which are valuable tools in evaluating swelling properties. The third method is based on direct measurement of the swelling properties of expansive soil in the laboratory, and offers the most useful data for design purposes.

Seed, Woodard and Esligeon (1962) proposed the activity method for classifying expansive soil based on work conducted on artificially prepared soils. The activity for the artificially prepared soil was defined as

\[ \text{Activity} = \frac{E}{C - 10} \]

where \( E \) denotes the percentage clay size passing sieve number 200. The proposed classification chart is shown in Fig. (3.6).

Chen (1975) from accumulated years of test data on expansive soils in the Rocky Mountain area proposed the classification shown in table (3.2). The classification gives a guide for estimating the probable volume changes in view of clay content, liquid limit and the standard penetration test (S.P.T.) N value. In addition, he
Fig.3.6 CLASSIFICATION CHART OF SWELLING POTENTIAL

HATEN, WOODWARD & ENSCHEN, 1967
Table 3.2: Data for making estimates of probable volume changes for expansive soil (Chen 1975)

<table>
<thead>
<tr>
<th>Age passing</th>
<th>Liquid limit</th>
<th>Standard penetration test</th>
<th>Probable swell degree</th>
<th>Degree of expansion pressure of exp. soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 200 sieve limit</td>
<td>%</td>
<td></td>
<td></td>
<td>(psi)</td>
</tr>
<tr>
<td>&gt;35</td>
<td>&gt;30</td>
<td>&gt;30</td>
<td>&gt;30</td>
<td>&gt;30</td>
</tr>
<tr>
<td>60-95</td>
<td>40 - 60</td>
<td>20 - 30</td>
<td>1 - 10</td>
<td>5 - 20</td>
</tr>
<tr>
<td>30-60</td>
<td>30 - 40</td>
<td>10 - 20</td>
<td>1 - 5</td>
<td>3 - 5</td>
</tr>
<tr>
<td>&lt; 30</td>
<td>&lt; 30</td>
<td>&lt; 1</td>
<td>&lt; 1</td>
<td>1</td>
</tr>
</tbody>
</table>

Holtz and Clabaugh (1955) correlated the percentage volume change to some soil properties: soil content, plasticity index, and shrinkage limit. Fig. (3.7) shows the relationship between these properties and swell potential. Based on the curves of Fig. (3.7), Holtz (1955) suggested the classification shown in Table 3.2.

Single index properties were also used by researchers to estimate the swell potential of expansive soils. Fig. (3.8) shows relationships between plasticity index and swell potential predicted by some researchers, Chen (1975).
Fig. 3.7 RELATION OF VOLUME CHANGE TO COLLOID CONTENT, PLASTICITY INDEX, AND SHRINKAGE LIMIT

AFTER CHEN 1975
Fig. 3.8 RELATIONSHIP OF VOLUME CHANGE TO PLASTICITY INDEX
AFTER CHEN 1975
Also the relationship between the soil potential and the percentage clay size present in a soil was studied, Reed et al (1953) believed that there is no correlation between soil potential and percentage of clay size. However, they concluded that, for a given clay type, the amount of swell will increase with the amount of clay present in the soil as shown in Fig. (3.2).

Table (3.3): Data for making estimates of probable volume changes for expansive soils

<table>
<thead>
<tr>
<th>Colloid percentage</th>
<th>Plasticity limit</th>
<th>Probable expansion of volume</th>
<th>Degree of swelling</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 40</td>
<td>&gt; 40</td>
<td>&lt; 11</td>
<td>&gt; 10</td>
</tr>
<tr>
<td>20-40</td>
<td>10-40</td>
<td>7-12</td>
<td>5 - 10</td>
</tr>
<tr>
<td>10-20</td>
<td>7-10</td>
<td>4-10</td>
<td>3 - 5</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>&lt; 7</td>
<td>&gt; 6</td>
<td>&lt; 2</td>
</tr>
</tbody>
</table>

Empirical correlations were introduced by several investigators to predict soil potential.

Table (3.4) below shows relationship between some simple soil properties and soil potential as reported in the literature.
Fig. 39  PERCENTAGE OF SWELL AGAINST PERCENTAGE OF CLAY SIZES

AFTER SEED ET AL. 1982
Data (1) swelling potential in terms of some single soil properties as predicted by some authors. (After Arnold, 1984)

<table>
<thead>
<tr>
<th>Relationship</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. $C_1 = 0.000001 E_{2.4}$</td>
<td>Seed et al (1962)</td>
</tr>
<tr>
<td>2. $C_2 = 0.000041 M_{5.67}$</td>
<td>Bergman and Swaminathan (1965)</td>
</tr>
<tr>
<td>3. $C_3 = 0.025 T^{1.45} e^{0.0633 S}$</td>
<td>Mogh and Christiansen (1971)</td>
</tr>
<tr>
<td>4. $C_4 = 1.310^{-0.203 (1 - w/w_{50})}$</td>
<td>Wijaya and Bhatia (1971)</td>
</tr>
<tr>
<td>5. $C_5 = 0.6610 (x^3 - 1 - 1.10)$</td>
<td>Schneider and Peer (1974)</td>
</tr>
<tr>
<td>6. $C_6 = 0.2922 e^{-0.248 w}$</td>
<td>Chen (1975)</td>
</tr>
<tr>
<td>7. $C_7 = 0.00019 (1 - 2.3^w)$</td>
<td>Weston (1989)</td>
</tr>
</tbody>
</table>

**Definitions:**
- $C$ = Swelling potential ($\phi^*$)
- $w$ = Initial moisture content
- $N_d$ = Shrinkage index ($N_d - C_l$)
- $L_l$ = Liquid limit
- $S_l$ = Shrinkage limit
- $P_i$ = Plasticity index


<table>
<thead>
<tr>
<th>Method No. and Description</th>
<th>Properties Used</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 BBST</td>
<td>IL, PI</td>
<td>Louisiana DOI, 1973</td>
</tr>
<tr>
<td>2 MCT</td>
<td>PI</td>
<td>Kansas Highway Comm., 1974</td>
</tr>
<tr>
<td>3 Soma</td>
<td>PI, ST</td>
<td>Kansas, 1967</td>
</tr>
<tr>
<td>4 Severence</td>
<td>SE,PI</td>
<td>Severence, 1967</td>
</tr>
<tr>
<td>5 MSF</td>
<td>L, L, SI, SI</td>
<td>Bishan and Kher, 1973</td>
</tr>
<tr>
<td>6 ANS</td>
<td>HI, SX, swell</td>
<td>Anderson &amp; Shannon, 1969</td>
</tr>
<tr>
<td>7 AAS</td>
<td>SI</td>
<td>Renganathan &amp; Sivaraman, 1969</td>
</tr>
<tr>
<td>8 SNM</td>
<td>resist. soil</td>
<td>Saito &amp; Iwai, 1973</td>
</tr>
<tr>
<td>9 USHR</td>
<td>β, ρ, χ, IL, ΔL</td>
<td>Saito, 1973</td>
</tr>
<tr>
<td>10 Altenberger</td>
<td>LI, SB</td>
<td>Altenberger, 1969</td>
</tr>
<tr>
<td>11 SWI</td>
<td>ρ, swell</td>
<td>Rekd et al., 1982</td>
</tr>
<tr>
<td>12 Shen</td>
<td>ρ, SV200, IL, ΔL</td>
<td>Chen, 1985</td>
</tr>
<tr>
<td>13 VML</td>
<td>W, IL</td>
<td>Vajayverdi &amp; Vinod, 1973</td>
</tr>
<tr>
<td>14 VML</td>
<td>W, IL</td>
<td>VIJAYVERDI &amp; VINO, 1973</td>
</tr>
<tr>
<td>15 Shroeder</td>
<td>Shulling Index</td>
<td>Shroeder, 1970</td>
</tr>
<tr>
<td>16 M &amp; C</td>
<td>T, V = 1.0, MP, N</td>
<td>Mogil &amp; Christiansen, 1974</td>
</tr>
<tr>
<td>17 K &amp; B</td>
<td>LE, χ, χ</td>
<td>Kooranik &amp; David, 1969</td>
</tr>
</tbody>
</table>

**Symbols:**
- IL = Liquid limit
- PI = Plastic limit
- LL = Liquid limit
- ΔLL = 29-33 Mv count
- χ = Natural dry density
- ΔL = Plasticity index
- χ = Swelling index
- LE = Linear shrinkage
- χ = Natural water content
The cone penetration test has been used in many countries in the field of soil exploration. The test offers valuable information about soil classification, strength, type, and stratification. The experience gained from the use of the C.P.T. was published in many books, proceedings of conferences, and professional journals. In Sudan, the C.P.T. was used successfully by the Building and Road Research Institute in research work as well as testing services.

Recently the DML initiated a comprehensive research program to study and evaluate the physical and mechanical properties of expansive soils. The present research work is part of this program. The purpose of this work is to present:

/ The possibility of classifying potentially expansive soil using the static C.P.T. data.
The possibility of describing the consistency of the soil based on the cone resistance.

4.1.2 Pneumatic Cone

4.1.2.1 Static cone penetrometer testing:

A 10 tonne batch static cone penetrometer testing was used in this research program. A brief description of the machine was given in Chapter 2. The type of cone used throughout the testing program was the American standard cone. The test procedure is the same as that described in Chapter 2. The cone was driven into the soil at a constant rate of 0.25 cm/s. The cone resistance and the skin resistance values were recorded at 0.2 metre intervals.

For the purpose of studying soil properties both borehole sampling and static cone penetration tests were performed. The static cone penetration tests were positioned near to the bores. This had two goals: it was simultaneously desired to use these for enough support so as to minimize the effects of nearby soil failure zones, and yet close to
satisfy the horizontal homogeneity of soil. A distance of 2 metres was chosen between the boreholes and the static C.F.T. locations.

4.1.1 Drilling Procedure

A model A KII Adco rotary drilling rig mounted on a truck as shown in Fig. (4.1) has been used for drilling the boreholes at the sites examined. Power operated continuous flight augers were used for advancing the bores. Disturbed soil samples were collected from the blades of these augers at intervals of 0.3 metre. Undisturbed soil samples were taken by an open drive sampling tube of 4 inch diameter (34 sampling) to 0.5 metre intervals. Disturbed and undisturbed soil samples were then transported to the laboratory for testing.

4.2 Geology of the Studied Areas

The results of site investigation of the areas are presented in this research work: Jangli Canal Break and Sharan Break. The location of the two areas on the country map are shown in Fig. (4.2). The general geology of these areas are given below.
POWER EARTH AUGER (TRUCK MTD)

CONTINUOUS FLIGHT AUGERS IN SECTIONS

CUTTER HEAD (REPLACEABLE TEETH)

Fig. 4.1 ROTARY DRILLING RIG.
① KHARTOUM AREA
② JONGLEI CANAL AREA

Fig. 4.2 LOCATION OF STUDY AREA ON COUNTRY MAP
In the southern Region the dominant geological feature is the great escarpment formed by the sinking of a land surface, curved by erosion out of the basement complex, the depression subsequently filled by alluvial deposits of the Rakaia formation (Andrew, 1974). The project site is in the flat clay plain with major slopes of 3m per kilometre towards the north. (Tenders project report, phase 1, 1974).

The exploratory boreholes drilled along the alignment of the proposed Canal indicate that the deposits are formed of a clay layer followed by sandy layers.

At the Bull Island site near Invercargill, the soil profile was found to consist of swelling clays with small subaqueous sediments in several places (Andrew, 1975). The clays contain mixed layers of more than 50% by weight montmorillonite whereas quartz and kaolinite minerals range between 15% and 25% (Tenders project report, preliminary investigation reports, 1973).
Typical expansive soil areas, where failures of building exist, were included in this study. At Umea, the depth of expansive soil was found to be at least 3m. X-ray diffraction identification method showed that the mineralogical composition of the soil is mainly montmorillonite, (Hammel, 1983).

4.3.1 Field Testing:

Only static cone penetration tests were conducted in the field. These tests were carried out using a 10 ton Dutch machine with an adhesion socket cone which permits measurement of the cone resistance $q_c$ as well as the skin friction $p_s$. $q_c$ and $p_s$ were measured with depth at 0.3 metre intervals.
Single laboratory tests were carried out on soil samples to predict the swell potential of the soil. These tests consisted of:

- Atterberg limits and particle size analysis;
- Also, to calculate consistency of the soil, water content measurements were carried out.

The laboratory tests were performed in accordance with the British Standard B.S. 1713-1975.
RESULTS AND DISCUSSION

3.1 Identification of expansive soil from Middle East

Identification and Classication of Soil

3.1.1 Introduction

Problems associated with expansive soils are now reported worldwide. Recognition of the behavior of these soils forms a major part of soil engineers' work. Research on this type of soil is still in progress, studying soil behavior and developing techniques for identifying swelling soils in engineering practice.

Identification and classification of swelling soils remain to be very important to soil engineers to decide on the need for more testing to be performed on it so as to evaluate the required design parameters.

Although the swelling phenomenon has been fully recognized for many years, no well-defined unique method of assessing the swelling characteristics of clays has been established. This may be due to the numerous variables involved.
Seed et al. (1962) defined the swelling potential as the percentage of soil of a laterally confined sample on swelling under 1 psi surcharge, after being compacted to a maximum density of optimum moisture content in the standard AASHO compaction test.

Isaka (1959) used the swell index to measure the expansion characteristics of clay. The swell index is defined as the slope of the e-log e curve. The pressure increment from 1.0 to 0.1 kPa/cm² was used.

Hadas prepared a standard method for testing expansive soils. This method utilizes the conventional fixed-ring consolidometer for conducting the test. Procedures are given for testing both undisturbed and remolded specimens.

Seed et al. (1962) described a method to identify and predict the swelling potential of expansive soils. Their study was performed on artificially prepared soils to predict the swelling potential of compacted clays. A relationship between the percentage of clay size present in the soil, the activity of the soil and the
null potential was then established. This relationship was used to define boundaries between four soil potential zones in a plot showing the activity versus clay content. This is shown in Fig. (2.1). The chart, though constructed from the results of artificially prepared soils, proved to be very valuable for predicting the soil potential of natural soils. Fig. (3.1) shows the applicability of the chart for classifying 27 natural soils.

However, no methods have been encountered for field identification of swelling soils. The present study is an attempt to fulfill the requirement of a field method for classifying potentially expansive soil. The static cone penetration field testing method was used to develop such classification. The study stressed the value of results obtained from field testing.

1.1.2 Experimental Results:

In this study static cone penetration tests were carried out near poweragner, drilled boreholes from which soil samples were collected and transported to the laboratory for further testing. The cone penetration tests were conducted at a distance of 3 meters away from
Fig. 5.1  APPLICABILITY OF PROPOSED CHART FOR CLASSIFICATION OF TWENTY-SEVEN NATURAL SOILS

SEED ET AL. 1962
the boreholes as detailed in Chapter Four.
The test procedure is also as that described
in Chapter Two section (2.7). Continuous
records with depth of penetration of the cone
resistance and skin friction were obtained
from the test. The records were made at
0.5m intervals. Later the friction ratio
was computed from the skin friction and cone
resistance values.

The soil samples collected from the boreholes
were transported to the laboratory where they
were further tested.

Classification tests were performed on these
samples to form the basis of comparison with
the field cone C.P.P. results. These tests
were: Liquid limit, Plastic limit and Grain
size distribution. These were performed in
accordance with the British Standards BS 1377-
1975. From these tests the plasticity index
and clay content of the soil were obtained.
These two parameters were used to classify
the soil potential of the soil using the
method described by Seed et al (1962) and
outlined previously, see section (3.3) and
section (4.1) with the aid of this method
the soil potential of the encountered soils
were classified as low, medium, high or very
5.1.3 Analysis of Results

Many methods for identifying and classifying potentially expansive soils were suggested by various authors, see Chapter 3. The definition of the swell potential vary greatly between these methods, the comparison between the different classifications is therefore not possible. A single method of classification is utilized here to estimate the degree of swell potential. This method is the one proposed by Seed et al. (1958). The method describes the swell potential of the soil in terms of its plasticity index, clay content (passing sieve No. 200) and activity.

For natural soils Seed et al. defined the activity of the soil as

\[ A = \frac{N}{C} \]  \hspace{1cm} (5.1)

Where:

- \( A \) = Activity
- \( N \) = Plasticity Index
- \( C \) = Clay size fraction passing sieve No. 200.
Given the activity as defined by eq. (3.1) and the clay fraction of the soil, a point can be located in the classification chart of Fig. (5.1). The zone into which this point falls defines the soil potential of the tested soil. This was repeated for all the soils encountered in the sites studied as mentioned in Chapter 4. Fig. (5.2) shows the distribution of all the points in the chart. The experimental points were found to fall into one of four zones, namely: very high, high, medium and low soil potential. Points falling within the boundaries of each zone were given similar symbols. These same symbols were used later on for the same points when comparison is made with the field test results.

The core resistance $R_c$ and the skin friction $f_s$ at the respective depths of each of the experimental points shown in Fig. (5.3) were calculated. Each point of $R_c$ and $f_s$ corresponding to a given soil potential zone were plotted in a graph as shown in Fig. (5.3a). The corresponding points in each soil potential zone of Fig. (5.3) bear the same symbol in Fig. (5.3a). This figure shows the combined plot of $(R_c, f_s)$. The lines representing $R_c = 2f_s$ and $S_1$ roughly bound the experimental points.
Fig. 5.2 DISTRIBUTION OF THE EXPERIMENTAL POINTS IN THE CLASSIFICATION CHART
Two vertical lines were arbitrarily chosen and drawn as to pass through $R_p=1500$ kN/m$^3$ and $R_p=3500$ kN/m$^3$. Guidelines for the choice of the values 1.5 and 3 were obtained by referring to Bogaard's curves, Fig. (2.13), Schmidt's curves, Fig. (2.14) and soil curves, Fig. (2.15). From these figures it is clear that $R_p = 3$ indicates the start of sandy clays and silty clays and the value of $R_p = 2$ is the limit for sands. The $R_p = 2$ limit excludes non-expansive soils. The line $R_p = 3$ indicates clayey sands and silt clay mixtures. It may be expected that this will involve low but at the most medium expansive properties. The data indicates that this region must be bounded by an $I_{c}$ line. The line $I_c = 1.5$ was chosen as the boundary. The line $R_p = 5$ values us well into the clays where above medium expansive characteristics are expected. Again a boundary was chosen. $I_{c} = 1$ above $R_p = 5$ non-sensitive inorganic clays are encountered. These may be expected to have very highly expansive properties.
5.1.4 Discussion

The soil potential of a given soil depends on many factors such as initial dry density, initial moisture content, clay mineral, specific surface area (S.S.A.) etc. (Kabz et al. 1982, Young 1986, Chen 1975). Various researchers have noted the influence of one of these factors, especially the dry density, water content and clay content on the cone penetration results (Gusaw, 1975, Genesis 1975, Bayrak et al. 1985, Schurtz, 1962). The question is how useful the possibility of using the static cone penetration test for identification of swelling soils. For the soil core study, Zain (1983) used a statistical method based on discriminant analysis and found that from the static C.P.T. results alone one could decide on the type of soil being penetrated. He could identify sands as a group, silty sands and silty clays as a group in addition to the fourth group of clays. However, no work has been done to classify and identify swelling soils from the static C.P.T. results.

The present study focuses on the possibility of finding those with which one could identify and identify potentially expansive soils using...
the static cone penetration field test method. The method developed by Seed et al. (1962) and shown in Fig. (5.1) has been used as a reference. Values of the cone resistance, $q_c$, and skin friction, $f_s$, as obtained from static cone penetration testing may be plotted on a graph. The study aims at the possibility of correlating actual experimental points obtained in this study to points defined by one of the zones of the chart shown in Fig. (5.1) and referred to in Eq. (5.2), to a zone of the graph of $q_c$ versus $f_s$ mentioned above.

Fig. (5.1 a) shows such a plot of $q_c$ against $f_s$. The experimental points are given the same symbols of Fig. (5.2) which shows the activity - clay content relationship for the equivalent points. From Fig. (5.1 b) a dependence between $q_c$, $f_s$ and the small potential of the investigated soil can be observed. The results show that the small potential increases with the increase of $q_c$ and $f_s$. The trend indicates the possibility of identifying zones that vary with the skin friction $f_s$ and friction ratio $f_r$. It was found possible to define three regions of points having similar symbols. These regions are separated by two lines:

one passes vertically through $f_r = 1.5$ kN/m$^2$.
up to $q_v = 50$ kN/m² and then extends along the line representing $R_v = 36$, the other one passes vertically through $q_v = 35$ kN/m² up to $q_v = 60$ kN/m² and then proceeds along the line $R_v = 6$. This illustration is shown in Fig. (5.1 b). The line $R_v = 26$ bounds non-expansive soils as previous investigations relate this value for safety.

The lines thus constructed divide the graph of $q_v$ vs $R_v$ into four zones: non-expansive, low to medium, medium to high, and very high small potential zones. There is, however, considerable scatter of data at the boundaries. Again distinction between low and medium small potential zones is not possible. This can, when needed, be obtained by providing more data and further investigations in that area.

The results of the static S.P.T. classification as obtained from Fig. (5.3 b) are summarized in Table (5.1) below.

<table>
<thead>
<tr>
<th>$q_v$ vs $R_v$</th>
<th>Degree of small potential from static S.P.T. results</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;23</td>
<td>Very high</td>
</tr>
<tr>
<td>21 - 23</td>
<td>High</td>
</tr>
<tr>
<td>11 - 20</td>
<td>Medium to low</td>
</tr>
<tr>
<td>&lt;11</td>
<td>Non-expansive</td>
</tr>
</tbody>
</table>
Fig. 5.3a PLOT OF QCS WITH SYMBOLS SHOWING THE SWELL POTENTIAL OF THE INVESTIGATED SOILS
Fig. 5.3b SWELL POTENTIAL CLASSIFICATION ZONES
This table gives the small potential using the static G.P.T. data only.

The data no data have been encountered to show that the static G.P.T. can be used to classify potentially expansive soils. The results presented here, may require further study and investigations. Nevertheless, the results shown in Fig. (5.3 b) and summarized in Table (5.1) show very useful identification criteria of swelling soils using the static cone penetration field test data only.

The value of friction ratio which permits identification of soil type, can be used to give a first trial on the soil that possesses swelling characteristic. Kain (1936) found that values of friction ratio equal to 1.75 correspond to sand. Draper (1978) stated that values of friction ratio less than 9% are typical of cohesive soil. This was also observed by Behera (1984). It has been mentioned earlier in section (3.1.3) that for values of $R_F = 3/4$ the soil may be expected to have low to medium expansive properties, whereas for $R_F > 3/4$ no expansive properties are to be expected. Hence for soils having friction ratios greater than $3/4$, but less than $10\%$, it may be expected that there will be low to medium swell potential. Soils having
$R_p > 1$ correspond to clayey and clay soils which usually exhibit swelling characteristics. This has been divided into two regions:

(a) High swell potential region for $R_p$ greater than $1/2$ but less than $5/2$.

(b) Very high swell potential region for $R_p > 5/2$.

It is thus possible to decide, even in the field and without any further laboratory investigations on the swell potential of the soil, that has been tested by the static cone penetration machine equipped with an adhesion jacket cone type. The results are shown graphically in Fig. (2.3.1) and in tabular form in Table (2.1).
To get an idea of the influence of water content on the friction ratio, the types of static cone penetration tests were performed in the field with varying initial water conditions. One of the tests was performed under natural conditions and the other one after the natural water conditions were altered by the addition of water to the soil. Water was allowed to enter the soil through a drilled borehole four metres deep and left to seep in the soil for three days. Soil samples were taken for measurements of water content before and after the addition of water. An average increase in the water content of 2.4% has been observed in the upper four metres.

Fig. (2.4) and Fig. (2.5) show values of the cone resistance and the skin friction plotted with depth for the tests before and after the addition of water respectively.
Fig 5.4 static c.p.t. results before addition of water
Fig 5.5 STATIC C.P.T. RESULTS AFTER ADDITION OF WATER
Comparison between the two tests shows that a drop of the cone resistance and the skin friction values has taken place. Friction ratios were then calculated in each case. Fig. 1.6 shows the variation of $R_p$ with depth for the two tests. It can be seen from this figure that on the average and except for a few points, the value of $R_p$ does not change much with increasing water content. In other words it can be stated that the cone resistance decreases proportionally with the skin friction when the water content of the penetrated soil is increased.

For the upper layer, the classification on the basis of $R_p$ alone remains unaltered for the two tests except at a depth of 2.2 where $R_p$ changed from 4.4 to 1.5. The limit between high and very high swell potential being $R_p = 5$. However, this is still within the range of salt weather. This indicates the possibility of using the friction ratio to decide on the potential of the soil to swell irrespective of the moisture content.

Variations in moisture content change both the cone resistance and skin...
Fig. 5.6 VARIATION OF FRICTION RATIO WITH DEPTH FOR THE STATIC CPT’S SHOWN IN Figs. 5.4 AND 5.5
tension. However, the results indicate that the value of $M_p$ changes within such limits that it does not lead to a change of classification. The results previously shown in Fig. (3.1) and Table (3.1) thus remain valid.
5.2.1 Introduction:

The consistency of the soil is usually described in terms such as soft, stiff or hard. It is thus a measure of the strength of the soil.

Relative consistency or the liquid limit index is defined as the ratio of the liquid limit (L.L.) minus the natural water content (W) to the plasticity index (P.I.) (Tunis 1983)

\[ C_r = \frac{L.L. - W}{P.I.} \]  \hspace{1cm} (9.2)

where:

- \( C_r \) = relative consistency

Thus if the relative consistency of the soil is equal to unity it is at the plastic limit. Similarly, a soil with \( C_r = 0 \) will be near its liquid limit. If \( C_r \) exceeds unity the soil is in semi-solid state and will be stiff.

A negative \( C_r \) indicates that the soil has an actual water content greater than the liquid limit and hence behaves just like a liquid. This index can therefore be used to study the field behaviour of soil.

Attempts to describe soil consistency based on other soil parameters when compared to this index were made by various authors. (See section 2.3.2).
In the following sections the dependence between the relative consistency and the cyclic cone resistance is studied.

3.3.2 Field and Laboratory Test Results:
Static cone penetration tests were conducted near power sugar drilled boreholes giving ultimate soil resistance to the movement of the cone. The procedures adopted in this test is the same as that described earlier (see section 2.7). Continuous records of cone resistance and skin friction values obtained from this test at 0.2m intervals were made.

Soil samples were collected from the boreholes and transported to the laboratory for further testing. The tests were carried out in order to determine the liquid limit, plastic limit and natural water content of the soil. The tests were conducted according to the British Standard B.S. 1377 - 1989. The results of these tests with the corresponding cone resistance expected at each depth are summarized in Appendix 1.

7.4.4 Analysis of Results:

The soils encountered in the Jamseti canal area and Amrabad area are considered in this study. The distribution of points representing each
The plasticity chart of Camboglue is shown in Fig. (5.7). It is noted that the soil can generally be described as inorganic soil of medium to high plasticity. The points are located above the A line and they are distributed along lines passing parallel to it.

The relative consistency of the soil has been calculated according to eq. (2.3), which gives the relative consistency, \( C_r \), in terms of liquid limit (L.L.), plasticity index (P.I.) and natural water content (W.n).

The average cone resistance, \( C_c \) (Kgf/cm²), values measured at each depth where the relative consistency has been obtained was evaluated from the static C.R.E. data.

This cone resistance value was then plotted against the relative consistency in a semi-logarithmic graph as shown in Fig. (5.8). The plot gives the cone resistance in the logarithmic scale and the relative consistency in the linear one. For both sites, the cone resistance values were found to vary between 1.4 Kgf/cm² and 220 Kgf/cm² with frequent values between 40 Kgf/cm² and 100 Kgf/cm². The relative consistency varied between 0.3 and 1.3 with frequent values between 0.8 and 1.1.
Fig. 5.7 DISTRIBUTION OF JUNGLEI AND MANSHIA SOILS IN CASSAGRANDE PLASTICITY CHART.
Fig. 5.8 VARIATION OF RELATIVE CONSISTENCY WITH THE CONE RESISTANCE
Regression analysis were carried out on each set of points separately. For soils encountered at Jungli Chawl area the regression analysis yielded the following equation:

$$C_p = 0.56 \log c_v - 0.02 \quad (5.3)$$

with $c_v$ in $kN/m^2$ and a correlation coefficient of 0.74.

For Bandra soils the relationship between $C_p$ and $c_v$ ($kN/m^2$) as obtained from the regression analysis was found to be as follows:

$$C_p = 0.23 \log c_v + 0.03 \quad (5.4)$$

with a correlation coefficient of 0.86.

5.2.1 Discussion of Results

Fig. 5.3 shows the variation of the cone resistance $c_v$ ($kN/m^2$) with the relative consistency $C_p$ of the soil encountered at Jungli Chawl area and Bandra area. The relative consistency was found to increase linearly with the logarithm of the cone resistance.

Furthermore, regression analysis carried out on the plotted data gives the following relationship between $C_p$ and $c_v$: --
\( c_p = 0.5 \times \log (\frac{c_p}{c_p}) - 0.02 \) \hspace{1cm} (5.3)

for soils at Nangal Canal area and

\( c_p = 0.63 \times \log (\frac{c_p}{c_p}) + 0.01 \) \hspace{1cm} (5.4)

for soils at Hardies, with correlation coefficients of 0.74 and 0.85 respectively.

In view of the close agreement between eq. (5.3) and eq. (5.4) a single relationship for the combined data is developed using regression analysis. This relationship can be written as follows:

\[ c_p = 0.6 \times \log (\frac{c_p}{c_p}) - 0.11 \] \hspace{1cm} (5.5)

again with \( c_p \) in kPa/m and a correlation coefficient of 0.92.

A good correlation is seen to exist between these two parameters. Some scatter of the plotted data, however, is expected. Some of the variance attributed to the scatter may be due to some or all of the following factors:

1) The investigated soil ranges between

inorganic soils of medium plasticity to

inorganic soils of high plasticity. More

strong correlation could be established

if these soils represent only one soil type

as discussed by Walker (1975).
The effect of overburden pressure on $c_r$, where has been ignored.

3) The effect of the liquid limit and plasticity index is not considered in the analysis.

4) Inaccuracy in measured field and laboratory results.

Nevertheless, the cone resistance can be used or least to give an idea of the relative consistency of the soil. This can be done by providing a consistency classification of soil in broad ranges of average cone resistance taking into account the soil type.

The types of soil now under study, although represent two different locations, but they show similar classification in Crosgen's chart, see Fig. (5.7). Also, the variation of the cone resistance and the relative consistency for both types of soil shows approximately similar behaviour. A good correlation still exists for the combined data. The data can therefore be analysed furtherly regardless of the location from which the data is obtained.
It is known from eq. (5.2) that if the relative consistency, $C_r$, of the soil is equal to unity it is at the plastic limit. If $C_r$ exceeds unity the soil is in some solid state and will be stiff.

Making use of eq. (5.3) this can be stated in terms of the cone resistance values as follows:

If $C_r = 1$ then $q_c = 70$ kPa/m² and the soil is at the plastic limit. If $C_r > 1$ then $q_c > 70$ kPa/m² and the soil is considered to be stiff. Values of $q_c$ less than 70 kPa/m² represent medium to soft soil. Furthermore, a limit can be introduced between soft and medium soils. Again subdivisions are possible. This may be taken arbitrarily as shown in Table 5.2 below:

<table>
<thead>
<tr>
<th>$C_r$</th>
<th>0.45-0.75</th>
<th>0.75-1</th>
<th>1-1.25</th>
<th>1.25-1.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_c$</td>
<td>10-20</td>
<td>22-70</td>
<td>70-125</td>
<td>125-160</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$q_c$</th>
<th>very soft</th>
<th>soft</th>
<th>medium stiff</th>
<th>stiff</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-20</td>
<td>Extra</td>
<td>very</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22-70</td>
<td></td>
<td>very</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70-125</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>125-160</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;160</td>
<td></td>
<td></td>
<td></td>
<td>very</td>
</tr>
</tbody>
</table>

Table (5.2) Estimate of consistency from cone resistance value.
Using this table it is possible to classify soil consistency from field C.P.T. data only. The classification is intended to give estimates of soil consistency based on the value of the cone resistance observed during field C.P.T. cone penetration testing.
6.1 CONCLUSION

6.1.1 Identification of Expansive Soils

6.1.1.1 Static Cone penetration tests were used in this study to identify and classify potentially expansive soils in the field. The study shows the possibility of classifying potentially expansive soils using the static C.N.T. data. The cone resistance, $f_c$, the skin friction, $f_u$, and the friction ratio, $R_f$ ($R_f = f_u / f_c$ 100), were found to be good indicators of each potential of soil penetrated by the static cone penetration machine.

6.1.1.2 Four zones of varying swell potential could be identified from the plot of $f_u$ versus $f_c$. These zones are: non-expansive soil zone, the low to medium zone, the high zone and the very high swell potential zone. The ranges of $f_u$ and $f_c$ representing these zones have been found to be as follows:


### Table

<table>
<thead>
<tr>
<th>Degree of expansion</th>
<th>( \delta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very high</td>
<td>( \geq 3 )</td>
</tr>
<tr>
<td>High</td>
<td>( 1.5 \leq \delta &lt; 3 )</td>
</tr>
<tr>
<td>Lesser to low</td>
<td>( 1.0 \leq \delta &lt; 1.5 )</td>
</tr>
<tr>
<td>Non-expansive</td>
<td>( \delta \leq 1.0 )</td>
</tr>
</tbody>
</table>

6.1.1.3 In order to show the influence of the moisture content on the friction ratio, static cone penetration tests were performed under varied soil moisture conditions. It was observed that the friction ratio changes only slightly with increasing moisture content. The slight change of friction ratio values indicates the possibility of using \( P_f \) for classification of expansive soil as described previously.

6.1.2 Variation of the relative consistency with the cone resistance:

6.1.2.1 The cone resistance was used in this study as a characteristic shear-strength parameter. The cone resistance \( q_{c,\text{unf}} \) has been correlated to the relative consistency \( Q_p \) of the soil encountered at Benaal Basin area and in certain cases.
and correlation has been found to exist between these two parameters. This correlation is represented by the mathematical relationships:

\[ c_1 = 0.54 \log P_{\text{atm}} - 0.42 \]

and

\[ c_2 = 0.33 \log P_{\text{atm}} + 0.03 \]

for jungle soils and pyandic soils respectively. Correlation coefficients being 0.75 and 0.95 respectively.

Regression analysis yielded the following equation for the combined data:

\[ c_0 = 0.5 \log P_{\text{atm}} - 0.11 \quad (5.5) \]

with \( c_0 \) in kPa/m. The correlation factor for this equation is 0.92.

(5.2) summaries of \( c_0 \) (kPa/m) that describe soil consistency based on eq. 5.5 above were arbitrarily identified as follows:

This table is intended to give estimates of soil consistency as derived from measured field cone resistance values, \( c_0 \).
<table>
<thead>
<tr>
<th>$o_p$</th>
<th>$0.05$</th>
<th>$0.25 - 0.5$</th>
<th>$0.5 - 0.75$</th>
<th>$0.75 - 1$</th>
<th>$1 - 1.25$</th>
<th>$1.25 - 1.5$</th>
<th>$&gt; 1.5$</th>
</tr>
</thead>
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Table of cone resistance and relative consistency.
1.7 Recommendations for Further Work:

The findings presented in this study indicate general trends towards the use of the static C.P.T. to identify potentially expansive soils.

Although the study has given rise to real values, it needs confirmatory studies made in other sites to widen the scope of its applicability.

It is therefore recommended that further work should be done in potentially expansive soils in different parts of the country to supplement the results presented in this thesis.

Secondly, it is recommended to elaborate on the study of the shear strength of swelling soils using static C.P.T. under varying water conditions.
Television and field test results of
JPL's first new radar system. ...
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Note: The table represents data from different studies or experiments, with columns indicating various measurements or parameters and rows detailing specific data points. The classification column categorizes the data points into different levels (Medium, High) based on the values in the other columns.
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