EFFECT OF SWELLING SOIL IN THE FOUNDATION OF RIVER VALLEY STRUCTURES

A DISSERTATION

submitted in partial fulfilment of the
requirements for the award of the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled, **"EFFECT OF SWELLING SOIL IN THE FOUNDATION OF RIVER VALLEY STRUCTURES"**, in partial fulfillment of the requirements for the award of Degree of Master of Engineering WRD (Civil) submitted in the Water Resources Development Training Centre, University of Roorkee, Roorkee is an authentic record of my own work carried out since 16^{th} July, 2000 to January, 2001 under the supervision of **Prof. Gopal Chauhan,** Professor, WRDTC and **Dr. N.K. Samadhiya**. Assistant Professor, Civil Engineering Department, University of Roorkee, Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Dated : January, 16th 2001

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This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

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(VIKAS KUMAR JAIN)

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SYNOPSIS

The safety of any River Valley Structure is of major concern. This safety becomes of paramount importance if the structure is required to be built on expansive/swelling soils and ubiquity of the BLACK COTTON SOILS (which contain predominance of mineral MONTMORILLONITE responsible for its high shrinkage and swelling characteristics) in various parts of our country. It may not be possible to avoid construction of WRD structure on them.

The swelling and abnormal volume changes occur in expansive soils due to variation in moisture content, make the behavior complicated and sometimes disastrous failure of structure founded in/on such soils may occur.

These problematic soils are encountered mostly in Central & Southern part of India. Instability of structure constructed on swelling soils are essentially due to volume expansion and contraction in the soil mass on saturation and drying during rainy and summer season respectively. The structure founded on swelling soils undergoes undesirable large-scale movement, due to resulting variation in strength & compressibility characteristics of the soils.

This paper is an attempt to study the general problems associated with swelling soils both as support material for foundation and also as construction material. This paper also presents the possible remedial measure for swelling soils.

Soils samples having swelling characteristics were tested to know their index and engineering properties and to evaluate swelling pressure.

INTRODUCTION

1.1 GENERAL

In India, Expansive/Swelling Soils which are popularly known as `Black Cotton Soils' cover about 810,000 km^2 i.e. 20% of the total area of the country. The major occurrence is south of Vindyachal range covering almost the entire Deccan Plateau as can been seen fig 1.1. Parent material is normally derived from the weathering of basalt. However there are black cotton soil deposits derived from other types of rocks and also from very old sedimentary deposits. The region has poor drainage, low to medium slope of 1 to 3 degree and a high rainfall of 300 to 900 mm a year. The Black Cotton Soil deposits are boon to farmers in India but are problematic to Civil Engineers.

It is the characteristic of swelling soil to exhibit large volume changes due to variations in the moisture content. In the summer their appearance is usually distinctive, i.e. the ground surface exhibit polygon cracking. The cracks are found to range in width from the thickness of a hair to 80 mm, extending to a depth of about 2m below ground level. The size of the polygons often indicates the degree of expansiveness, the smaller the polygons, greater the clay content and the greater the expansiveness. Cracks occurs in both the horizontal and vertical directions. On rain fall, the soil absorbs water and swells, initially closing the cracks and then with further swelling vertical movements of the ground takes place. The magnitude of the swelling measure and heave are high enough in comparison to the weight of lightly loaded structures to move their foundation considerably.

These movements don't occur uniformly and swelling pressure values may range from less than 1 kg/cm^2 to as high as 13 kg/cm^2. Differential movements of the soil cause considerable damage to any Water Resources Development structure with inadequate & proper foundation. The instability of the structures constructed on such soils is essentially due to volume expansion inherent in the soil mass on saturation during rainy season to attain heaved condition and reduction in volume on drying and shrinkage during summer season.

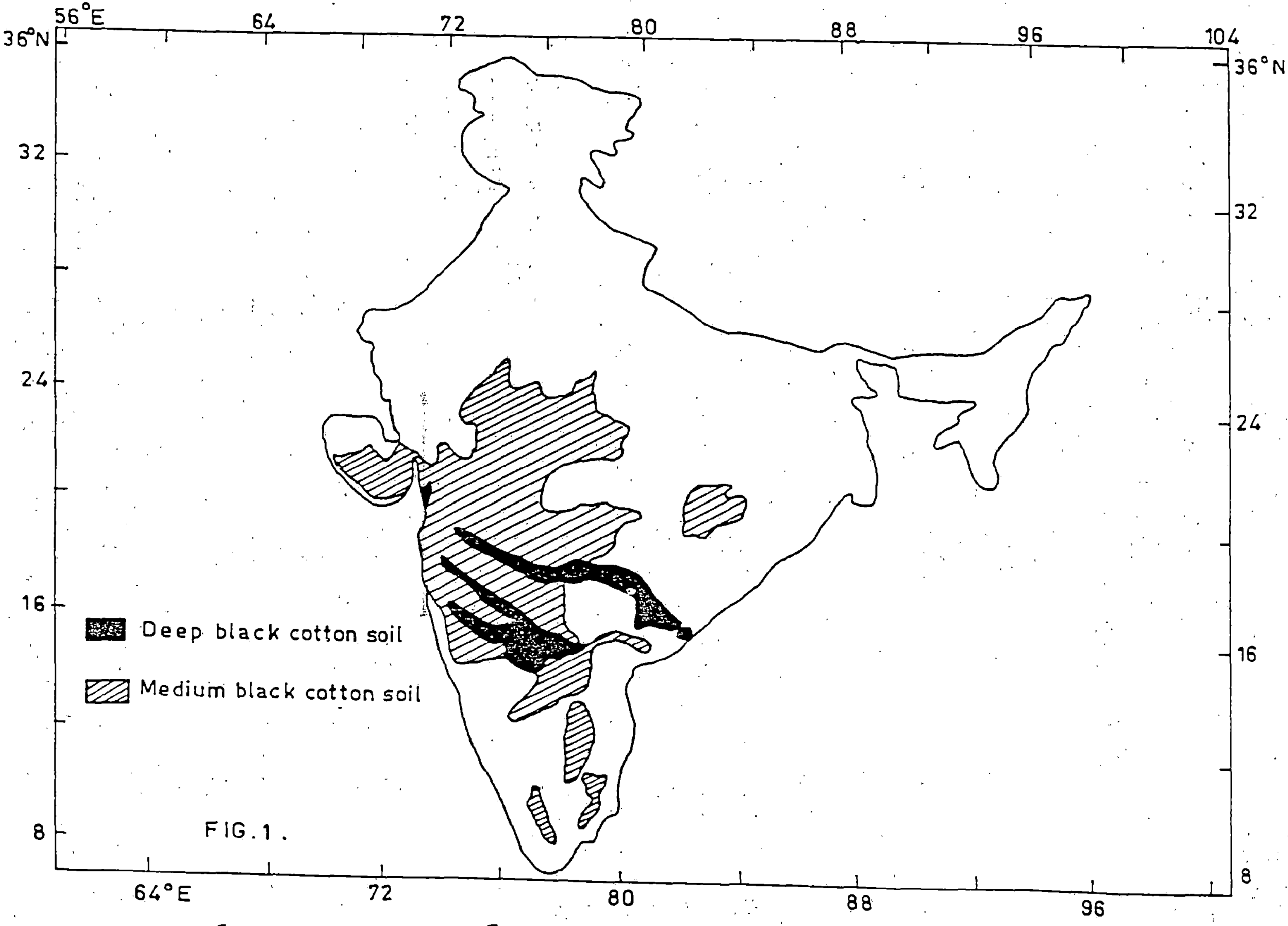


Fig 1.1 Map of India Showing approximate regions of Swelling Soils

The structure constructed using swelling soil as foundation material or construction material, thus experience large scale movements and consequently variation in strengths and compressibility properties during rainy and summer season [C. Prakash, 1988].

The US Bureau of Reclamination first recognised the swelling soil problem in 1938 in connection with a foundation for a steel siphon.

1.2 PLACEMENT CONDITIONS

It is almost universally established that increase in the dry density cause large amount of swell and also cause higher swelling pressure. The reason is that higher densities result in closer particles spacing thus causing greater particles interaction. [Srirama Rao, 1998]. This results in greater volume change. Initial water content affects suction which in turn affects the amount of swelling. The lower the initial moisture content, the higher the suction and the capacity of the soil to uptake water and swell.

Thus distinction must be made between soils that have the capacity to swell and those that actually exhibit the swelling characteristics in the field. Soils having little or no capacity to swell will not do so under any circumstances. On the other hand, soil with high swelling capacity may or may not swell; their behaviour depends upon the physical condition of the material at the beginning of the construction and the change of stress and moisture content to which they are subjected.

1.3 OBJECTIVE AND SCOPE OF STUDY

WRD structures experience large-scale damage due to heaving accompanied by loss of strength of swelling soil during rainy season and shrinkage during summer. It is but natural that engineers are apprehensive of using swelling soils as a foundation material as well as for construction material.

This problem has assumed economic importance at the national level because one fifth (20%) of the surfacial depth of the country are swelling soils. Heavy investments are made for the construction of Water Resources Structures. A large number of dams and embankments are constructed in the regions of swelling soils needing gigantic quantity of earth for construction. Thus it is clear that various types of civil engineering activities are taking place on swelling soils. In the past few plan periods, these activities have increased and it is definite that these activities will be increasing in future also.

Therefore it becomes essential to study the nature, property, behaviour of these soils so that engineer can design the structures as per the behaviour of soil and also to study the remedial measures required for decreasing the volumetric changes in these soils due to change in the moisture content.

To tackle the problem of swelling soils, the following studies have been attempted in this dissertation work:

1. Nature and properties of swelling soil
2. Engineering behaviour of the swelling soil deposits under various seasonal conditions and
3. To develop mechanical / chemical methods to tackle the problem.

LITERATURE REVIEW

2.1 COMPOSITION OF SWELLING SOILS

Most soil classification systems arbitrarily define clay particals as having an effective diameter of 2 micron (0.002 mm) or less. Particle size alone does not determine clay mineral. Probably the most important grain property of fine grained soil is the mineralogical composition. The clay minerals are a group of complex alumino-silicates, mainly formed during the chemical weathering of primary minerals having tiny platy or flaky structure. These tiny plates are in turn composed of different layers of crystal sheet which have repeating atomic structure. Because of their small size and flat shape, they have very large specific surface. There is usually a negative electric charge on the crystal surface and the electro-chemical forces on these surfaces are therefore predominant in determining the engineering property.

The three most important groups of clay minerals are Montmorillonite, Illite and Kaolinite. Montmorillonite in the clay mineral presents most of the expansive soil problem [CHEN. F.H 1975].

The name 'Montmorillonite'is used currently both as a group name for all clay minerals with an expanding lattice, and also as a specific mineral name. Absorption of water by clay leads to expansion. From the mineralogical stand point the magnitude of expansion depends upon the kind and amount of clay mineral present, their exchangeable ions and the internal structure. Table2.1 gives the relative swelling properties of the 3 clay minerals after CSMRS New Delhi 1998 report.

Table 2.1 Properties of clay minerals

Property	Kaolinite	Illite	Montmorillonite
Particle Dia (Micron)	0.5-4	0.5-10	0.5-10
Particle thickness (Micron)	0.5-2	0.003-0.1	0.001
Specific surface m^2/g	5-30	65-100	600-800
Cation exchange capacity meq/100g	3-15	10-40	80-150
Max pressure (% of surcharge pressure)			
(1) 9.5 kpa ($0.98 t/m^2$)	Negligible	350	1500
(2) 19 kpa ($1.96 t/m^2$)	Negligible	150	350

Two types of crystals are encountered in swelling type of clay as shown in Fig. 2.1 & 2.2:

(a) Tetrahedra Sheet

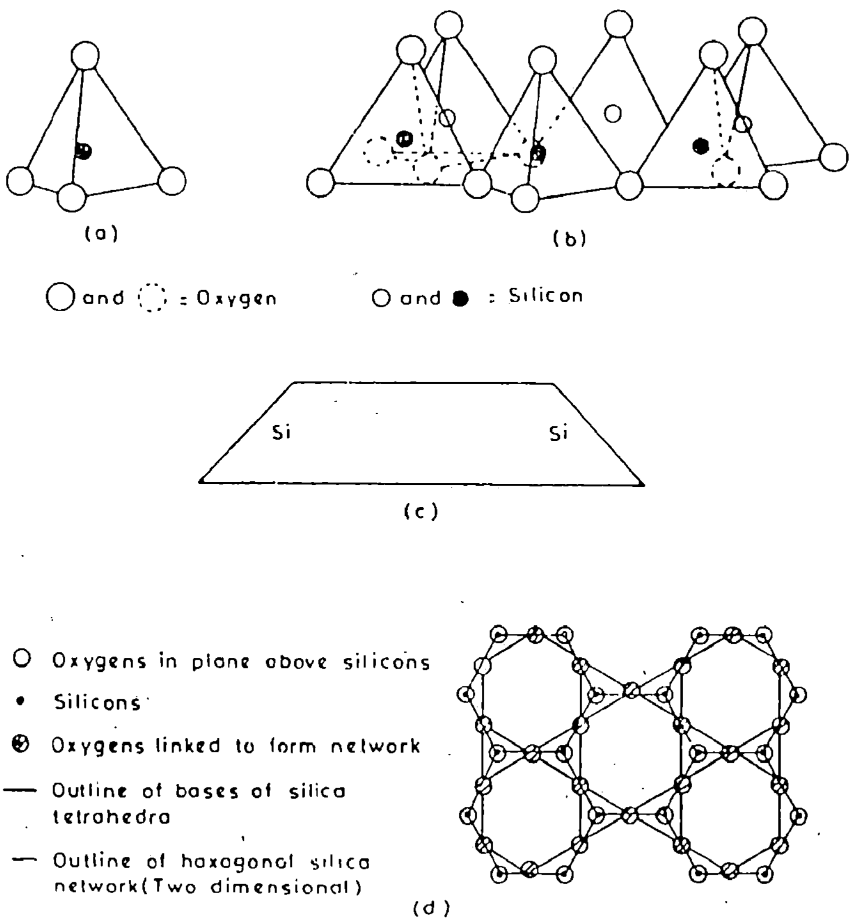


Fig. 2·1 (a) Single silica tetrahedron
(b) Isometric view of the tetrahedral or silica sheet
(c) Symbolic representation of the silica sheet
(d) Top view of the silica sheet (after Warshaw and Roy, 1961)

(b) Octahedre Sheet

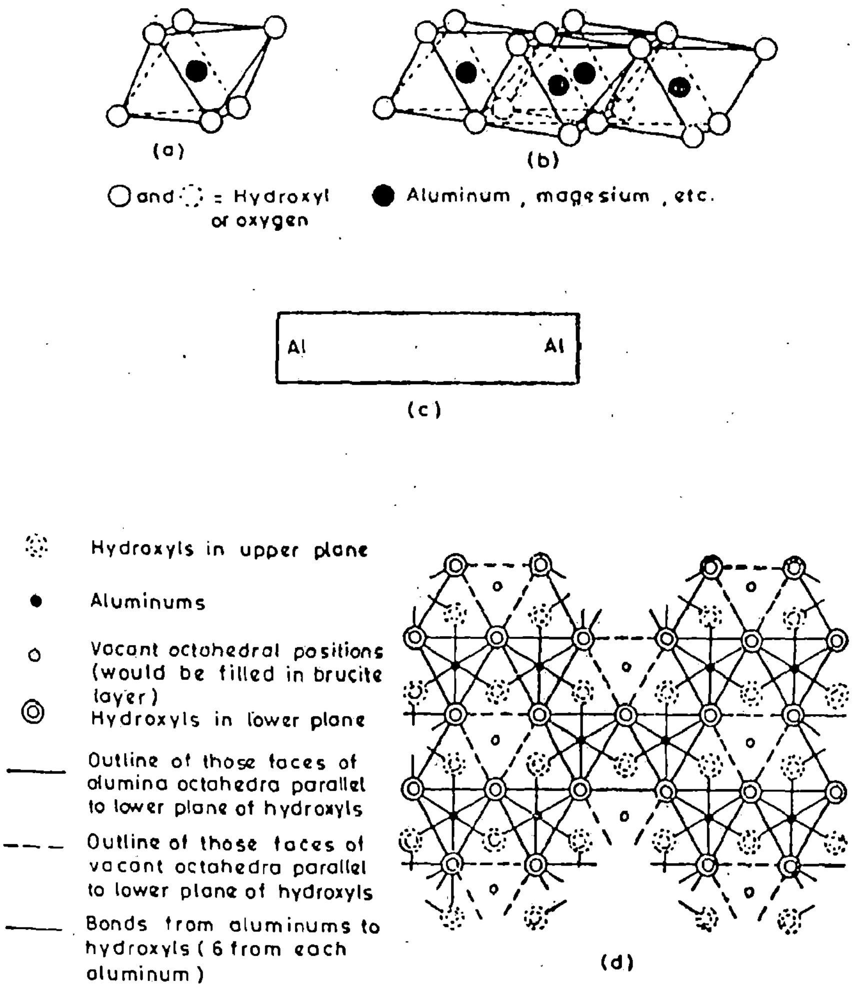


Fig.2.2 (a) Single aluminum (or magnesium) octahedron
(b) Isometric view of the octahedral sheet
(c) Symbolic representation of the octahedral or alumina (or magnesia) sheet
(d) Top view of the octahedral sheet (after Warshaw and Roy, 1961)

The tetrahedra sheet is basically a continuation of silica tetrahedra units which consist of four oxygen (0) atoms at the corners, surrounding a single silicon atom. The octaheadra sheet is basically continuation of octahedra units having an Aluminum or Magnesium, Iron or other atom at the centre surrounded by six oxygen (0) atoms with hydroxyl (OH) group at the corner.

All the minerals contain the above two sheets in different combination and orientation and certain cations in both the sheets. (C. Sudhindra, Ratnam M. 1986)

For engineering purposes and particularly while dealing with swelling soils, it is sufficient to have background information on few key minerals as detailed below:

Montmorillonite

It contains two silica sheets interposed with an Alumina sheet. Schematically it is represented as per Figure No. 2.3.

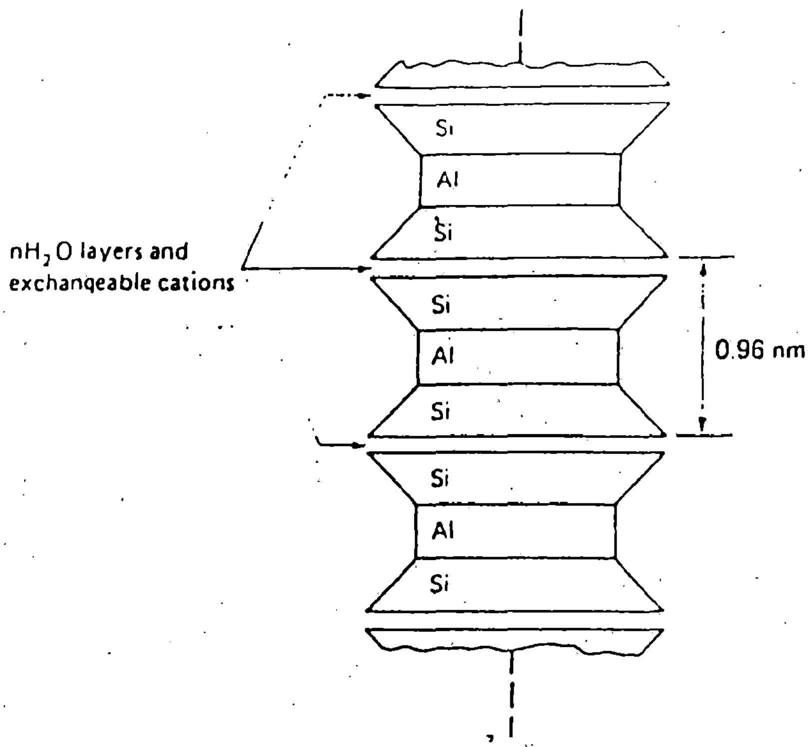


Figure 2.3- Schematic diagram of the structure of montmorillonite (after Lambe, 1953).

Illite

It has structure similar to montmorillonite, except that the layers are strongly held together by Potassium ion (K). These potassium ions have about the exact size as the hexagonal holes in the silica sheet and fit in properly.

Schematic diagram is as given as per Figure No.-2.4.

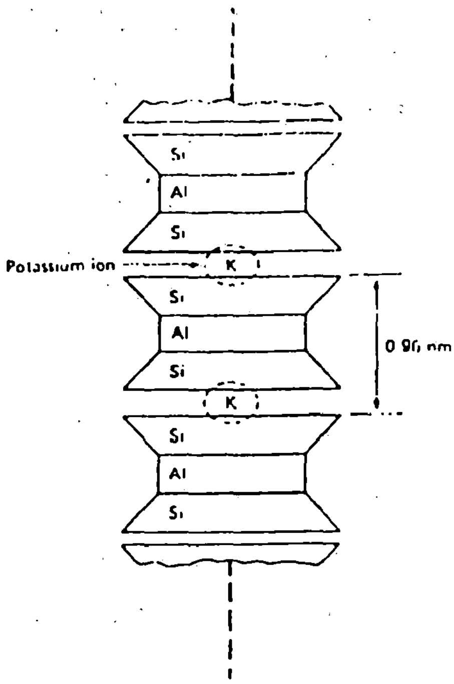


Figure 2.4- Schematic diagram of illite (after Lambe, 1953).

Kaolinite

As the tip of one tetrahedral sheet is bonded to the layer of octahedral sheet to form a layer of about 0.72 mm thickness as shown in Fig. 2.5., these layers are held together by hydrogen bonding between the hydro-oxyls of theoctahedral sheet and oxygen of the tetrahedral sheet. A typical kaolinite is about 70-100 layers thick.

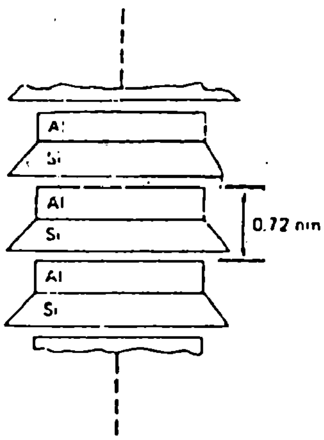


Figure 2.5 - Schematic diagram of the structure of kaolinite (after Lambe, 1953).

2.2 MECHANISM OF SWELLING

Mechanism of swelling is a complex process. The simplest approach to know this is to understand the nature of the mineral present at the type of interacting forces operating between the layers and the surrounding water medium with these layers. Knowledge of the mechanism of swelling in a soil undoubtedly needs background information on the type of bonds between atoms encountered in the soil which are usually classified as primary and secondary.

The primary bonds are strong when atoms are amenable for separation only with the application of large amount of energy.

Under the category of secondary bonds, the hydrogen bonding and vander waal forces are of importance the - vander waal forces are very weak forces.

The water and soil particles being not inert chemically, interact with each other inturn influencing the physical and the physico-chemical behaviour of the soil. Thus understanding of each interacting force between the clay particles including their interactions with water will be necessary to know the swelling characteristics of soils.

Swelling occurs due to the water molecule entering in the intermediate layers. Considering an example of Montmorillonite present in the soil, the two adjacent layers are held together by weak vander waal's forces. Several monolayers of water molecule enter the interspace and cause swelling. Incase of Illite, since the successive layers are held together by potassium ions through strong electrostatic forces, the separation of layers is difficult. Kaolinite has the successive layers held together by fairly strong hydrogen bonding between the hydroxyl of the octahedral sheet and the oxygen of the tetrahedral sheet, thus preventing hydration and hence the mineral resist swelling action. (C. Sudhindra, 1986)

2.3 COMPOSITION AND CHEMICAL PROPERTIES OF SWELLING SOIL

The parent material associated with expansive soils is either basic igneous or sedimentry rock. The formation of Indian Black cotton soil is usually associated geologically with Basalt. However they sometimes occur with granite, gnesis, shale, sandstone, slate or limestone. The thickness of black cotton soil cover is highly variable from 30 cms to 15m.

The Indian swelling soils are classified as clay or silty clay made up of 30-70% clay, 17.45% silt & 10-25% sand. The clay fraction is very rich in silica. They are heavy, montmorillonite being the predomant clay mineral. In basic igneous rocks, montmorillonite the mineral that causes excessive swelling and shrinkage of the soil, is formed by decomposition of felspar and pyroxene and in sedimentry rocks, it is a constituent of the rock itself.

The range of chemical composition is given in Table No.2.2. (Katti 1979)

Table No. 2.2 Chemical Properties of Swelling Soils

Description	Formula	Range %
Silica	SiO_2	45-58
Alumina	Al_2O_3	13-18
Ferric oxide	Fe_2O_3	7-15
Lime	CaO	1-8
Magnesium oxide	MgO	2-5
Carbonate	CO_3	0.5-5
Sulphate	SO_3	1-2
Loss on ignition	-	5-17
pH value	-	6-8

It has been proved by Roy –Char 1969, Sridharan –Rao 1973 and Katti 1979. by differential thermal analysis and X ray diffraction pattern analysis that montmorillonite is the predominant clay mineral in these soils. The base exchange capacity of clay fraction is in the range of 100 to 130 m.eq/100 gm.

The swelling silts are generally dark in color but sometimes they are light brown or yellowish at depths below one meter. At greater depth they sometimes contain less clay and are relatively rich in carbonate usually formed in the form of small `Kankar' modules. These light coloured soils with lower clay content have often been mistakenly considered to be non-swelling and have been used as foundation strata. They are however expansive and color cannot be used as a guide to the expansiveness of a soil.

The range of physical properties of Swelling Soil is as follows:

Liquid Limit	:	40-100% (exceptionally higher for Bentonites)
Plasticity Index	:	20-60%
Shrinkage Limit	:	6-18%
Free Swell Index	:	20-150%

(Chandra Prakash 1988)

2.4 IDENTIFICATION AND CLASSIFICATION OF SWELLING SOILS:

2.4.1 General

The black cotton soil deposit shows the greatest desiccation to a depth of 2 m. Field measurements revealed that the depth of moisture change may vary from 1.5 to 3.5 m depth or even more. Gupta et al measured changes in water content in the top 1.5 m between 11 and 18% whereas below 1.5 m the variation was less than 8 percent.

For identification and classification of black cotton soils in terms of degree of expansiveness numerous procedures ranging from visual examination and simple laboratory tests to sophisticated electron micrographs, similar to other parts of the world, have been used. The purpose of identification and classification is to qualitatively characterize potential volume change for assessing the likelihood of a serious design problem and to plan detailed investigations as well as arrive at decision regarding foundations. Based on the observations of cracked buildings Sharma and Soneja suggested the relationship between differential free swell (DFS) and the degree of expansiveness as in Table 2.3.

Table No. 2.3 Relationship between Differential Free Swell and Expansiveness

DFS(%)	Degree of expansiveness
20	Low
20-35	Moderate
30-50	High
>50	Very high

This method is normally substantiated with the classification given in Table 2.4.

Table 2.4 Generalised Classification of Swelling Soils

Degree of expansiveness	Liquid limit	Plasticity index	Shrinkage index	Swelling potential (%)
Low	< 30	<12	<20	0-1.5
Medium	30-40	12-23	20-30	1.5-5
High	40-60	23-32	30-60	5.0-25
Very high	>60	>32	>60	>25

Shrinkage Index= Liquid Limit- Shrinkage Limit

In Table 2.4 the swelling or swell potential or the per cent free swell is defined as the percentage swell of a laterally confined sample soaked under 1 psi (7 kN/m^2) surcharge after being compacted at optimum moisture content to maximum dry density in the standard AASHO compaction test. According to this definition it is obvious that per cent free swell is determined for the disturbed soil specimen and as such it does not represent actual conditions in the field.

Also, the per cent free swell is either determined directly in the laboratory or by the correlations established based on the tests on black cotton soils. It should be borne in mind that while some of these correlations are based on the measurement of per cent free swell in the laboratory on disturbed sample following the same definition as given above, some are based on the measurement on undisturbed sample and following different definition. Snethen (1984) compared the values predicted by various correlations with the measured values in the laboratory on undisturbed samples and suggested the following definition for use to bring uniformity:

"Potential swell is the equilibrium vertical volume change on deformation from an oedometer type test (i.e. total lateral confinement) expressed as percentage of original height of an undisturbed specimen from its natural moisture content and density to a saturation under an applied load equal to the insitu overburden pressure.

2.4.2 Identification of Swelling Soils in the Field

- Colour may be black, gray, brownish & yellowish as in case of Bentonite.
- During summer wide and deep map type crack is observed.
- During heavy rains when such soils get saturated, it would be very difficult to walk through these soils because of high stickiness.
- Slope of the terrain is normally very flat: in the range of $0.2^{O.}$
- Drainage is very poor
- Vegetation: In India, the vegetation in such area may consists of thorny bushes, thorny trees (Babul), Cactus etc.

- Structures constructed on these deposits exhibit heaving of floor, lifting of columns and walls usually accompanied by cracking. Doors normally get jammed during rainy season.
- In case of canals whether it be in embankments in partial cuts or embankments in cuttings, bed heaving accompanied by cracking of the bed concrete is observed.
- Retaining structures show tilting and distress. Roads gets rutted.

2.4.3 Recognition of Expansive Soils

2.4.3.1 General

There are three different methods of classifying potentially expansive soils. The first, mineralogical identification, can be useful in the evaluation of the material but is not sufficient in itself when dealing with natural soils. The various methods of mineralogical identification are important in a research laboratory in exploring the basic properties of clays, but are impractical and uneconomical for practicing engineers. (Chen. F.H. 1975)

The second includes the indirect methods, such as the index property, and activity method which are valuable tools in evaluating the swelling property. Soil suction may prove to be very useful with more general application and improved testing techniques. None of the indirect methods should be used independently. Erroneous conclusions can be drawn without the benefit of direct tests.

The third method, direct measurement, offers the most useful data for a practicing engineer. The tests are simple to perform and do not require any costly and exotic laboratory equipment. A word of caution has been introduced here. Testing has been performed on a number of samples rather than of a few to avoid erroneous conclusions.

2.4.3.2 Mineralogical identification

The mineralogical composition of expansive soils has an important bearing on the swelling potential. The negative electric charges on the surface of the clay minerals, the strength of the interlayer bonding, and the cation exchange capacity all contribute to the swelling potential of the clay. Hence, it is claimed by the clay mineralogist that the swelling potential of any clay can be evaluated by identification of the constituent mineral of this clay. The five techniques which may be used are as follows:

X-ray diffraction,
Differential thermal analysis,
Dye adsorption,
Chemical analysis, and
Electron microscope resolution

The various methods listed above are generally used in combination. Using combination of the methods, the different types of clay minerals present in a given soil can be evaluated quantitatively. Unfortunately, though a great deal of research has been done in the various fields of mineralogical study, the test results require expert interpretation and the specialized apparatus required are costly and not economically available in most soil testing laboratories.

2.4.3.3 Single index method

Simple soil property tests can be used for the evaluation of the swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of sites in those areas having expansive soil. Such tests may include:

Atterberg limits tests,
Shrinkage tests,
Free swell tests etc.

Atterberg Limit Tests:

Atterberg limit tests demonstrated that plasticity index and liquid limit are useful indices for determining the swelling characteristics of most clays, the plasticity index can be used as a preliminary indication of swelling characteristics of most clays.

From the definition of swell potential (which is defined as the percentage swell of a laterally confined sample which has soaked under a surcharge of 1 pound per square inch after being compacted to maximum density at optimum moisture content according to the AASHO compaction test), the following simplified relationship has been developed (Seed, 1962)

$$S = 60K(PI)^{2.44}$$

in which:

S = Swell potential

K = 3.6×10^{-5} and is a constant.

PI = Plasticity Index

The above equation applies only to soil with clay content between 8 and 65 percent and the computed value is probably accurate to within about 33 percent of the laboratory determined swell potential.

Since liquid limit and swelling of clays both depend on the amount of water a clay tries to imbibe, it is not surprising that they are related.

Relation between swelling potential of clays and plasticity index has been established as follows:

Swelling potential	Plasticity index
Low	0 - 15
Medium	10 - 35
High	20 - 55
Very high	35 and above

While it may be true that high swelling soil will manifest high index property, the converse is not true.

Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. In theory it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential.

Potential expansiveness for various values of shrinkage limits and linear shrinkage have been suggested as follows: (Altmeyer, 1955)

Shrinkage limit as a percentage	**Linear shrinkage as a percentage**	**Degree of expansion**
Less than 10	Greater than 8	Critical
10-12	5-8	Marginal
Greater than 12	0-5	Non-critical

Free Swell Test

Free swell test consists of placing a known volume of dry soil in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of initial volume, is the free swell value. The swell test is very crude and was used in the early days when refined testing methods were not available.

Experiments indicated that a good grade of high swelling commercial bentonite will have a free swell value of from 1200 to 2000 percent. Researchers have suggested that soils having free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings.

Colloid Content

The grain size characteristics of clay appear to have a bearing on its swelling potential, particularly the colloid content. Seed, Woodward, and Lundgren believed that there is no correlation between swelling potential and percentage of clay sizes. However, for a given clay type, the amount of swell will increase with the amount of clay present in the soil as shown on figure 2.6.

For any given clay type, the relationship between the swelling potential and percentage of clay size can be expressed by the equation:

S	=	KC^x
Where S	=	Swelling potential, expressed as a percentage of swell under 1-psi surcharge for a sample compacted at optimum moisture content to maximum density in standard AASHO compaction test.
C	=	Percentage of clay sizes finer than 0.002 mm,
x	=	An exponent depending on the type of clay, and
K	=	Coefficient depending on the type of clay.

Where the quantity of the clay size particles is determined by a hydrometer test, the quality or kind of colloid which is reflected by x and K in the above equation, controls the amount of swell. Colloid content as well as Atterberg limits should be included in the routine laboratory investigation on expansive soils.

Classification method

By utilizing routine laboratory tests such as Atterberg limits, colloid contents, shrinkage limits, and others, the swelling potential can be evaluated without resorting to direct measurements.

The USBR Method developed by Holtz and Gibbs (1956) is based on the simultaneous consideration of several soil properties. The typical relationship of these properties with swelling potential are shown on figure 2.7.

Based on the curves presented in figure 2.7, Holtz (1959) proposed the identification criteria of expansive clay as per Table No. 2.5 follows:

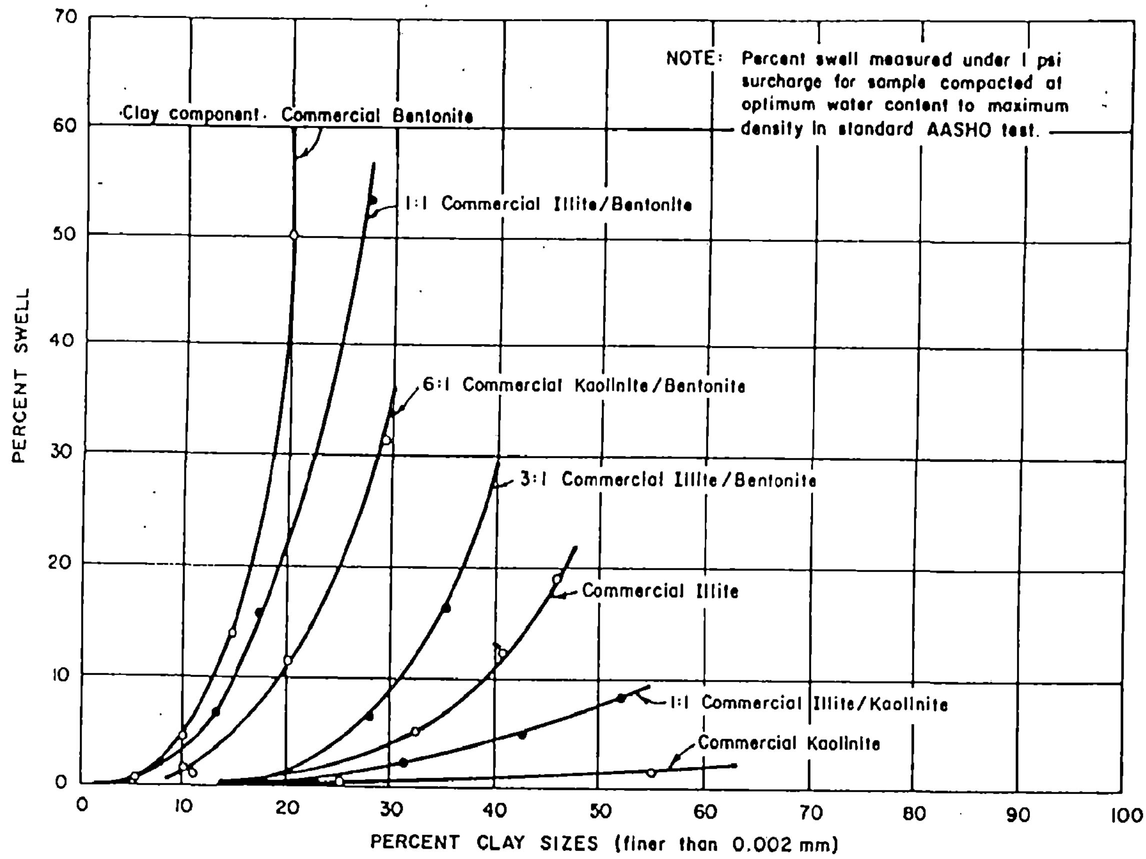


Figure 2·6 Relationship between percentage of swell and percentage of clay sizes for experimental soils. (After Seed, Woodward & Lundgren)

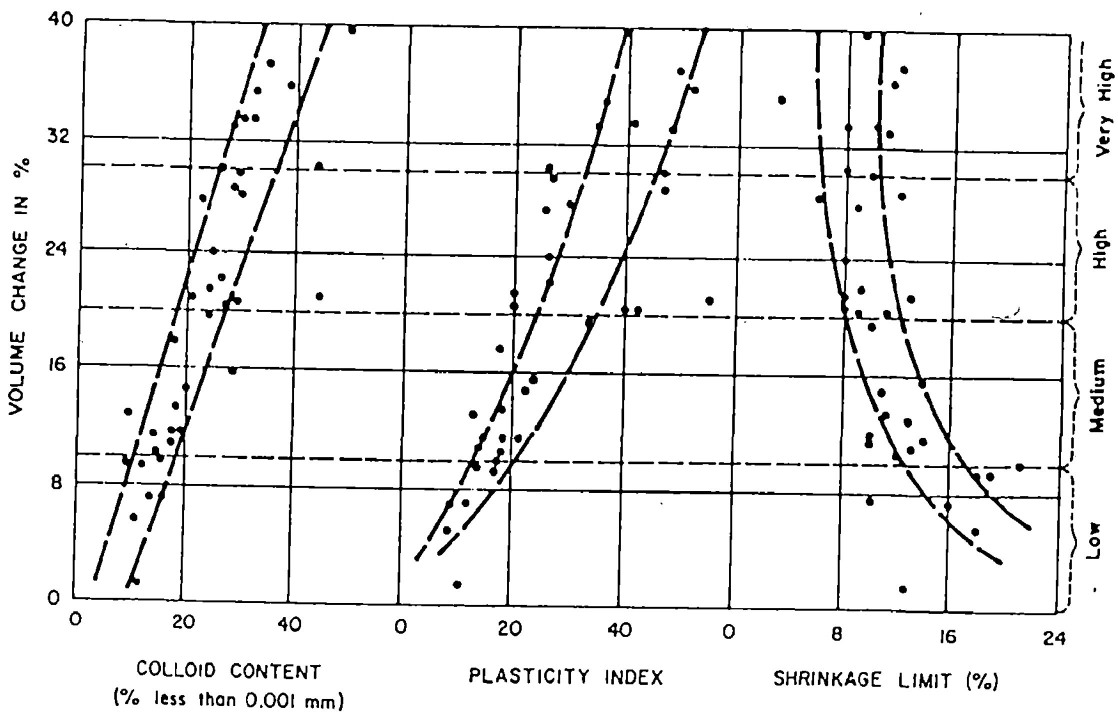


Figure 2·7 Relation of volume change to colloid content, plasticity index, and shrinkage limit (air-dry to saturated condition under a load of 1 lb. per sq. in.) (After Holtz and Gibbs)

Table 2.5 Data for making estimates of probable volume changes for expansive soils (After Holtz and Gibbs)

Colloid content %	Plasticity index %	Shrinkage limit %	Probable expansion % of total volume change	Degree of expansion
>28	>35	<11	>30	Very high
20-13	25-41	7-12	20-30	High
13-23	15-28	10-16	10-30	Medium
>15	<18	>15	<10	Low

From the test results of undisturbed samples by the researchers a regression curve can be fitted as shown on figure 2.8 The relationship between swell potential and plasticity index can be expressed as follows:

$$S = Be^{A(PI)}$$

In which $A = 0.0838$ and

$B = 0.2558$

From figure 2.8 it is seen that with increase of plasticity index, the increase of swelling potential is much less than predicted by Holtz and Gibbs or from Seed, Woodward and Lundgren.

PVC Rating or Indirect measurement of swelling potential of expansive soils has been approached by many investigator. The Ladd and Lambe method aided by a PVC (Potential volume change) meter is probably the simplest and quickest method, while the soil suction method is considered to be a new approach toward the measurement of swelling potential and swelling pressure. The determination of the potential volume change (PVC) of soil was developed by T W Lambe under the auspices of the Federal Housing Administration. Remolded samples were specified. The sample was first compacted in a fixed ring consolidometer with compaction effort of 2.64×10^3 kN/m^2. Then an initial pressure of 1400 kN/m^2 was applied, and water added to the sample which is partially restrained from vertical expansion by a proving ring. The proving ring reading is taken at the end of 2 hours. The reading is converted to pressure and is designated as Swell Index.

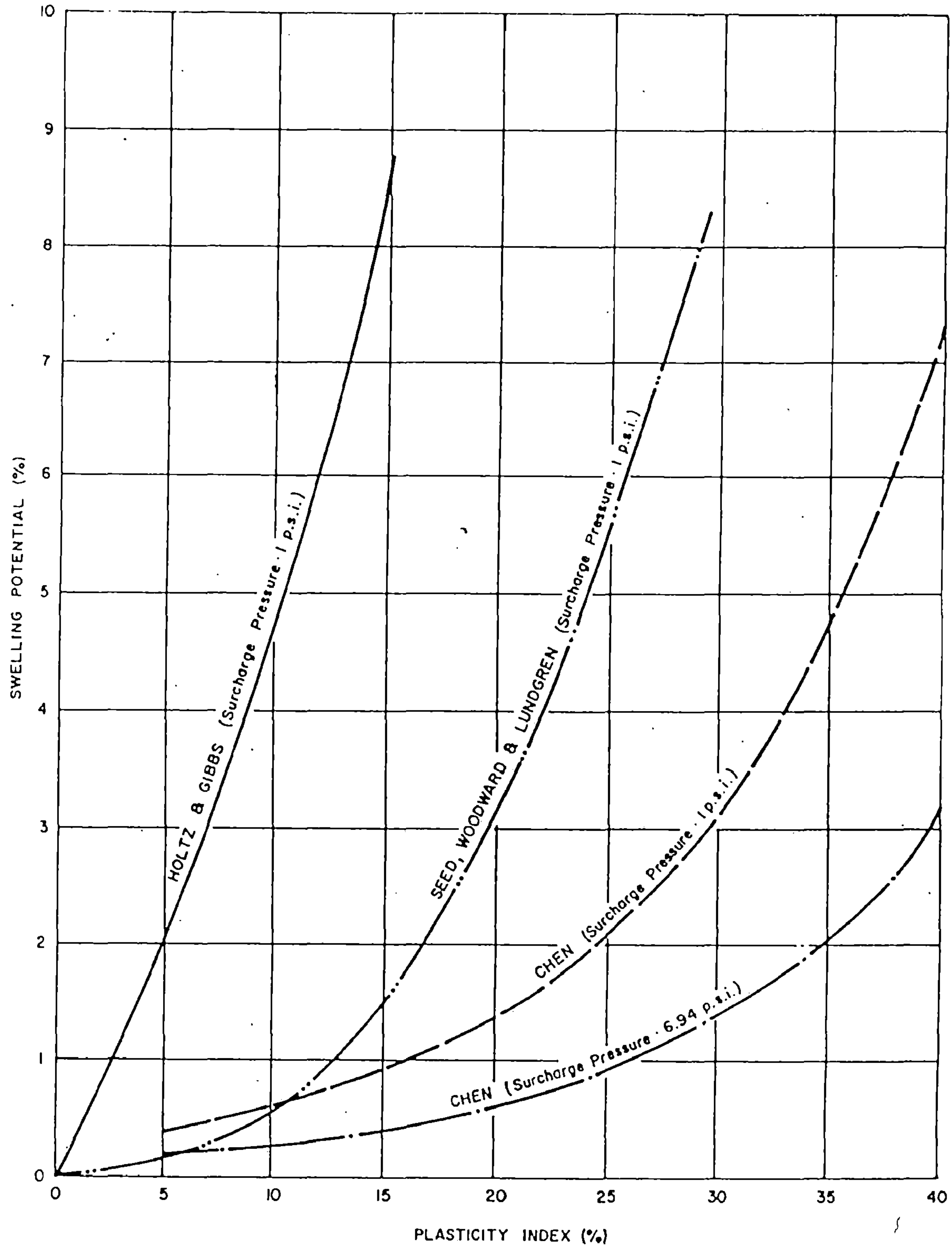


Figure 2-8 Relationship of volume change to plasticity index as predicted by Holtz, Seed, and Chen.

From figure 2.9 the swell index can be converted to potential volume change. Lambe established the following categories of PVC rating :

PVC Rating	Category
Less than	non critical
2 – 4	Marginal
4 - 6	Critical
Greater than 6	Very critical

It should be pointed out that the PVC meter test in itself does not measure the swell potential. The true swell potential of clay measured can be much greater than the indicated value. The PVC meter test should be used only as a comparison between various swelling soils.

2.4.3.4 Direct measurement

The most satisfactory and convenient method of determining the swelling potential and swelling pressure of an expansive clay is by direct measurement. Direct measurement of expansive soils can be achieved by the use of the conventional one-dimentional consolidometer. The consolidometer can be platform type, scale type, or other arrangement. The load is applied by direct weight as in the case of cantilever consolidometer. The soil sample is enclosed between two porous plates and confined in a metal ring. The diameter of the ring ranges from 3.8 to 6.0 cm depending upon the type of sampling device. The thickness of the sample ranges from 12-24 mm. The soil sample can be flooded both from the bottom and from the top. Vertical expansion measurement is reported as percentage of the initial height of the sample and is frequently referred to as the percent of swell.

Such a device enables an easy and accurate measurement of the swelling potential of a clay under various conditions. After the soil has reached its maximum volume increase, the sample can be reloaded and swelling pressure determined. Thus, swelling pressure can be evaluated easily without resorting to devices to hold the soil volume constant.

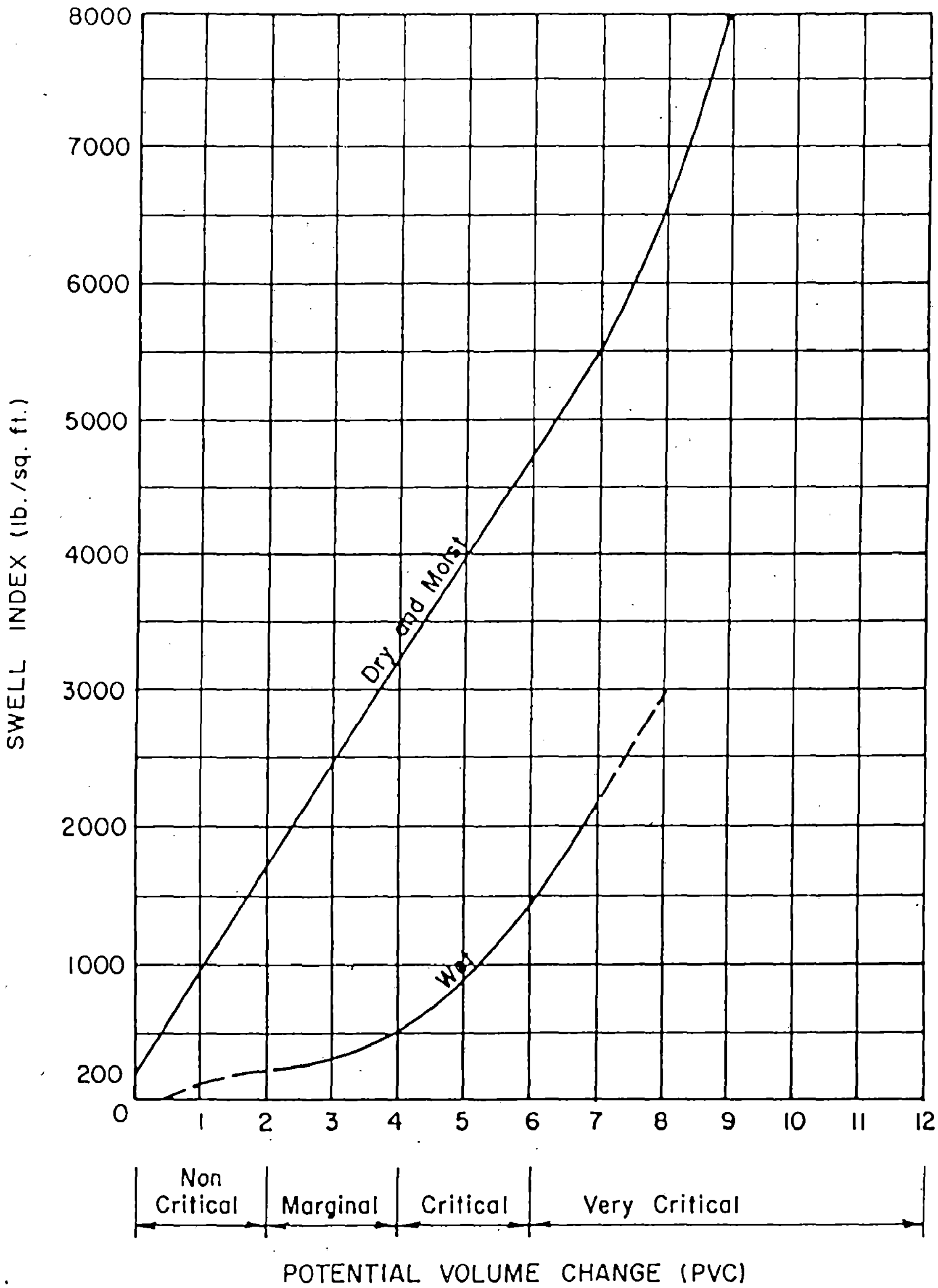


Figure 2·9 Swell index versus potential volume change. (From "FHA Soil PVC Meter Publication," Federal Housing Administration Publication No. 701)

A great deal of data has been accumulated in files of soil engineers, academic or governmental organisations on expansive tests using a consolidometer. Unfortunately, dissimilar test procedures have been used. Thus, it is difficult to evaluate and compare the test data. A standardisation of test procedure of a one-dimentional swell test does not appear difficult and will salvage much of the valuable data accumulated in the hands of the private consultants. In the performance of a typical swell test, the more important variables involved are as follows :

1. State of sample. For an undisturbed sample, this would include the condition of the sample, sampling method, and stress history of the sample. For remolded samples, this would include the method of compaction, curing time before and after compaction, and compaction density.
2. Moisture content. The lower the initial moisture content the higher the swell. The initial moisture content is affected by :
   (a) The time allowed for the sample to remain in the ring before wetting ,
   (b) The extent of evaporation allowed while the sample is in the ring, and
   (c) The temperature and humidity of the laboratory.
3. Surcharge load. Increasing the applied load will reduce the magnitude of swell. Surcharge load for most laboratory practice ranges from 1 to 10 psi. Sometimes, attempts were made to duplicate the surcharge load with the actual footing dead load.
4. Time allowed. The time required to fully complete the swell process may vary considerably and depends on the permeability of the clay, the molding water content, the dry density, and thickness of the sample. For an undisturbed sample having a thickness of 1 inch, it may require as much as several days to complete the total available swell.

Undoubtedly, the direct measurement method is the most important and reliable test on expansive soils. By standardising the above variables, a reliable and reproducible test can be obtained. (Chen F.H, 1975)

2.5 PHYSICAL PROPERTIES OF EXPANSIVE SOILS

It is well known to soil engineers that montmorillonite clays swell when the moisture content is increased, while swelling is absent or limited in illite and kaolinite. The types of soils, and the conditions under which the most critical situation exists, can be outlines as follows :

Moisture Content

Irrespective of high swelling potential, if the moisture content of the clay remains unchanged, there will be no volume change; and structures founded on clays with constant moisture content will not be subjected to movement caused by heaving. When the moisture content of the clay is changed, volume expansion, both in the vertical and horizontal direction, will take place. Complete saturation is not necessary to accomplish swelling. Slight changes of moisture content, in the magnitude of only 1 to 2 percent, are sufficient to cause detrimental swelling. In the laboratory, clay samples swell in the consolidometer with slight increase of humidity. It is known that floor slabs founded on expansive soils cracked most severely when the moisture content increased slightly due to local wetting. If the floor slab is flooded, as in the case of a rising water table, the floor will heave but the extent of cracking will not be severe.

The initial moisture content of the expansive soils controls the amount of swelling. This is true both for soils in undisturbed and in remolded states.

Very dry clays with natural moisture content below 15 percent usually indicate danger. Such clays will easily absorb moisture to as high as 35 percent with resultant damaging expansion to structures. Conversely, clays with mositure contents above 30 percent indicate that most of the expansion has already taken place and further expansion will be small. However, moist clays may desiccate due to lowering of water table or other changes in physical conditions and upon subsequent wetting will again exhibit swelling potential.

Dry density

Directly related to initial moisture content, the dry density of the clay is another index of expansion. Soils with dry densities in excess of 110 pcf generally exhibit high swelling potential. Remarks made by excavators complaining that the soils are as hard as a rock is an indication that soils inevitably will present expansion problems.

The dry density of the clays is also reflected by the standard penetration resistance test results. Clays with penetration resistance in excess of 15 usually possess some swelling potential.

Index properties

It is more convenient to correlate the expansive properties with the percentage of silt and clay (-200), liquid limit, and field penetration resistance. Since most lightly loaded structures will exert a maximum dead load pressure of about 48 kN/m^2 on the footings, it is realistic to use a vertical load of 48 kN/m^2 to gauge the swelling potential. Table 2.6 is a guide for estimating the probable volume changes of expansive soils.

Table No. 2.6 Index properties and expansiveness

% passing no. 200 sieve	Liquid limit %	Standard pentation resistance	Probable expansion % total volume change	Degree of expansion
>35	>60	>30	>10	Very high
60-95	40-60	20-30	3-10	High
30-60	30-40	10-20	1-5	Medium
<30	<30	<30	<1	Low

The simplified classification of the expansive properties can be conveniently used by engineers as a guide for the choice of type of foundation on expansive soils. For example, for soils with a low degree of expansion, spread footing type foundations can usually be used, if sufficient reinforcement is provided in the foundation walls to compensate for slight movements. For soils of medium degree of expansion, individual footings or pads can be used where the dead load of the structure can be concentrated to an intensity of 144-240 kN/m^2 For soils of high- to-very-high degree of expansion, special consideration should be given as to the foundation type.

Fatigue of swelling

A clay sample is subjected to full swelling in the consolidometer, allowed to desiccate to its initial moisture content, then is saturated again. This is repeated for a number of cycles. It was observed that the soil showed signs of fatigue after each cycle of drying and wetting. It has been noted that structures founded on expansive clays which have undergone seasonal movement due to wetting and drying have a tendency to reach a point of stabilisation after a number of years. The fatigue of swelling probably can furnish the answer. Figure2.10 shows a typical laboratory fatigue curve of swelling.

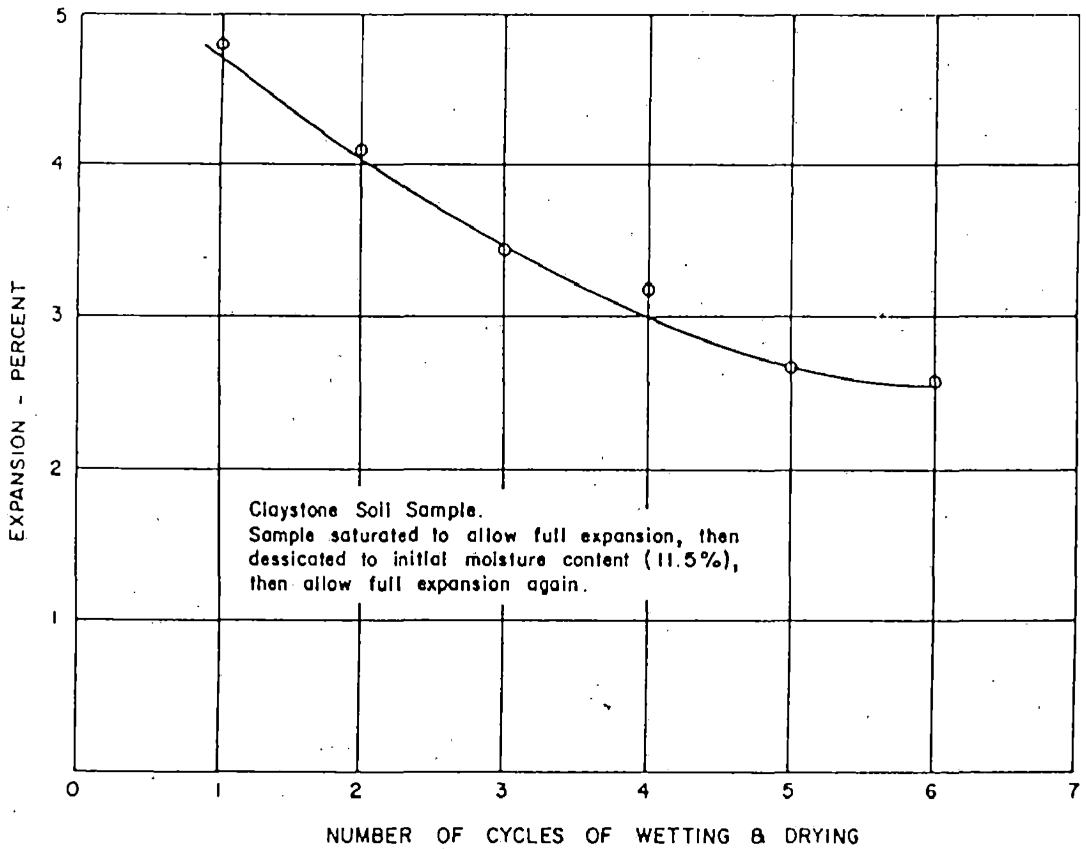


Figure 2.10Fatigue of swelling (After Chen, 1965)

EXPERIMENTAL INVESTIGATION AND ANALYSIS OF TEST DATA

3.1 GENERAL

In order to study the behaviour of swelling soils following laboratory investigation had been planned and conducted on samples collected from Indore, M.P., (Sample No. 1), Yamuna Nagar, Haryana (Sample No. 2), Jaisalmer, Rajasthan (Sample No. 3). Various tests as per details in table no. 3.1 have been conducted according to the procedure laid down in relevant Indian Standards given in the last column of the table to determine the index properties and engineering properties of soils. It will be seen from the table 3.1 that most of the test like Pipette analysis, Specific gravity, Atterberg limit, Shrinkage limit, Proctor compaction, One dimension consolidation and Triaxial test are standard tests for soils, except for the last two tests i.e. Free swell index and Swelling pressure test for swelling soils. Accordingly it is considered suitable to include the procedure for these tests in this chapter in paragraph 3.2 & 3.3.

Table No. 3.1 Details of test conducted

S. No.	Test	Purpose	IS Code No.
1.	Pipette analysis	To determine the % of clay present in the soil	IS : 2720 (Part 4)-1985
2.	Specific gravity	To find out the degree of saturation and unit weight of moist soil	IS : 2720 (Part 3/ Sec 1)-1980
3.	Atterberg limit	To determine the liquid limit, plastic limit and plasticity index so as to determine the classification of soil	IS : 2720 (Part 5)-1985
4.	Shrinkage limit	To determine the moisture content at which the remouled soil sample should be packed to determine the swelling pressure	IS : 2720 (Part 6)-1985
5.	Proctor compaction	To determine the optimum moisture content and max dry density	IS : 2720 (Part 7)-1980
6.	One dimension consolidation	To determine the coefficients of compressibility, consolidation permeability and compression index	IS : 2720 (Part 15)-1986
7.	Triaxial test	To determine the shear strength parameters	IS : 2720 (Part 12)-1981
8.	Free swell index ,	To determine the swelling nature of the soil	IS : 2720 (Part 40)-1977
9	Swelling pressure	--do--	IS: 2720 (Part 41)-1977

3.2 DETERMINATION OF FREE SWELL INDEX OF SOILS [AS PER IS : 2720 (PART 40) – 1977]

3.2.1 General

Free swell is the increase in volume of a soil, without any external constraints, on submergence in water. The possibility of damage to structures due to swelling of expensive clays need be identified, at the outset, by an investigation of those soils likely to possess undesirable expansion characteristics. Inferential testing is resorted to reflect the potential of the system to swell under different simulated conditions. Actual magnitude of swelling pressures developed depends upon the dry density, initial water content, surcharge loading and several other environmental factors.

3.2.2 Apparatus Required

Sieve : 425 – micron IS Sieve

Glass Graduated Cylinders - Two, 100-ml capacity

3.2.3 Procedure

Take two 10 gm soil specimens of oven dry soil passing through 425-micron IS Sieve.

Note : In the case of highly swelling soils, such as sodium benonites, the sample size may be 5 gm. Or alternatively a cylinder of 250 ml. Capacity may be used.

Each soil specimen shall be poured in each of the two glass graduated cylinders of 100 ml capacity. One cylinder shall then be filled with kerosene oil and the other with distilled water up to the 100 ml mark. After removal of entrapped air (by gentle shaking or stirring with a glass rod), the soils in both the cylinders shall be allowed to settle. Sufficient time (not less than 24 h) shall be allowed for the soil sample to attain equilibrium state of volume without any further change in the volume of the soils. The final volume of soils in each of the cylinders shall be read out.

3.2.4 Calculation

The level of the soil in the kerosene graduated cylinder shall be read as the original volume of the soil samples, kerosene being a non-polar liquid does not cause swelling of the soil. The level of the soil in the distilled water cylinder shall be read as the free swell level. The free swell index of the soil shall be calculated as follows :

$$\text{Free swell index, percent} = \frac{Vd - Vk}{Vk} \times 100$$

Where

Vd = the volume of soil specimen read from the graduated cylinder containing distilled water

Vk = the volume of soil specimen read from the graduated cylinder containing kerosene.

3.3 MEASUREMENT OF SWELLING PRESSURE OF SOILS [As per IS: (Part-41) – 1972]

3.3.1 General

The main purpose of swelling pressure test is to determine the intrinsic swelling pressure of the expansive soil tested. The expansive clays increase in their volume when they come in contact with water owing to surface properties of these clay types. Light structures founded on these type of clays-popularly known in India as 'Black Cotton Soil', experience severe structural damage due to the swelling of the subsoil. Since the intrinsic swelling pressure is to be associated with the design of structures against such damages, measurement of swelling pressure assumes importance.

The swelling pressure is dependent upon several factors, namely, (a) the type and amount of clay in the soil and the nature of the clay mineral (b) the initial water content and dry density, (c) the nature of pore fluid, (d) the stress history of the soil including the confining pressure, and (e) drying and wetting cycles to which the soils have been subjected to Besides, the dependence of swelling pressure on volume change makes a precise measurement of swelling pressure difficult.

3.3.2 Scope

This standard covers the laboratory method of conducting one-dimensional swelling pressure test using either fixed or the floating rings on both undisturbed or remoulded soils in the partially saturated condition to determine the swelling pressure of the soil. Two methods, namely, consolidometer method in which the volume change of the soil is permitted and the corresponding pressure required to bring back the soil to its original volume is measured and the constant volume method in which the volume change is prevented and the consequent pressure is measured, are covered.

3.3.3 Terminology

For the purpose of this standard, the following definition shall apply as per IS code

3.3.3.1 Swelling Pressure

The pressure which the expansive soil exerts, if the soil is not allowed to swell or the volume change of the soils is arrested.

3.3.3 Consolidometer Method

3.3.4.1 Apparatus and equipment

Consolidometer

A device to hold the sample in a ring either fixed or floating with porous stones (or ceramic discs) on each face of the sample. A consolidometer shall also provide means for submerging the sample, for applying a vertical load and for measuring the change in the thickness of the specimen. The provision for fixing of the dial gauge shall be rigid; in no case shall the dial gauge be fixed to a cantilevered arm. Suitable provision shall be made to enable the dial gauge to be fixed in such a way that the dial gauge records accurately the vertical expansion of the specimen.

Specimen Diameter

The specimen shall be 60 mm in diameter (specimens of diameter 50,70 and 100 mm may also be used in special case).

Specimen Thickness

The specimen shall be at least 20 mm thick in all cases. However, the thickness shall not be less then 10 times the maximum diameter of the grains in the soil specimen . the diameter to thickness ration shall be a minimum of 3.

Ring

The ring shall be made of a material, which is non-corrosive in relation to the soil tested. The inner surface shall be highly polished or coated with a thin coating of silicon grease or with a low-friction material. The thickness of the ring shall be such that under assumed hydrostatic stress conditions in the sample, the change in diameter of the ring will not exceed 0.03 percent under the maximum load applied during the test. The ring shall have one edge bevelled suitably so that the sample is pressed into the ring with least disturbance. The ring shall be placed with its cutting edge upwards in the consolidometer and clamped with a special clamp which should in no way damage the sharp edge. The clamp should be made circular with central hole equal in diameter of the porous stone and should be perfectly concentric with the sample. The ring shall be provided with a collar of internal diameter same as that of the ring and of effective height 20mm. The collar shall rest securely on the specimen ring.

Porous Stones

The stones shall be of silicon carbide or aluminum oxide and of medium grade. It shall have a high permeability compared to that of the soil being tested. The diameter of the top stone shall be 0.2 to 0.5 mm less than the internal diameter of the ring. The thickness of the stone shall be a min of 15 mm. The top stone shell be loaded through a corrosion-resistant plate of sufficient rigidity and of minimum thickness 10mm to prevent breakage of the stone. The loading plate shall have suitable holes for free drainage water.

Dial Gauge

Accurate of 0.01 mm with a traverse of at least 20mm

Water Reservoir

To keep the soil sample submerged.

Moisture Room

For storing samples and for preparing samples in climates where there is likelihood of excessive moisture loss during preparation (optional).

Soil Trimming Tools

Fine wire-saw knife, spatula etc, for trimming sample to fit into the inside diameter of the consilidometer ring with minimum disturbance.

Oven

Thermostatically controlled oven with interior of non-corroding material to maintain the temperature between 105 and 110^0C,

Desiccator

With any desiccating agent other than sulfuric acid.

Balance

Sensitive to 0.01 g.

Containers

For water content determination.

3.3.5 Preparation Of Test Specimen

3.3.5.1 Preparation of specimen from undisturbed soil samples

The container ring shall be cleaned and weighted empty. From one end of the undisturbed soil sample about 30mm, or more if desired, of the soil sample if desired shall be cut off and rejected. The specimen shall be cut off either from the undisturbed tube sample or from block sample, the later generally being more representatives of the field conditions. In either case, the consolidation ring should be gradually inserted in the sample by pressing with hands and carefully removing the material around the ring. The soil specimen so cut shall project as far as 10mm on either side of the ring. The specimen shall then be trimmed smooth and flush with the top and bottom of the ring. Any voids in the specimen caused due to removal of gravel or limestone pieces shall be filled back by pressing lightly the loose soil in the voids, care being taken to see that the specimen is not affected. The container ring shall be wiped clear of any soil sticking to outside and weighed again with the soil. The whole process should be quick to ensure minimum loss of moisture and if possible shall be carried out in the moisture room. Three representative specimens from the soil trimming shall be taken in moisture content cans and their moisture content is determined.

3.3.5.2 Preparation of specimen from disturbed soil sample

In case, where it is necessary to use disturbed soil samples the soil sample shall be compacted to desired (field) density and water content in a standard compaction in a standard compaction proctor mould. Samples of suitable of suitable sizes are cut from it as given in note 1.

Note-1

Since the swelling pressure of the soil is very much influenced by initial water content and dry density, it shall be ensured that, in the case of undisturbed soil samples, the specimen shall be collected from the field for test during the driest season of the year, namely, April, May and June, so that the swelling pressure recorded shall be maximum.

In the case of remoulded soil samples, the initial water content shall be at the shrinkage limit or filed water content so that the swelling pressure recorded shall be maximum.

3.3.6 Procedure

The porous stones shall be saturated. All surfaces of the consolidometer which are to be enclosed shall be moistened. The porous stones shall be saturated by boiling in distilled water for at least 15 minutes. The consolidometer shall be assembled with the soil specimen (in the ring) and porous stones at top and bottom of the specimen, providing a filter paper rendered wet (Whatman No.1 or equivalent) between the soil specimen and the porous stone. The loading block shall then be positioned centrally on the top porous stone.

This assembly shall then be mounted on the loading frame such that the load when applied is transmitted to the soil specimen through the loading cap. The assembly shall be so centred that the load applied is axial.

In the case of the lever loading system, the apparatus shall be properly counterbalanced. If a jack with load measurements by platform scales is used as the loading system, the tare weight with the empty consolidation apparatus, excluding those parts which will be on top of the soil specimen, which rest on the platform shall be determined before filling the ring with the soil and this tare weight shall be added to the computed scale loads required to give the desired pressures at the time of loading the soils specimen.

The holder with the dial gauge to record the progressive vertical heave of the specimens under no load, shall then be screwed in place and adjusted in such a way that the dial gauge is near the end of its release run, allowing small margin for the compression of soil, if any.

An initial setting load of 50 gf/cm^2 (this include the weight of the porous stone and the loading pad) shall be placed on the loading hanger and the initial reading of the dial gauge shall be noted.

The system shall be connected to a water reservoir with the level of water in the reservoir being at about the same level as the soil specimen and water allowed to flow in the sample. The soil shall then be allowed to swell. The free swell reading shown by the dial gauge under the seating load of 5 kN/m^2(0.05 kg/cm^2) shell be recorded by at different time interval.

The dial gauge readings shall be taken till equilibrium is reached. The equilibrium swelling is normally reached over a period of 6 to 7 days in general for all expansive soils.

The swollen sample shell then be subjected to consolidation under different pressures. The compression dial readings shall be recorded till the dial readings attain a steady state for each load applied over the specimen. The consolidation loads shall be applied till the specimen attains its original volume.

3.3.7 Calculations and Report

The observed swelling dial reading shall be plotted with swelling dial reading as ordinates and corresponding pressure on the abscissa on natural scale. A smooth curve shall be drawn joining these points. The swelling pressure exerted by the soil specimen under zero swelling condition shall be obtained by interpolation and expressed in kN/m^2 (kgf/cm^2).

3.4 ANALYSIS OF TEST DATA

Analysis of test result of lab investigations on the 3 soil samples is as follows

3.4.1 Chemical Properties

The chemical properties shown in table 3.2 are for 3 swelling soil samples as determined in the lab. The SiO_2 content range between 45-58%, Al_2O_3 from 13-18%, CaO from 7 to 8.5% and MgO 4-4.8%, the pH value is from 8.5-8.9. Carbonate contents are of the order of 3-4%. Base exchange capacity is in the range of 84 to 130 meq/100g which shows the presence of Montmorillonite type of clay minerals.

Table No. 3.2 Chemical Properties Of Soils

Description	Sample No. 1	Sample No. 2	Sample No. 3
PH at 27^0	8.5	8.75	8.9
Carbonate content %	4.35	3.3	4.40
SiO_2 %	49.3	45.6	58.10
Al_2O_3 %	18.1	13.7	14.50
CaO %	7.4	6.9	8.5
MgO %	3.9	4.1	14.8
Fe_2O_3%	9.77	14.8	12.6
Sulphate %	1.6	1.1	1.9
Loss on ignition %	9.6	16.5	13.9
Base exchange capacity meq /100 g for fraction Finer than 2 μ	109.2	84.40	130

3.4.2 Grain Size and Index Properties

These soil samples contains 2 micron clay fractions varying between 80-90% silt 3-10% and gravel less than 10%. Liquid limit, Plastic limit, Plasticity Index and Shrinkage limit ranges are respectively 70-263%, 20-40% 21-223% and 6-20%.

Low value of shrinkage limit indicates large volume changes. Fig 3.1 to 3.9 show the results of Pipette Analysis, Atterberg limit and Compaction test.

3.4.3 Strength and Compressibility Properties

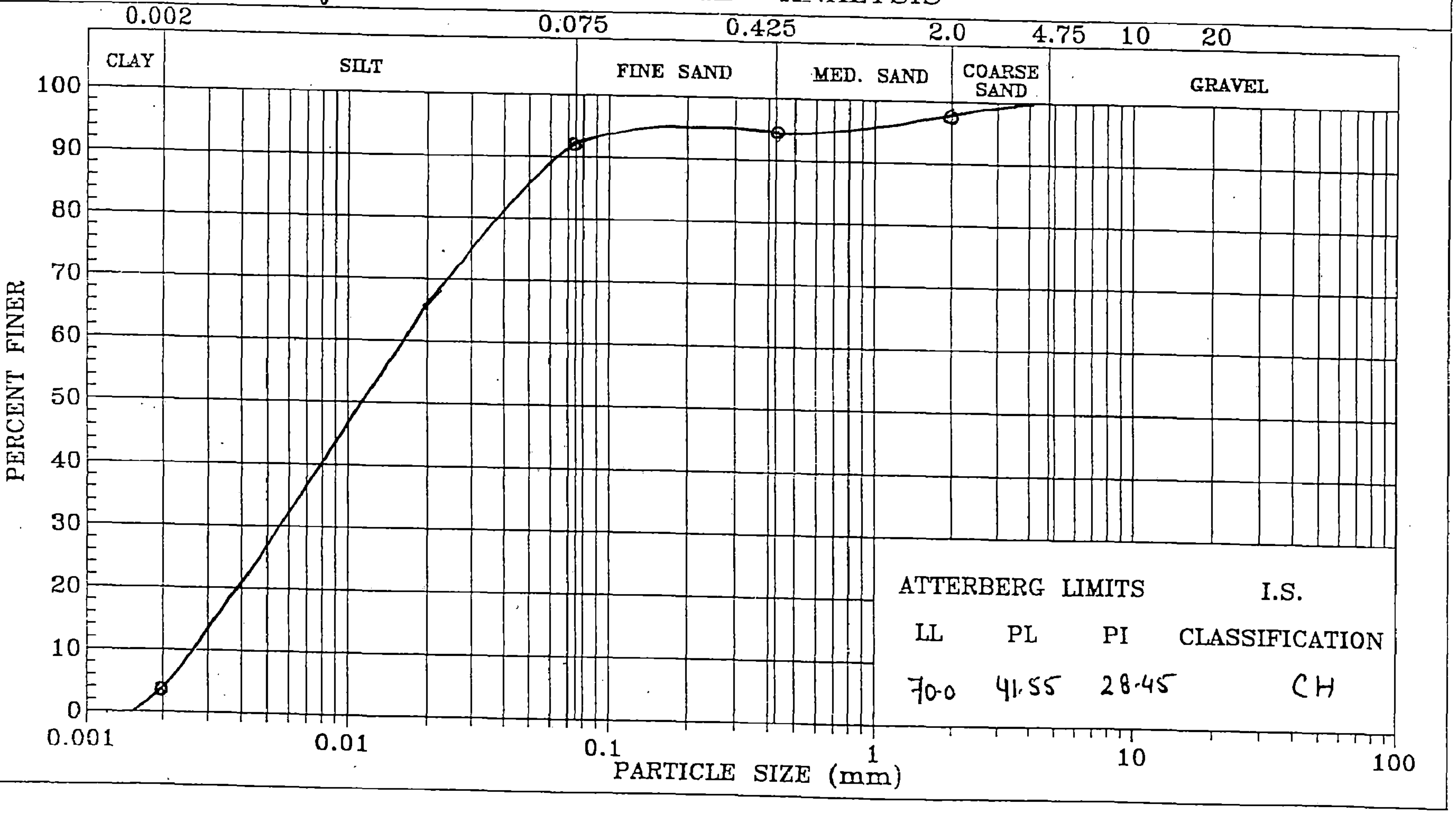
3.4.3.1 Triaxial test

Consolidated undrained triaxial compression test are performed on all the soil samples to determine the shear strength parameters with the measurement of pore water pressure. In this test, drainage take place during the application of confining pressure. After the complete drainage and consolidation of the specimen the deviator stress (additional axial load) is applied with drainage valve closed. This test is conduct on the cylinder specimen of nominal diameter 38mm with height 76mm. Mohr circle for the state of stress at failure in terms of effective stress for each of the 4 specimen of all the 3 samples is drawn and then a best common tangent to the 4 circle is drawn. The angle made by tangent with the horizontal is the angle of shearing resistance in terms of effective stress ϕ' and the intercept of the tangent makes with Y axis is cohesion in terms of effective stress C'.

It is seen from the Mohr circles as per Fig 3.10 to Fig. 3.12. that the shearing resistance varies from $12^0 - 20^0$ and cohesion varies from 18-40 kN/m^2/.

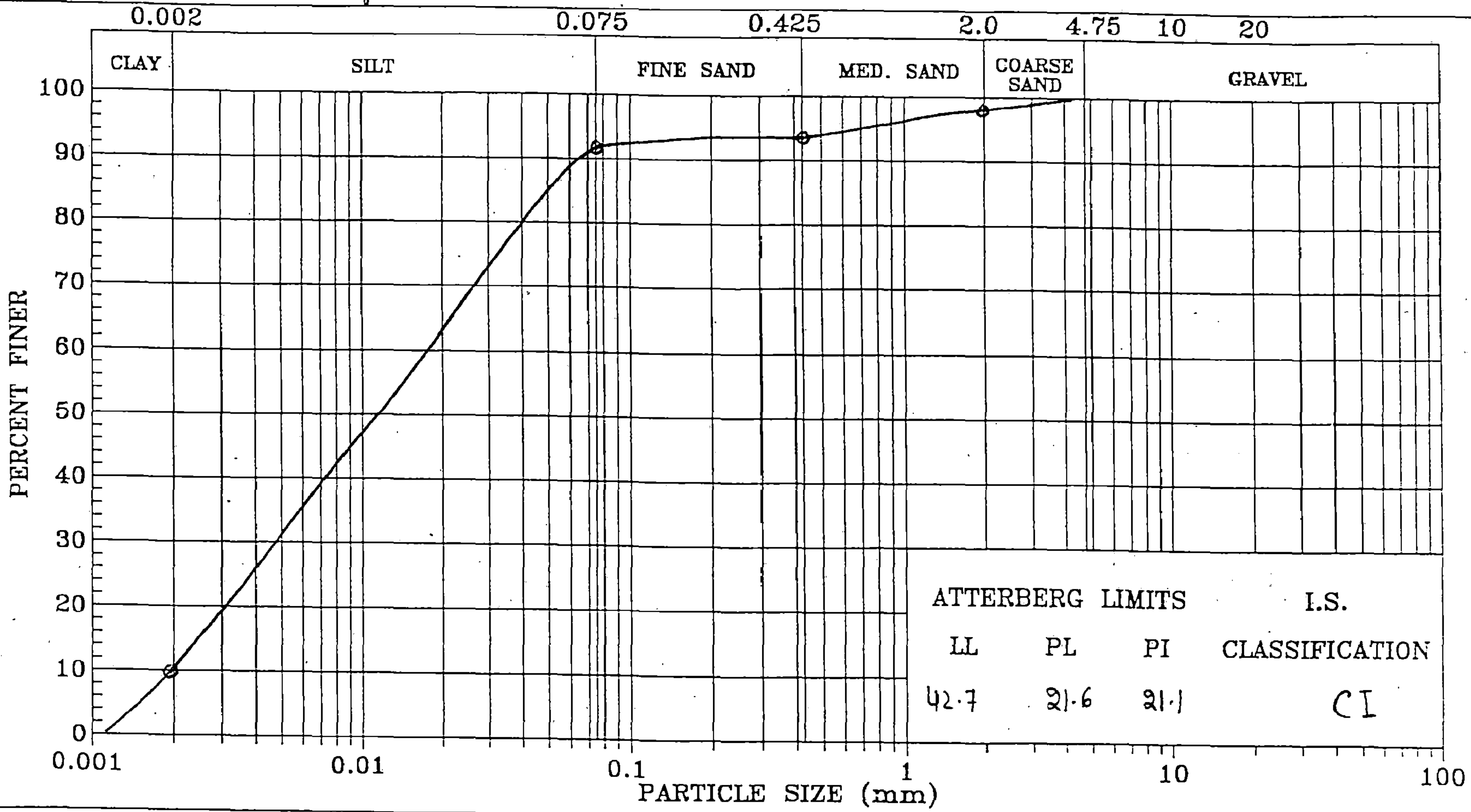
LAB. SAMPLE NO.: 1 LOCATION: PIT NO.: DEPTH (M) :

Fig No 31 GRAIN SIZE ANALYSIS



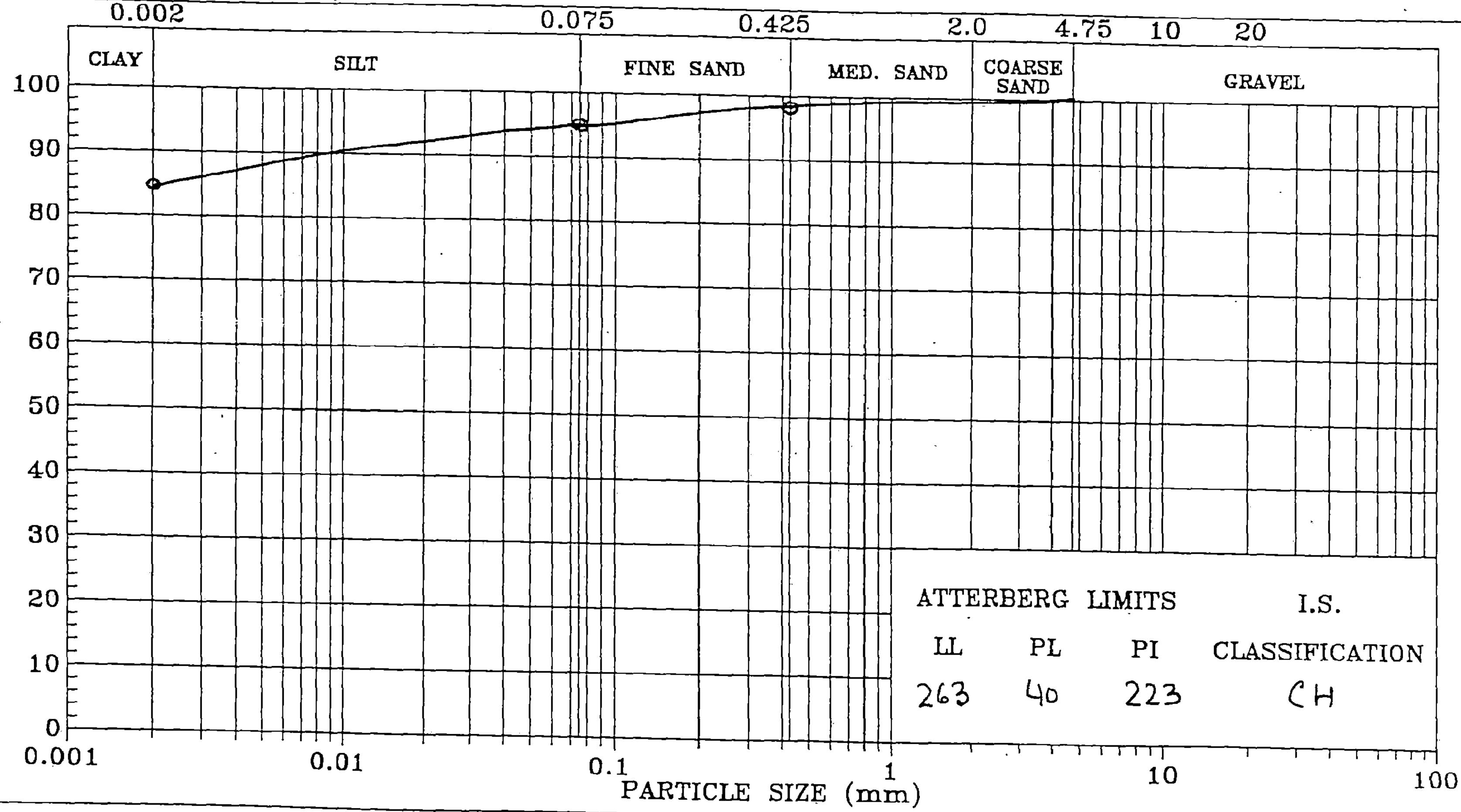
LAB. SAMPLE NO.: 2 LOCATION: PIT NO.: DEPTH (M) :

Fig No 3.2 GRAIN SIZE ANALYSIS



LAB. SAMPLE NO.: 3 LOCATION: PIT NO.: DEPTH (M) :

Fig NO 3.3 GRAIN SIZE ANALYSIS



SAMPLE No. 1
L.L. = 70.00
P.L. = 41.55
P.I. = 28.45

SAMPLE NO. 2
L.L. = 42.7
P.L. = 21.6
P.I. = 21.1

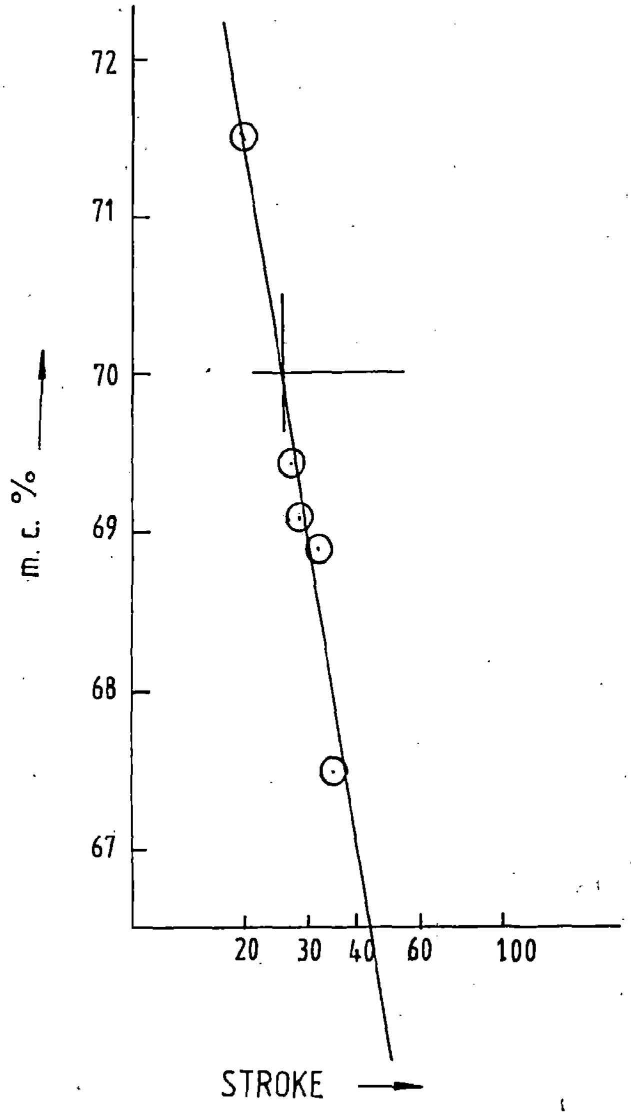


Fig 3.4 Determination of liquid limit for Sample NO 1

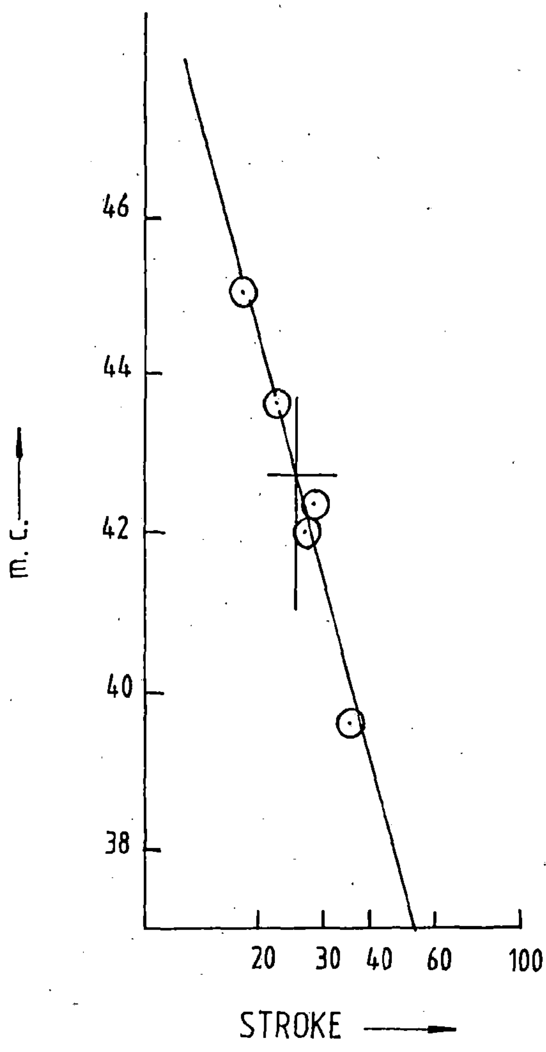


Fig 3.5 Determination of liquid limit for Sample NO 2

SAMPLE No. 3

LIQUID LIMIT = 263.00

PLASTIC LIMIT = 40.00

PLASTIC INDEX = 223.00

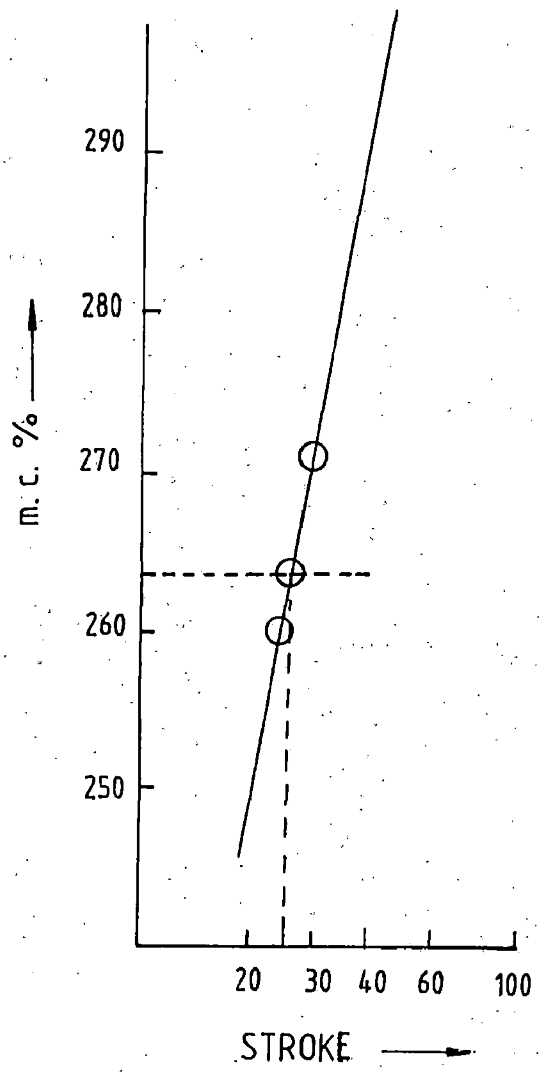


Fig 3.6 Determination of Liquid limit for Sample No 3.

SAMPLE No. **VKJ**/1

G = 2.69

$\gamma d_{max.}$ = 1.45 gm/cc

OMC = 25.6%

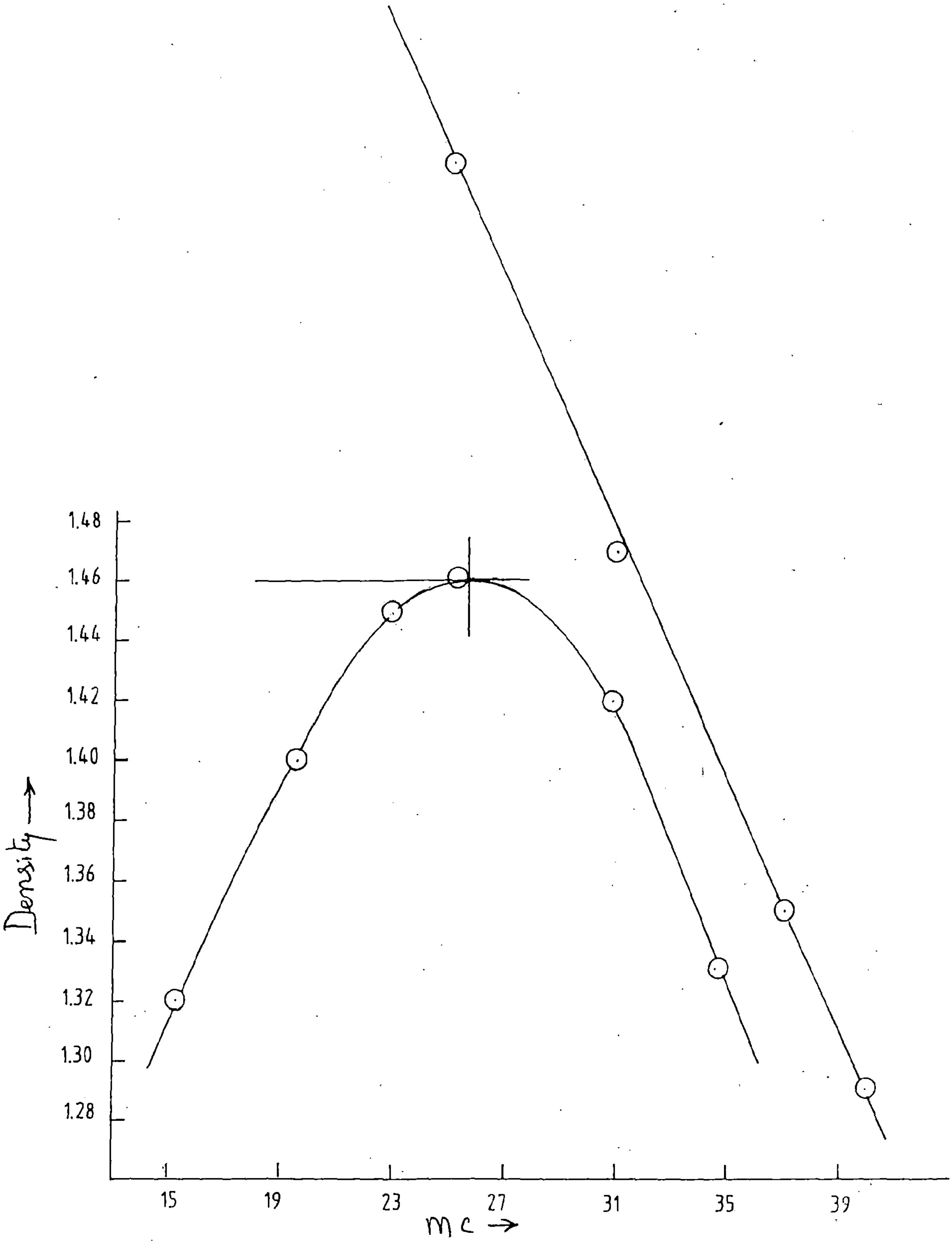


Fig 3.7 Determination of max. dry density & optimum moisture Content for Sample No 1

SAMPLE No. VKJ /2

G = 2.75

$\gamma d_{max.}$ = 1.87 gm/ml

OMC = 14.7 %

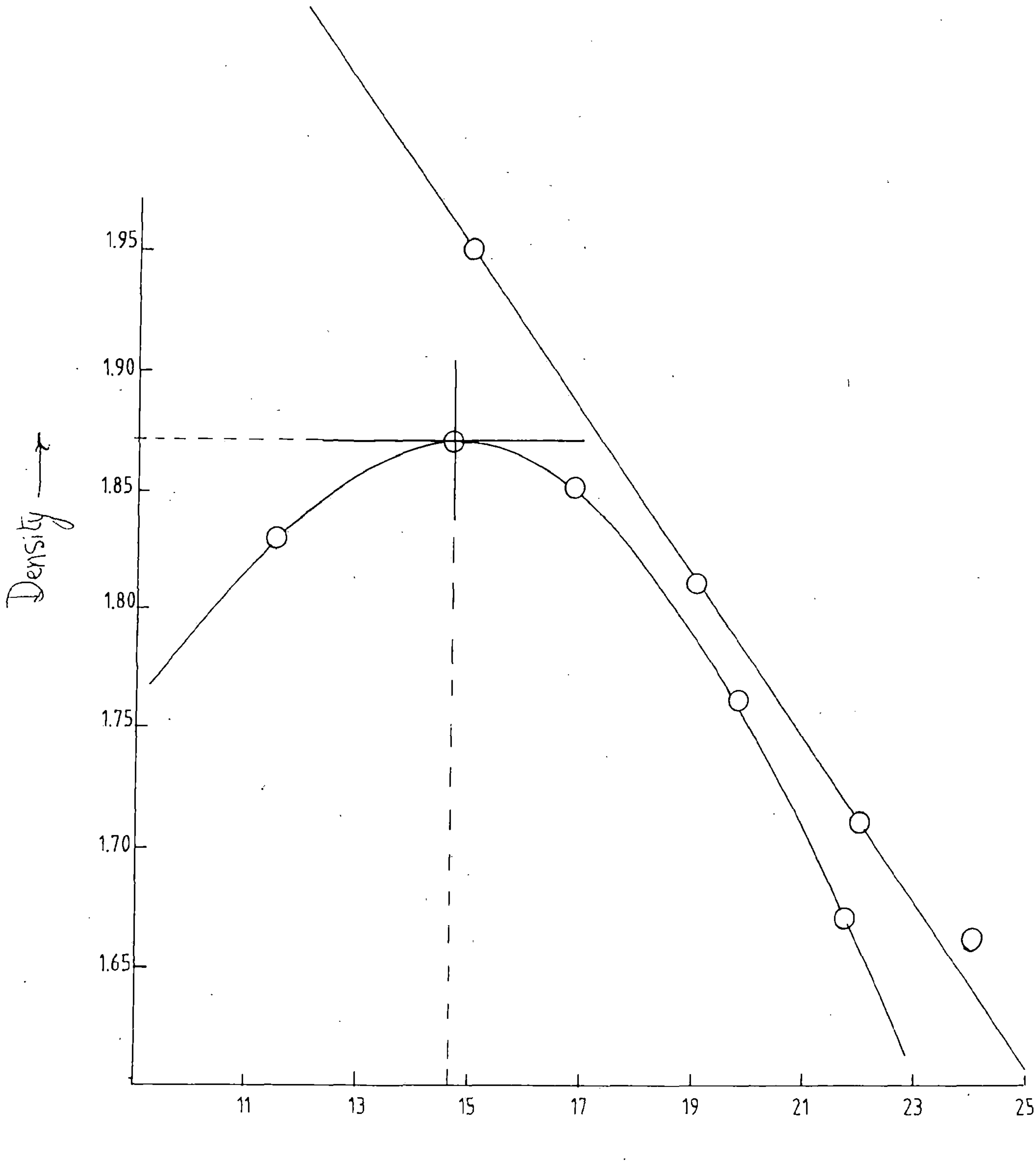


Fig 3.8 Determination of max dry density & optimum moisture content for Sample No 2

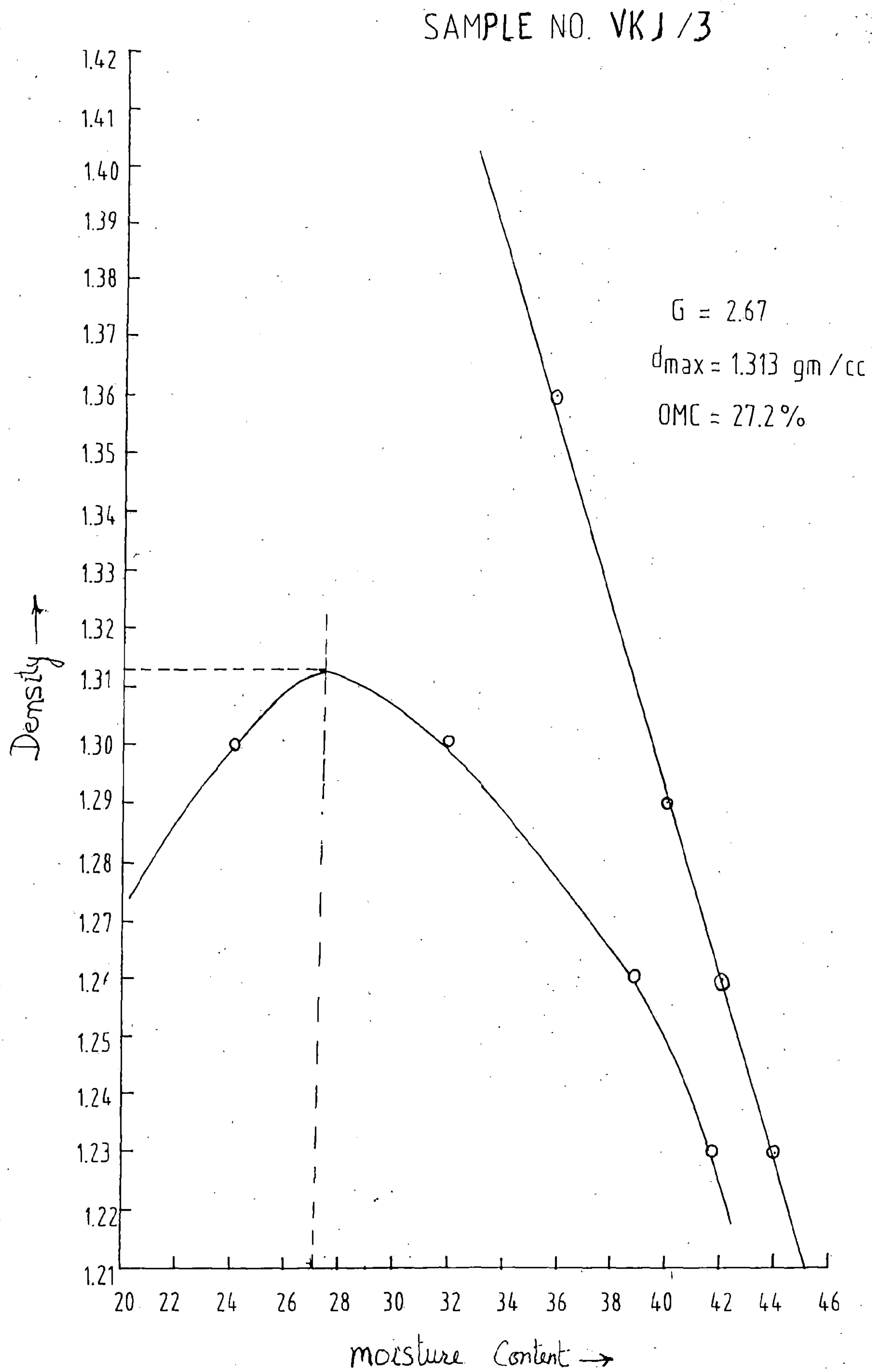


Fig 3.9 Determination of max. dry density & optimum moisture content for Sample No 3

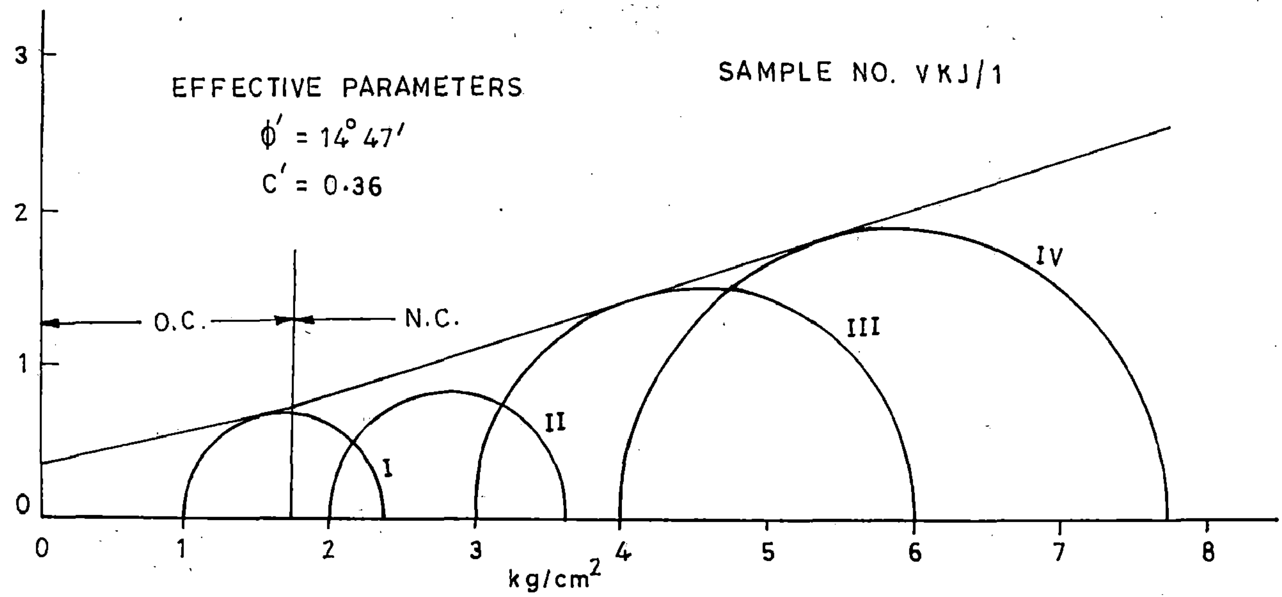


Fig 3.10 Mohr Circle of Triaxial test for Sample no 1

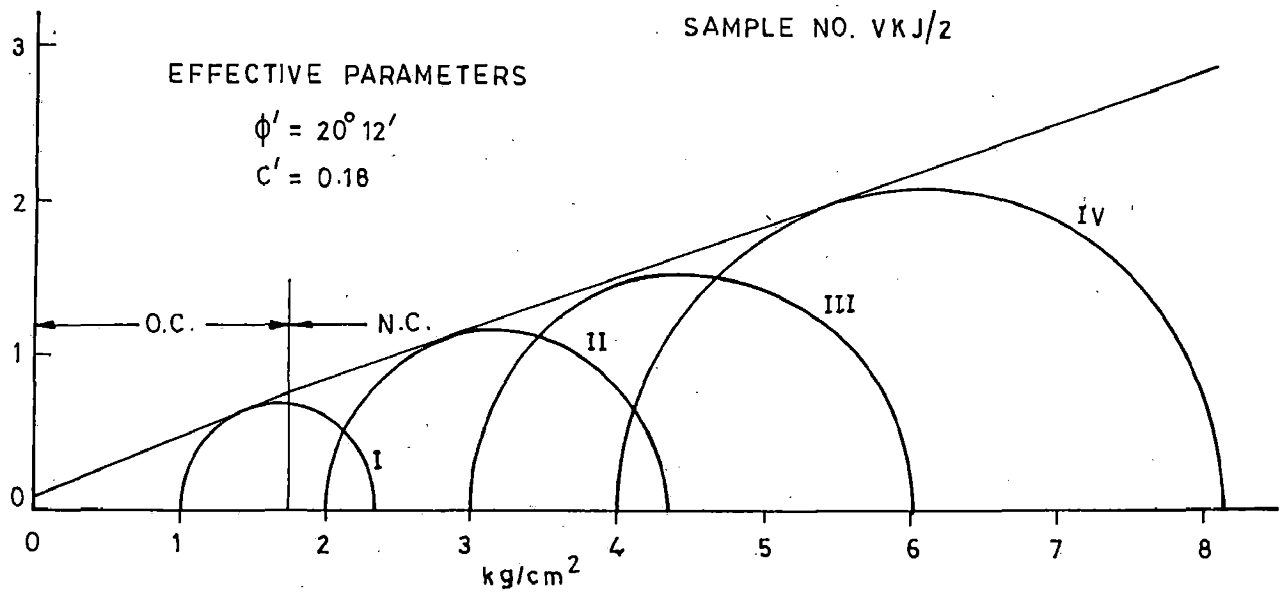


Fig 3.11 Mohr Circle of Triaxial test for Sample no 2

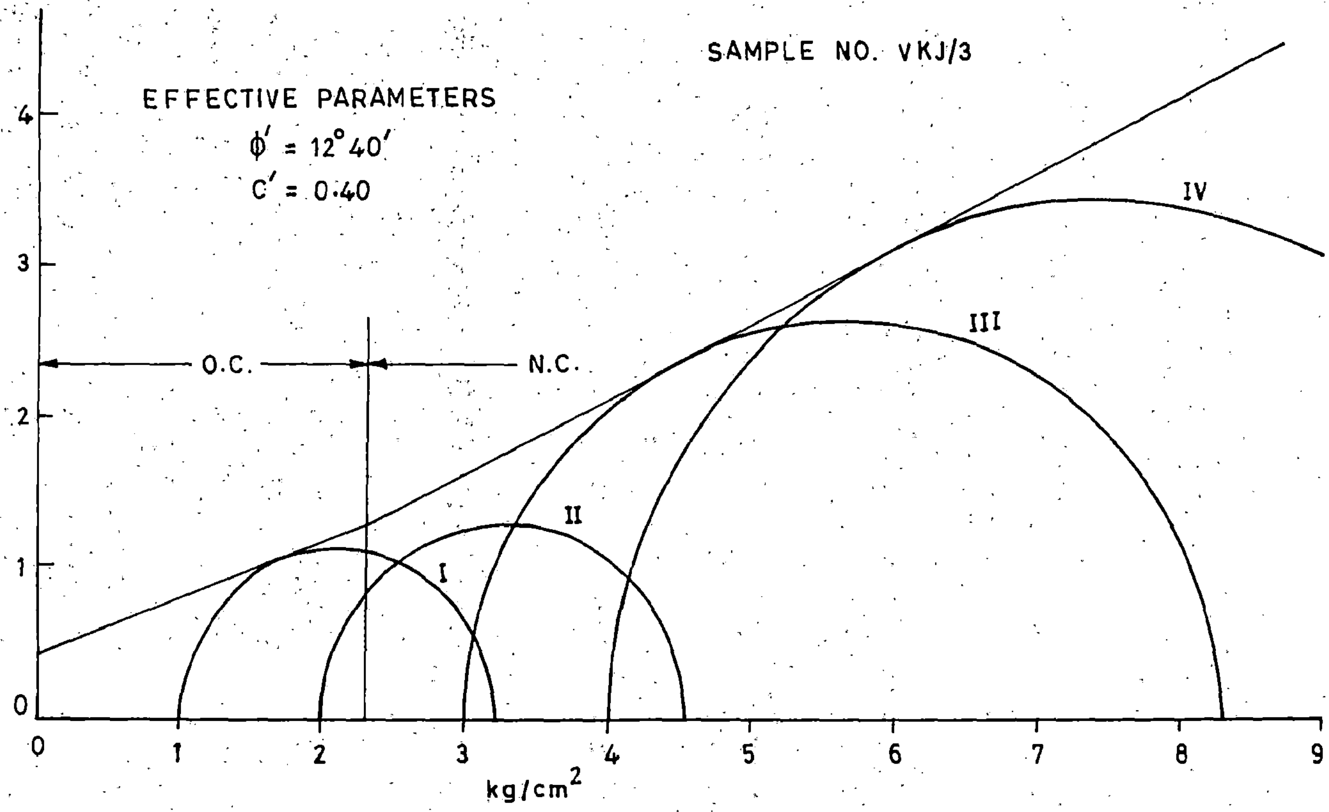


Fig 3.12 Mohr circle of Triaxial test for Sample No 3

3.4.3.2 Consolidation test

The main purpose of consolidation test is to obtain soil data which are used for predicting the rate and amount of settlement of the structure. The two most important soil properties furnished by the consolidation test are the coefficient of compressibility (av) through which magnitude of compression is determined and the coefficient of consolidation (Cv) which gives the rate of compression under load increment.

From the consolidation calculation Table no 3.3 to Table no 3.8. it is seen that the values of coefficient of consolidation (Cv) are in the range of 3.8×10^{-3} to 3.2×10^{-4} cm/sec, 1.1×10^{-2} to1.1×10^{-3} and 1.9×10^{-2} to 6.1×10^{-4} cm^2/sec.

As per Shashi K Gulati (1983) the typical values of Cv as per Table no. 3.9 given below

Table No. 3.9

soil	liquid limit	Cv
Clay	60	0.010
Clay	77	0.006
Clay	115	0.004

The values of coefficient of permeability (k) are on very lower side of the order of 1.4×10^{-6} to 8.8×10^{-9} cm/s which shows that the soil in highly impermeable.

From the stress strain curve for sample no. 1 (Fig. 3.12), it indicates that the soil is over-consolidated up to the stress level of 195 kN/m^2 and after that it is normally consolidated which is indicated by the straight line portion of the curve.

For sample no. 2 and 3 (Fig. 3.13 & 3.14) they are over consolidated upto a stress level of 200 $kN//m^2$.

3.4.3.3 Free swell index and swelling pressure

Free swell index result shown that sample no.1 and 3 are having high degree of expansion wit FSI value of 84% and 155% whereas the sample no.2 with FSI value 20% is having low degree of expansion. Swelling pressure of the order of 187, 170 and 375 kN/m^2 as per Fig. no. 3.15 and 3.16 respectively indicates the highly nature of sample 1 and 3.

Table no 3.3 CONSOLIDATION TEST DATA

Project : vkjain Location : Project No. :
Test No. : 1
Boring No. : - Test Date : 21.09.00 Tested by : cbs
Sample No. : 1 Sample Type : disturbed Depth :
Checked by : klk
Soil Description :
Remarks : clayey sample

	APPLIED PRESSURE (kN/m^2)	FINAL DISPLACEMENT (mm)	VOID RATIO	STRAIN AT END (%)	FITTING TIME (min) SQ.RT.	FITTING TIME (min) LOG	COEFFICIENT OF CONSOLIDATION (m^2/s) SQ.RT.	COEFFICIENT OF CONSOLIDATION (m^2/s) LOG	COEFFICIENT OF CONSOLIDATION (m^2/s) AVE
1)	24.5166	0.32	0.713	1.26	1.4	0.0	3.84E-007	0.00E+000	0.00E+000
2)	49.0333	0.46	0.703	1.83	5.2	0.0	9.88E-008	0.00E+000	0.00E+000
3)	98.0665	0.76	0.683	2.98	5.2	0.0	9.79E-008	0.00E+000	0.00E+000
4)	196.1330	1.32	0.645	5.20	6.4	0.0	7.57E-008	0.00E+000	0.00E+000
5)	392.2660	2.26	0.581	8.88	9.4	0.0	4.87E-008	0.00E+000	0.00E+000
6)	784.5320	3.27	0.511	12.88	13.3	0.0	3.16E-008	0.00E+000	0.00E+000
7)	392.2660	3.08	0.524	12.13	3.2	0.0	1.26E-007	0.00E+000	0.00E+000
8)	196.1330	2.95	0.533	11.61	11.3	0.0	3.65E-008	0.00E+000	0.00E+000
9)	98.0665	2.79	0.544	10.99	0.0	0.0	0.00E+000	0.00E+000	0.00E+000
10)	49.0333	2.64	0.555	10.39	0.0	0.0	0.00E+000	0.00E+000	0.00E+000
11)	24.5166	2.49	0.564	9.82	0.0	0.0	0.00E+000	0.00E+000	0.00E+000

Table no 3.4 CONSOLIDATION CALCULATION SHEET

PROJECT : VK JAIN & Co.

SAMPLE NO: 1.000 DATE : 21.09.00

TESTED BY: CBS

Results of LOADING

LOAD kg/cm^2 (p)	VOID RATIO ei	ef	av= del e/ del p	mv= av/ (1+eo)	E=1/mv	cv cm^2/s	k cm/s	Cc del e/ log(Pf/Pi
0-0.25	0.880	0.713	0.668	0.355	2.81	3.8E-03	1.4E-06	-
0.25-0.5	0.713	0.703	0.040	0.021	47.00	9.9E-04	2.1E-08	0.033
0.5-1	0.703	0.683	0.040	0.021	47.00	9.8E-04	2.1E-08	0.066
1-2	0.683	0.645	0.038	0.020	49.47	7.6E-04	1.5E-08	0.126
2-4	0.645	0.581	0.032	0.017	58.75	4.9E-04	8.3E-09	0.213
4-8	0.581	0.511	0.017	0.009	*******	3.2E-04	2.9E-09	0.233

Results of UNLOADING

LOAD kg/cm^2 (p)	VOID RATIO ei	ef	av= del e/ del p	mv= av/ (1+eo)	E=1/mv	cv cm^2/s	k cm/s	Cs del e/ log(Pf/Pi
8-4	0.511	0.524	0.003	0.002	464.92	1.3E-03	2.7E-09	0.043
4-2	0.524	0.533	0.005	0.003	335.78	3.6E-04	1.1E-09	0.030
2-1	0.533	0.544	0.011	0.007	137.36	0.0E+00	0.0E+00	0.037
1-0.5	0.544	0.555	0.022	0.015	68.68	0.0E+00	0.0E+00	0.037
0.5-0.25	0.555	0.564	0.036	0.024	41.97	0.0E+00	0.0E+00	0.030
0.25-0	0.564	-	-	-	-	-	-	-

Table no 3.5 CONSOLIDATION TEST DATA

Project : vkjain   Location :   Project No. :
Test No. : 1
Boring No. : -   Test Date : 20.09.00   Tested by : cbs
Sample No. : 2   Sample Type : disturbed   Depth :
Checked by : klk
Soil Description : brownishchocolate
Remarks :

	APPLIED PRESSURE (kN/m^2)	FINAL DISPLACEMENT (mm)	VOID RATIO	STRAIN AT END (%)	FITTING TIME (min) SQ.RT.	FITTING TIME (min) LOG	COEFFICIENT OF CONSOLIDATION (m^2/s) SQ.RT.	COEFFICIENT OF CONSOLIDATION (m^2/s) LOG	COEFFICIENT OF CONSOLIDATION (m^2/s) AVE
1)	24.5166	0.26	-0.010	1.04	0.5	0.0	1.15E-006	0.00E+000	0.00E+000
2)	49.0333	0.41	-0.016	1.63	1.7	0.0	3.12E-007	0.00E+000	0.00E+000
3)	98.0665	0.71	-0.028	2.80	3.5	0.0	1.45E-007	0.00E+000	0.00E+000
4)	196.1330	1.18	-0.047	4.66	5.7	0.0	8.59E-008	0.00E+000	0.00E+000
5)	392.2660	1.83	-0.072	7.20	5.3	0.0	8.87E-008	0.00E+000	0.00E+000
6)	784.5320	2.68	-0.106	10.56	4.2	0.0	1.06E-007	0.00E+000	0.00E+000
7)	392.2660	2.51	-0.099	9.88	1.8	0.0	2.43E-007	0.00E+000	0.00E+000
8)	196.1330	2.27	-0.089	8.92	3.1	0.0	1.38E-007	0.00E+000	0.00E+000
9)	98.0665	1.97	-0.078	7.75	6.7	0.0	6.66E-008	0.00E+000	0.00E+000
10)	49.0333	1.65	-0.065	6.48	0.0	0.0	0.00E+000	0.00E+000	0.00E+000
11)	24.5166	1.32	-0.052	5.20	0.0	0.0	0.00E+000	0.00E+000	0.00E+000

Table no 3.6. CONSOLIDATION CALCULATION SHEET

PROJECT : VK JAIN & Co.

SAMPLE NO: 2.000 DATE : 21.09.00

TESTED BY: CBS

Results of LOADING

LOAD kg/cm^2 (p)	VOID RATIO ei	ef	av= del e/ del p	mv= av/ (1+eo)	E=1/mv	cv cm^2/s	k cm/s	Cc del e/ log(Pf/Pi
0-0.25	0.380	0.368	0.048	0.035	28.75	1.1E-02	4.0E-07	-
0.25-0.5	0.368	0.360	0.032	0.023	43.12	3.1E-03	7.2E-08	0.027
0.5-1	0.360	0.344	0.032	0.023	43.12	1.4E-03	3.4E-08	0.053
1-2	0.344	0.318	0.026	0.019	53.08	8.6E-04	1.6E-08	0.086
2-4	0.318	0.283	0.018	0.013	78.86	8.9E-04	1.1E-08	0.116
4-8	0.283	0.237	0.011	0.008	*******	1.1E-03	8.8E-09	0.153

Results of UNLOADING

LOAD kg/cm^2 (p)	VOID RATIO ei	ef	av= del e/ del p	mv= av/ (1+eo)	E=1/mv	cv cm^2/s	k cm/s	Cs del e/ log(Pf/Pi
8-4	0.237	0.246	0.002	0.002	549.78	2.4E-03	4.4E-09	0.030
4-2	0.246	0.259	0.007	0.005	190.31	1.4E-03	7.3E-09	0.043
2-1	0.259	0.275	0.016	0.013	77.31	6.7E-04	8.6E-09	0.053
1-0.5	0.275	0.293	0.036	0.029	34.36	0.0E+00	0.0E+00	0.060
0.5-0.25	0.293	0.311	0.072	0.058	17.18	0.0E+00	0.0E+00	0.060
0.25-0	0.311	-	-	-	-	-	-	-

Table no 3.7 CONSOLIDATION TEST DATA

Project : VK. JAIN
Test No. : 3
Boring No. : -
Sample No. : VKJ/3
Checked by : VKJ
Soil Description : YELLOWISH
Remarks : BENTONITE

Location :
Test Date : 29.11.00
Sample Type : DISTURBED

Project No. :
Tested by : CBS
Depth :

	APPLIED PRESSURE (kN/m^2)	FINAL DISPLACEMENT (mm)	VOID RATIO	STRAIN AT END (%)	FITTING TIME (min) SQ.RT.	FITTING TIME (min) LOG	COEFFICIENT OF CONSOLIDATION (m^2/s) SQ.RT.	COEFFICIENT OF CONSOLIDATION (m^2/s) LOG	COEFFICIENT OF CONSOLIDATION (m^2/s) AVE
1)	24.5166	0.14	-0.005	0.54	0.3	0.0	1.87E-006	0.00E+000	0.00E+000
2)	49.0333	0.20	-0.008	0.79	0.6	0.0	8.36E-007	0.00E+000	0.00E+000
3)	98.0665	0.36	-0.014	1.40	1.2	0.0	4.47E-007	0.00E+000	0.00E+000
4)	196.1330	0.72	-0.028	2.83	2.9	0.0	1.75E-007	0.00E+000	0.00E+000
5)	392.2660	1.73	-0.068	6.80	4.1	0.0	1.16E-007	0.00E+000	0.00E+000
6)	784.5320	3.08	-0.121	12.13	7.1	0.0	6.11E-008	0.00E+000	0.00E+000
7)	392.2660	2.88	-0.113	11.34	5.8	0.0	7.14E-008	0.00E+000	0.00E+000
8)	196.1330	2.64	-0.104	10.41	8.7	0.0	4.86E-008	0.00E+000	0.00E+000
9)	98.0665	2.42	-0.095	9.52	0.0	0.0	0.00E+000	0.00E+000	0.00E+000
10)	49.0333	2.20	-0.087	8.67	0.0	0.0	0.00E+000	0.00E+000	0.00E+000
11)	24.5166	2.00	-0.079	7.87	0.0	0.0	0.00E+000	0.00E+000	0.00E+000

Table no 3 . 8 Consolidation Calculation Sheet

PROJECT : VKJ.PROJECT

SAMPLE NO: VKJ\3 DATE : 29.11.2000

TESTED BY: VKJ

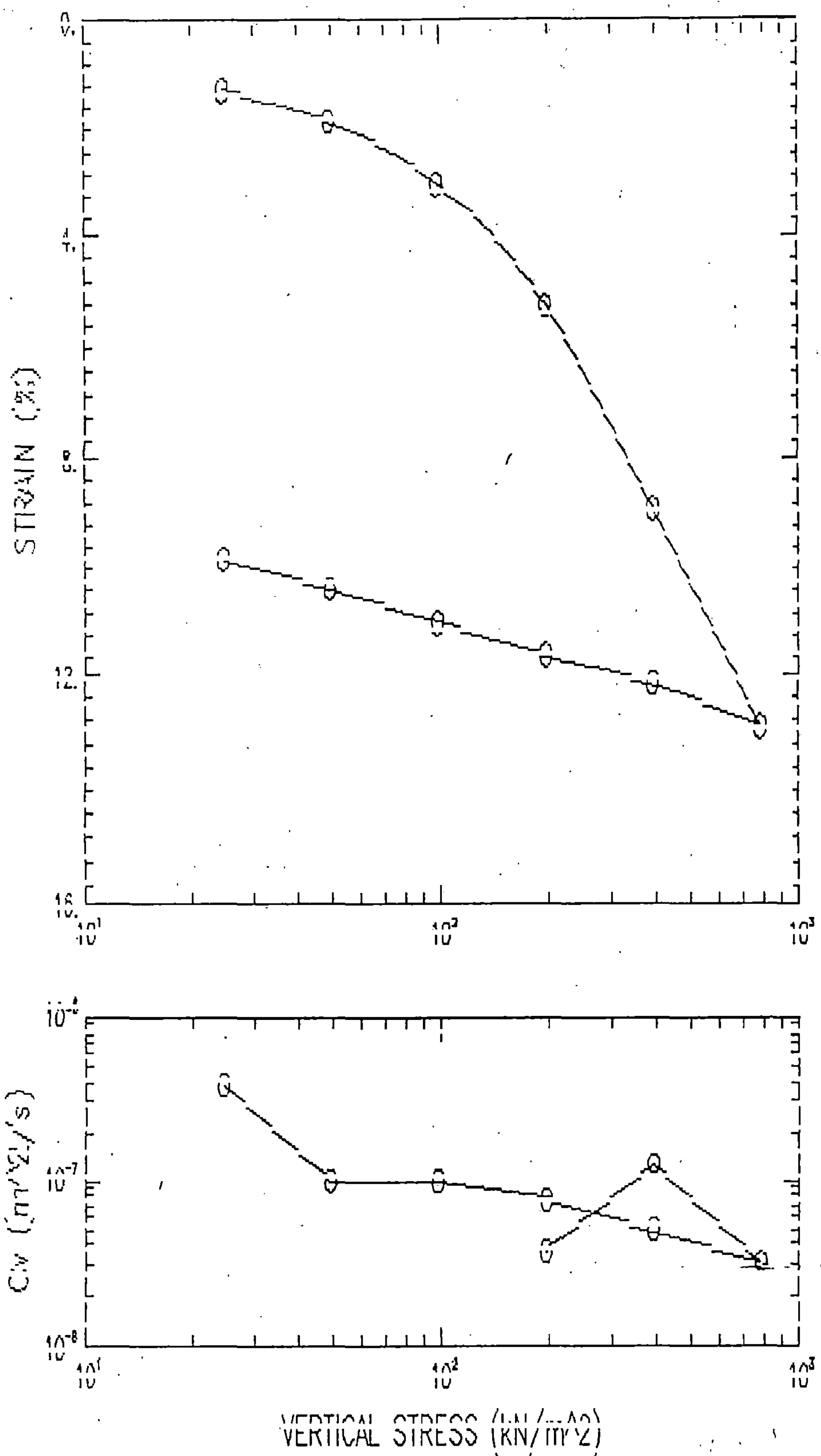
Results of LOADING

LOAD kg/cm^2 (p)	VOID RATIO ei	ef	av= del e/ del p	mv= av/ (1+eo)	E=1/mv	cv cm^2/s	k cm/s	Cc del e/ log(Pf/Pi
0-0.25	1.060	1.049	0.044	0.021	46.82	1.9E-02	4.0E-07	-
0.25-0.5	1.049	1.044	0.020	0.010	103.00	8.4E-03	8.1E-08	0.017
0.5-1	1.044	1.032	0.024	0.012	85.83	4.5E-03	5.2E-08	0.040
1-2	1.032	1.002	0.030	0.015	68.67	1.8E-03	2.5E-08	0.100
2-4	1.002	0.920	0.041	0.020	50.24	1.2E-03	2.3E-08	0.272
4-8	0.920	0.811	0.027	0.013	75.596	6.1E-04	8.1E-09	0.362

Results of UNLOADING

LOAD kg/cm^2 (p)	VOID RATIO ei	ef	av= del e/ del p	mv= av/ (1+eo)	E=1/mv	cv cm^2/s	k cm/s	Cs del e/ log(Pf/Pi
8-4	0.811	0.827	0.004	0.002	452.75	7.2E-04	1.6E-09	0.053
4-2	0.827	0.846	0.010	0.005	190.63	4.9E-04	2.5E-09	0.063
2-1	0.846	0.864	0.018	0.010	100.61	0.0E+00	0.0E+00	0.060
1-0.5	0.864	0.882	0.036	0.020	50.31	0.0E+00	0.0E+00	0.060
0.5-0.25	0.882	0.898	0.064	0.035	28.30	0.0E+00	0.0E+00	0.053
0.25-0	0.748	-	-	-	-	-		-

CONSOLIDATION TEST
SUMMARY REPORT



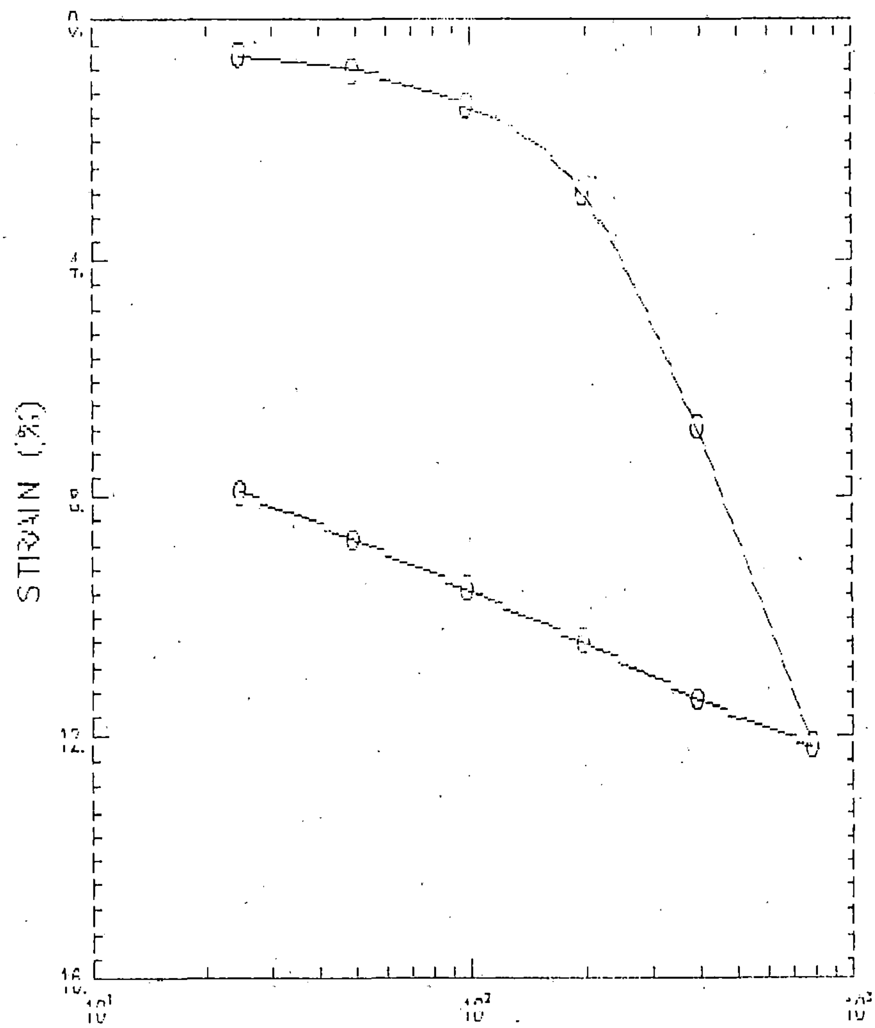
CSMRS
Project Name : vkjain

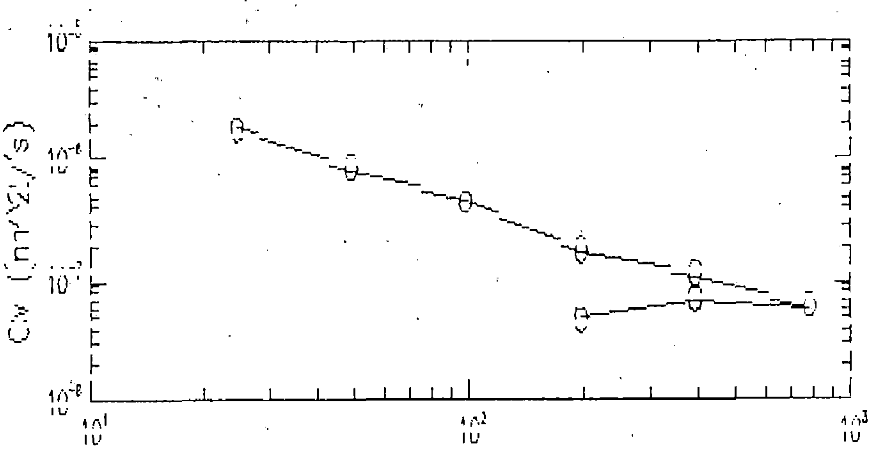
Project No :	Boring No : –	Sample No : 1
Test Date : 21.09.00	Test No : 1	Depth :

Description :

Fig 3.13 Consolidation Test Result of Sample No 1

CONSOLIDATION TEST
SUMMARY REPORT





VERTICAL STRESS (kN/m^2)

CSMRS

Project Name : VK. JAIN

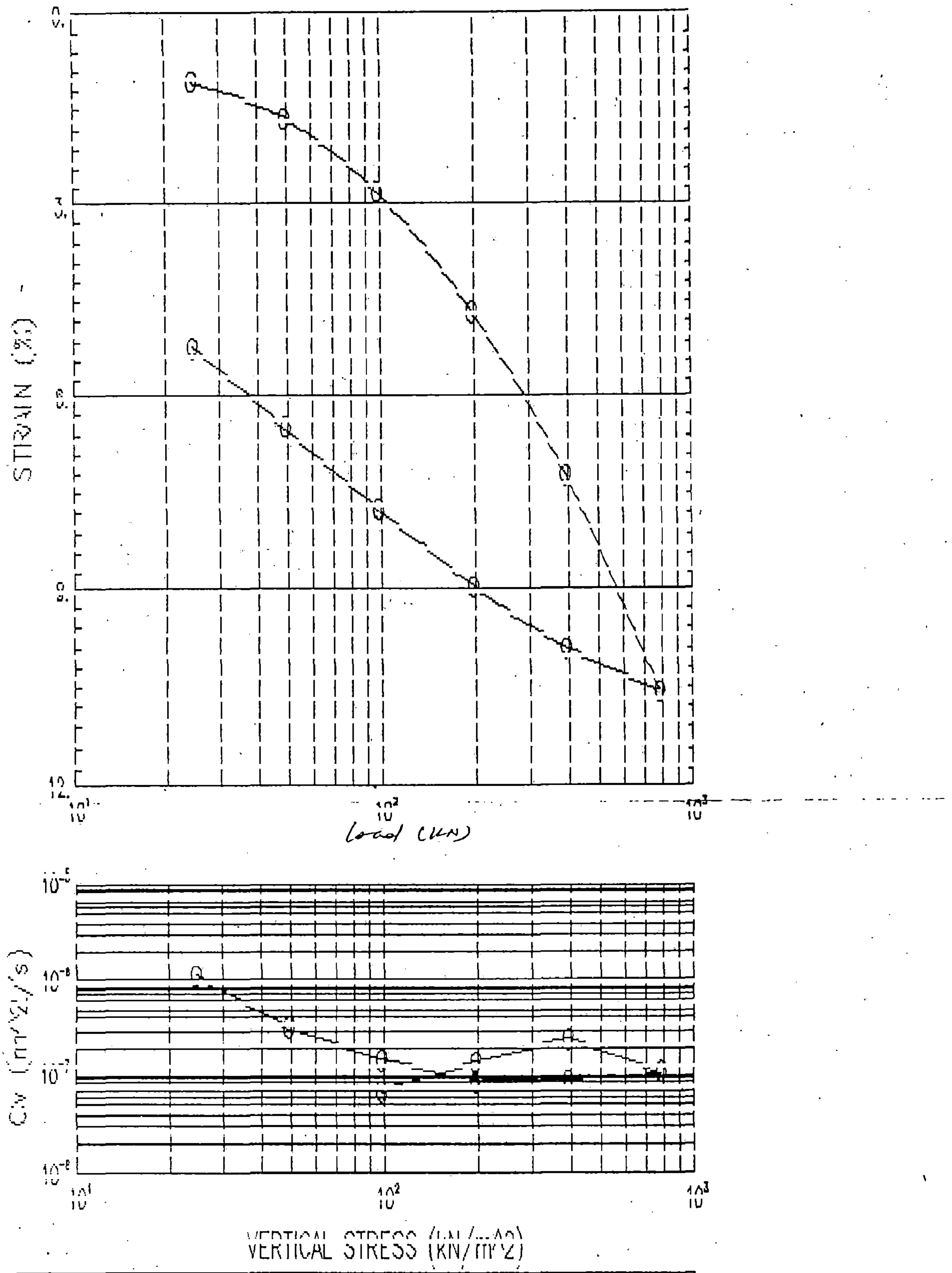
Project No : Boring No : – Sample No : VKJ/3

Test Date : 29.11.00 Test No : 3 Depth :

Description : YELLOWISH

Fig 3.15 Consolidation test results of Sample No 3

SUMMARY REPORT



CSMRS

Project Name : vkjain

Project No :	Boring No : –	Sample No : 2
Test Date : 20.09.00	Test No : 1	Depth :

Description : brownishchocolate

Fig 3.14 Consolidation test result of Sample No 2

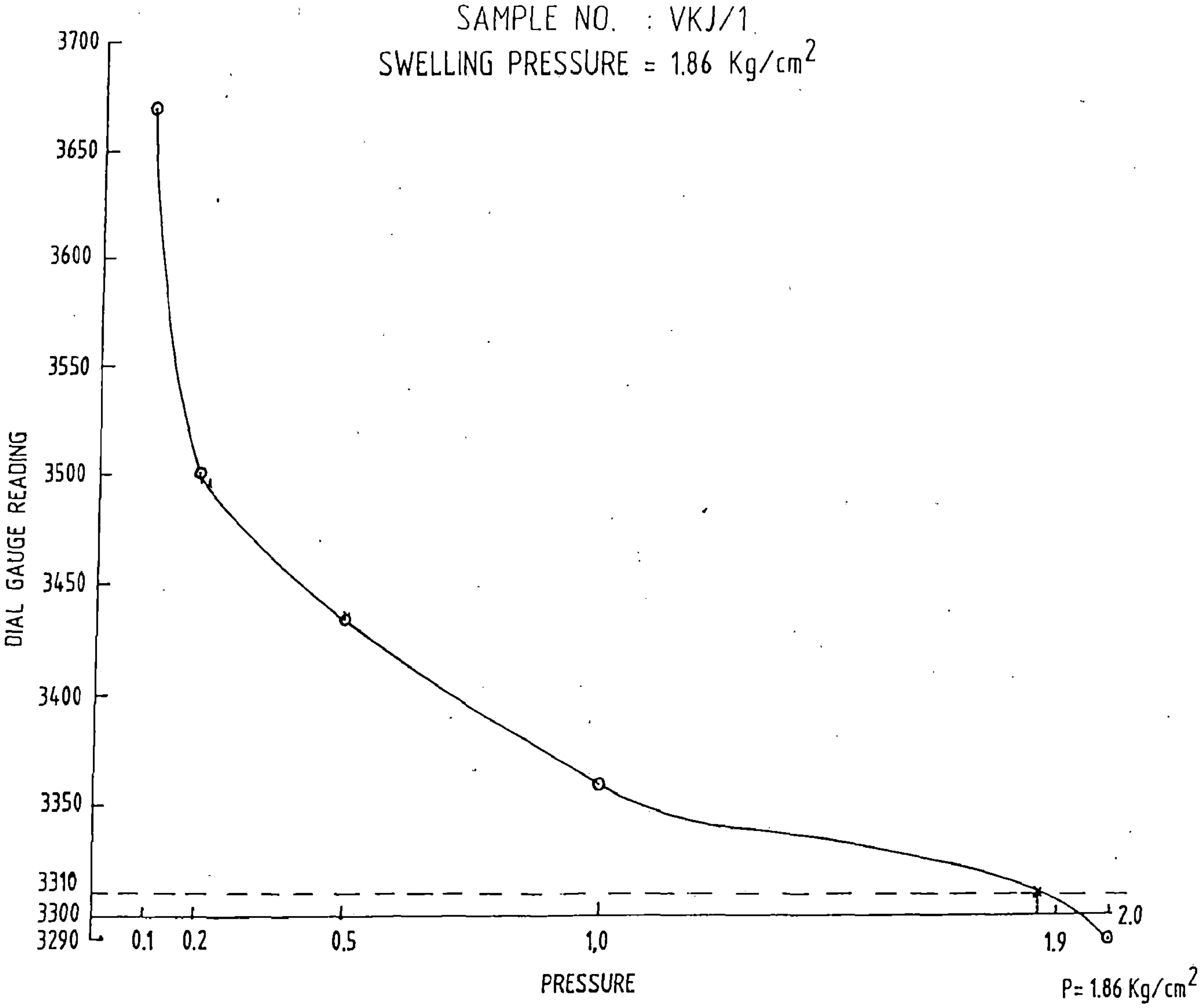


Fig 3.16 Swelling Pressure Curve for Sample No 1

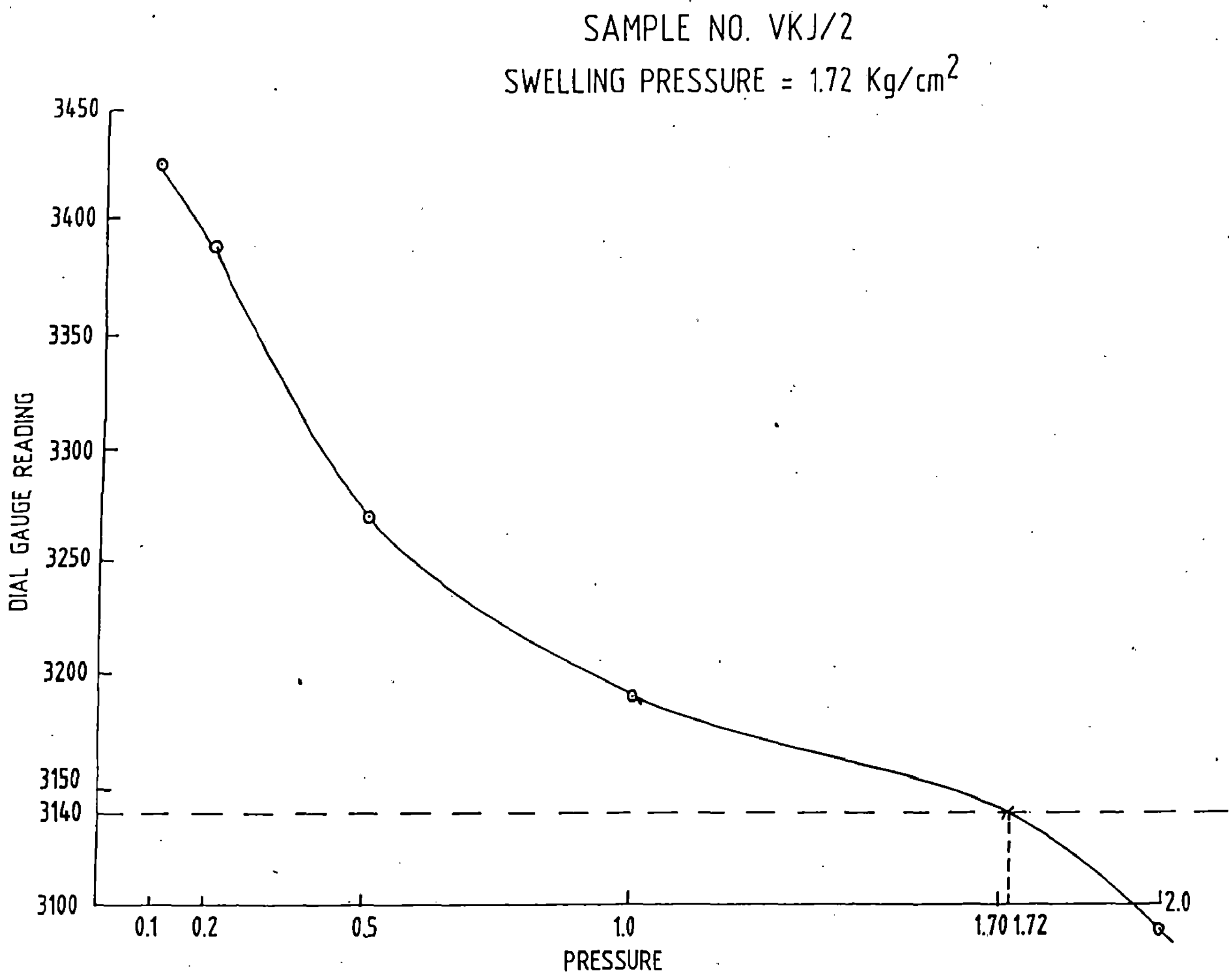


Fig 3.17 Swelling Pressure Curve for Sample No 2

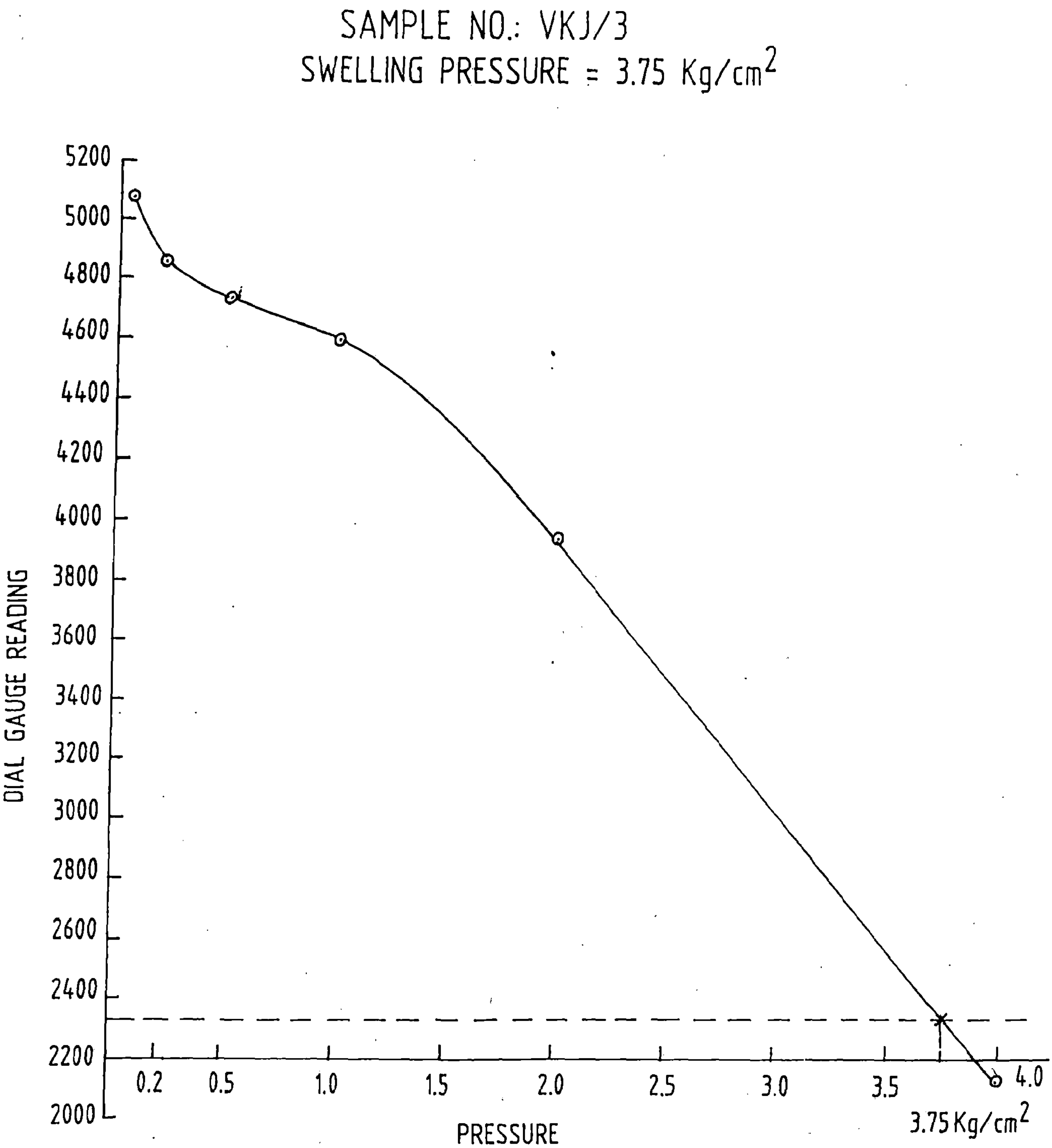


Fig 3.18 Swelling Pressure Curve for Sample No3

Test result of all the three samples have been tabulated in Table no. 3.10.

Table No. 3.10 Test results of all the three samples

Index Properties and Engineering Properties				
S No.	Name of test	Sample No. 1 Black cotton soil	Sample No. 2 Red soil	Sample No. 3 Bentonite
1.	Pipeet analysis Particle less than 0.075 mm	90.30%	81.07%	85.34%
2.	Specific gravity	2.67	2.75	2.67
3.	Liquid limit	70%	42.70%	263%
4.	Plastic limit	41.55%	21.55%	40%
5.	Plasticity index	28.45%	21.15%	223%
6.	Shrinkage limit	19.29%	11.87%	6%
7.	Max. Dry density gm/cc	1.47	1.87	1.313
8.	Optimum moisture content	25.6%	14.7%	27.20%
9.	Triaxial test (1) Angle of internal fraction (2) Cohesion kN/m^2	 17^0 36	 11^0 18	 12.4^0 40
10.	Free swell index	84%	19.56%	155%
11.	Swelling pressure kN/m^2	187	170	375

TABLE NO. 3.11 QUANTITY OF SOIL SAMPLES REQUIRED FOR CONDUCTING THE LABORATORY TEST

Sl.No.	Name of Test	Temperature & Duration of drying	Amount of soil sample Required	Passing IS Sieve Size
1.	Water content	Oven dry 24 hours	25 gms	425 micron
2.	Specific Gravity	105 – 110 C 24 hours	50 gms	2 mm
3.	Grain size Analysis	Air drying	500 gms.	4.75 mm
4.	Liquid Limit	Air drying	300 gms.	425 micron
5.	Plastic limit	- do -	50 gms.	- do -
6.	Shrinkage factor	- do -	100 gms.	- do -
7.	Compaction	- do -	3 kg.	2 mm
8.	Free Swell Index	Oven dry	20 gms.	425 micron
9.	Swelling Pressure	Air dry	2 kg.	2 mm
10.	Consolidation Properties	Air drying 105-110 C	500 gms.	--------
11.	Triaxial Test	110 C + 5	1 kg.	--------

REMEDIAL MEASURES

4.1 GENERAL

In theory, the swelling potential of an expansive clay can be minimised or completely eliminated by one of the following methods :

1. Flood the in-place soil to achieve swelling prior to construction ;
2. Decrease the density of the soil by compaction control ;
3. Replace the swelling soils with non swelling soils ;
4. Change the properties of expansive soils by chemical injection, or
5. Isolate the soil so there will be no moisture change

4.2 PREWETTING

An old established concept among engineers and contractors as well as laymen in dealing with swelling soils is prewetting. Moisture can migrate from a moderate-depth water table to an upper moisture-deficient soil by means of capillary rise. Moisture migration can also take place from a high-temperature area to a low-temperature area by means of thermo-osmosis or other mechanisms. Normally this moisture evaporates at the surface and moisture equilibrium is maintained in the soil. The presence of covered areas, such as floor slabs, pavements, or similar structures which inhibit this evaporation increases the moisture content of the foundation soil with resultant swell.

The prewetting theory is based on the assumption that if soil is allowed to swell by wetting prior to construction and if the high soil moisture content is maintained, the soil volume will remain essentially constant, achieving a no-heave state and therefore structural damage will not occur.

4.2.1 Ponding

The present prewetting practice usually involves direct flooding or ponding of the building area. The foundation and floor area is flooded by constructing a small earth berm around the outside of the foundation trenches to impound the water floods the foundation and floor area. Another practice includes first prewetting the foundations trenches, then placing the foundation which is used as a dike to flood the floor area. In some cases, where the moisture content at footing depth is stable, it is possible to place concrete footings and utilise them as dikes so that only the floor area is prewetted.

The effect of ponding or flooding on the moisture content at various depths has been investigated by the Texas Highway Department. The subgrade was ponded and the moisture content at various depths was taken. The following observations were made:

1. The moisture content achieved a significant penetration of only 4 feet below the pond during a period of 24 days.
2. To obtain desirable moisture distribution at greater depths, ponding should extend approximately 30 days.

For slab-on-ground construction, after completing of the prewetting treatment, the ground surface must be kept moist until the slab is placed. A gravel or sand bed 10-15 cm. thick should be placed over the subgrade prior to the prewetting period. The gravel layer prevents the clay from drying and shrinking.

The prewetting operation must not be at the discretion of a contractor or owner. The treatment should be based upon an engineering investigation and evaluation of the site, subsoil condition, swelling potential, climatic conditions foundation system, and prior local experience. The moisture content profile should be checked frequently by tests in the field to assure that the desired results are achieved.

4.2.2 Evaluation

Most engineers strongly endorse the use of prewetting to minimise subgrade heaving. In view of the past experience and actual case studies, it is doubtful if prewetting can be successfully used with llightly loaded structures. The effective migration of moisture, depth of penetration, time required for saturation, and swelling of partially saturated soils is not fully understood. Prewetting practice is much more complicated than assumed by most laymen. A great amount of research will be required before complete evaluation of the prewetting practice can be made. Some of the disadvantages of the prewetting method are as follows :

1. Allowing the ponding water to migrate into the lower moisture-deficient soils. Experience indicates that in a covered area, the moisture content of the underslab soil seldom decreases. Wet soil will induce swelling. After the swelling has reached its maximum potential, moisture migrates to the lower moisture-deficient soil and induced further swelling. The procedure can continue for as long as 10 years.
2. From a construction standpoint, the time required for prewetting can be critical. A moisture condition of less than saturation is often adequate to inhibit objectionable uplift. The length of ponding time required is usually about 1 to 2 months. Even this length of time may be objectionable as being too great.
3. It is highly questionable if a uniform moisture content can be obtained in prewetted areas. Water can only seep into the stiff clay through firrures, and consequently, uniform distribution of moisture content is not likely to take place. As a result, differential heaving can be critical even after a prolonged period of prewetting.
4. Experiments indicate that ponding water can effectively penetrate the soil to a depth of 1.2 meter within a reasonable time. Such depth is insufficient to provide a balanced moisture zone for the construction of important structures.
5. While prewetting may prove to be a possible method of stabilising the soil beneath the floor slab, pavement, or canal lining, it is doubtful that footing foundations can be placed on prewetted soil. In saturated conditions, the bearing

capacity of a stiff clay can be reduced to a very low value, less than 48 kN/m^2 which prohibits the use of conventional footing foundation.

While prewetting may play an important role in the construction of slabs, it is doubtful that this method will be an important construction technique for building foundations on expansive soils.

4.3 COMPACTION CONTROL

The amount of swelling that occurs when a structural fill is exposed to additional moisture depends upon the following :

The compacted dry density ;

The moisture content ;

The method of compaction ; and

The surcharge load.

The last two requirements are not critical in actual construction. The method of compaction is generally limited by available equipment. For lightly loaded structures, the surcharge load is usually very small.

4.3.1 Placement Condition

As early as 1959, Dawson suggested that highly expansive soils be compacted to some minimum density rather than to a maximum density.

Holtz and Gibbs show the influence of density and moisture on the expansion of a compacted expansive clay, as shown on figure.

It can be seen that expansive clays expand very little when compacted at low densities and high moisture but expand greatly when compacted at high densities and low moistures.

Gizienski and Lee show that when their test soil was compacted at about 4-1/2 percent above optimum, which is 10-1/2 percent, the swell was negligible for any degree of compaction.

The main reason moisture content is important is that moisture content can generally result in low density fill; not that high moisture content will reduce swelling. The controlling element is density. Compacting stiff clay at 4 to 5 percent above optimum is very difficult. The process of recompacting swelling clays at moisture contents slsightly above their natural moisture content and at a low density should be an excellent approach.

It has been established that the swelling pressure of lay is independent of the surcharge pressure, initial moisture content, degree of saturation, thickness of stratum, and increases only with the increase of initial dry density. For instance, with reference to figure 4.1, by decreasing the dry density of a typical expansive clay from 109-100 pcf, the swelling pressure decreases from 13000 –5000 psf, and the swelling potential decreases from 6.7 to 4.2 percent. All of this can be accomplished without changing the moisture content.

The main advantage of using this approach is that the swelling potential can be reduced without the adverse effects caused by introducing excessive moisture into the soil. This indicates indicates that to decrease the swelling potential from 6.7 to 4.2 percent, an increase of moisture content of about 5 percent will be required.

The shortcomings of prewetting methods mentioned in the preceding section can be eliminated by compaction control. Excess water will not be present in the soil ; therefore, there will not be migration of moisture to the underlying moisture-deficient soils and long waiting periods, prior to construction, will be unnecessary. A reasonably good bearing capacity can be assigned to the low density soil.

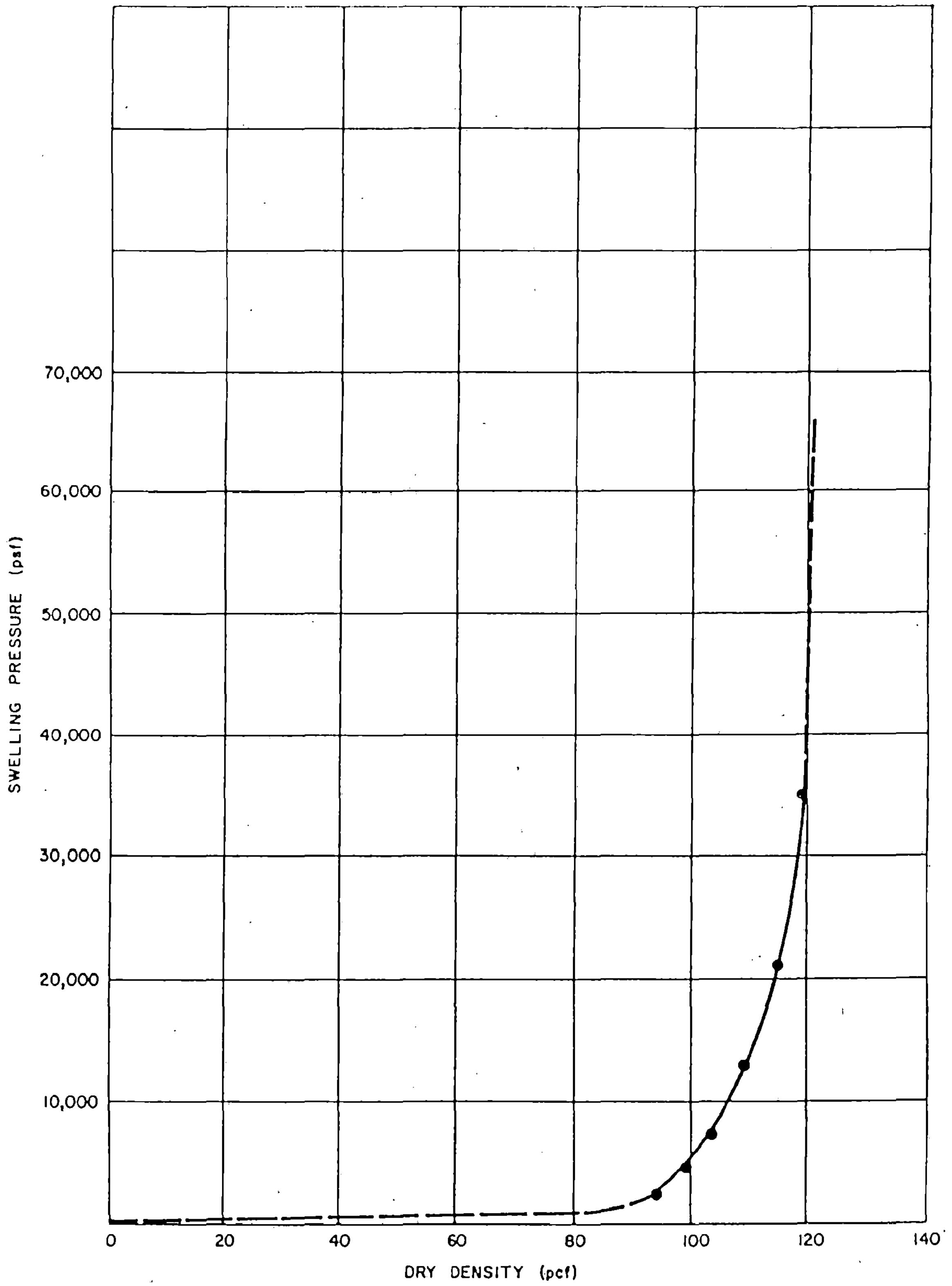


Figure 4.) Effect of varying density on swelling pressure for constant moisture content samples.

With modern construction techniques, it is possible to scarify, pulverise, and recompact the natural soil effectively without substantially increasing the construction costs.

4.4 SOIL REPLACEMENT

A simple and easy solution for slabs and footings founded on expansive soils is to replace the foundation soil with nonswelling soils. Experience indicates that if the subsoil consists of more than about 1.5 meter of granular soils (SC-SP), underlain by highly expansive soils, there is no danger of foundation movement when the structure is placed on the granular soils. The mechanics and the path of surface water seeping through the upper granular soils and into the expansive soils is not clear. It is concluded that either seepage water has never reached the expansive soils, or the heaving of the lower expansive soils is so uniform that structural movement is not noticeable.

This is not true in the case of man-made fill. For economic reasons, the extent of the selected fill must be limited to a maximum of 3 meter beyond the building line. Therefore, the possibility of edge wetting exists. A guideline has not been established as to the thickness requirement for the selected fill. A minimum of 1 meter should always be insisted upon, although 1.5 meter is preferred. This thickness refers to thickness of selected fill beneath the bottom of the footings or bottom of floor slabs.

The pertinent requirements concerning soil replacement are the type of replacement material, the depth of replacement, and the extent of replacement.

4.4.1 Type of Material

Obviously, the first requirement for the replacement soil is that it be non expansive. All granular soils ranging from GW to SC in the Unified Soil Classification System may fulfill the nonexpansive soil requirement. However, for clean, granular soils such as GW and SP, surface water can travel freely through the soil and cause wetting of the lower swelling soils. In the other extreme, SC material with a high percentage of plastic clay sometimes will exhibit swelling potential. The following criteria have been used with a certain degree of success :

Liquid limit,percent	Percent minus No. 200 sieve
Greater than	15 – 30
30 – 50	10 – 40
Less than 30	5 – 50

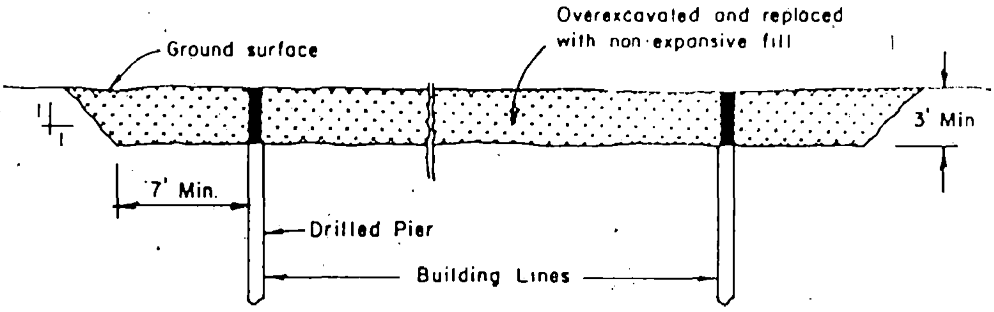
It is becoming increasingly difficult to locate materials, fulfilling the above requirements, If necessary, the requirement for imperviousness can be forfeited. Any selected fill will be satisfactory provided the material is nonexpansive. Also, swell tests are the only positive method of determining the expansiveness of the material. When in doubt, such tests should be conducted rather than relying on plasticity tests.

A great deal of emphasis has been given to the possibility of blending granular soil with the on-site swelling soils, thus reducing the amount of imported fill required. Theoretically, such a method is reasonable ; but in practice it is difficult to incorporate granular soil with stiff, dry expansive clays. Disc harrows and plows will be required to break the clay into reasonably sized clods. Such an undertaking will probably be as expensive as using the lime stabilization method.

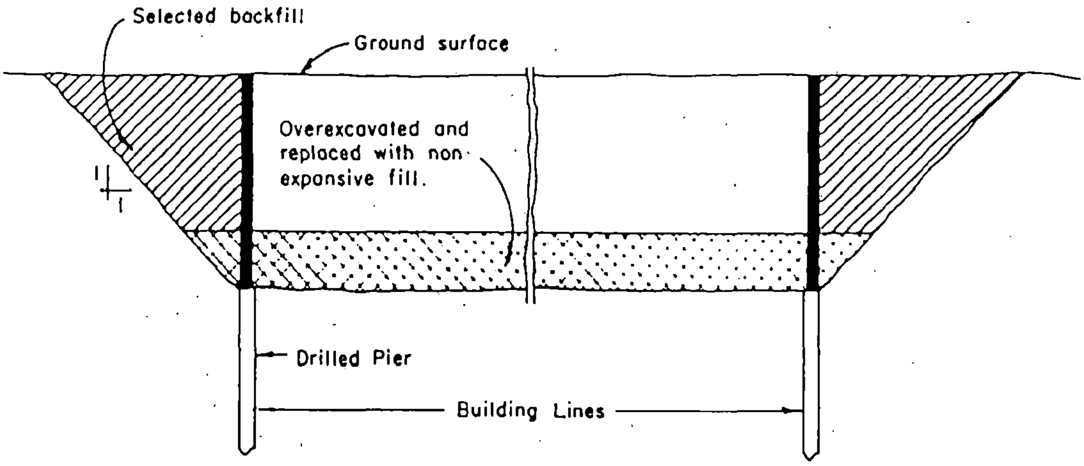
4.4.2 Extent of Replacement

The main reason that an artificially selected fill cushion is less effective than a natural granular soil blanket is that in natural conditions, the blanket extends over a large area, much larger than in the artificial condition. In an artificial fill situation, it is always possible for surface water to seep into the deep-seated expansive soil at the perimeter of the fill. Therefore, the larger the area of replacement, the more effective the fill.

Figure 4.2 shows the suggested extent of replacement for both basement and non basement conditions. With this arrangement, the possibility of surface water entering the foundation soil is greatly reduced. The type of material used for backfill should be the same as used for the under slab selected fill.



NON-BASEMENT CONDITION



DEEP BASEMENT CONDITION

Figure 4.2 Suggested extent of fill replacement.

4.4.3 Evaluation

With present technology on expansive soils, soil replacement is the best method to use in obtaining a stabilized foundation soil. The following are the evaluations of soil replacement method

1. It is possible to compact the replaced non expansive soil to a high degree of compaction, thus enabling the material to support either heavily loaded slabs or footings. Such capability cannot be obtained by the prewetting method. Also,

with the compaction control method, a high degree of compaction on expansive soils is not desirable, and, consequently, the load carrying capacity is limited .

2. The cost of soil replacement is relatively inexpensive when compared to chemically treating the soil. No special construction equipment, such as disc harrow, spreader, or mixer will be required. The construction can be carried out without delay as is encountered in the prewetting method.
3. The granular soil cushion also serves as an effective barrier against the rise of ground water or perched water.
4. With the exception of a structural floor slab (suspended floor), soil replacement provides the safest approach to slab-on-ground construction.
5. To guard against unexpected conditions which might cause heaving, it is strongly suggested that floating slab construction be used. Slip joints must be provided for all slab bearing partition walls so there is no chance of slab movement disturbing the structure.
6. Surface drainage around the building must be properly maintained so there is no opportunity for water to enter the expansive soils beneath the selected fill.

4.5 STABILISATION OF SWELLING SOIL WITH COMBINATION OF LIME AND FLYASH

4.5.1 General

Over the last few years, the use of lime and fly ash has increased as stabilising materials for naturally occurring fine-grained soils. Fly ash is a waste product generated by coal fired thermal power station and poses serious environmental problem if not disposed off properly. The use of fly ash serves two purposes, firstly the disposal of a waste material and secondly, the use as a construction, material. Normally, on the addition of water, a mixture of lime and fly ash reacts with soil to form a new material with strength and durability properties superior to those exhibited by either of the materials reacting alone. For comparing the strength and durability of the various soil-stabilising mixtures, unconfined compressive strength tests have been conducted on cylindrical samples prepared from each mixture. Lime percentage in the soils has been varied from 0 to 10 percent with fly ash ranging from 0 to 20 percent. Stabilised soil samples have been subjectd to 7, 14 and

28 day curing periods to study its effect on unconfined compressive strength. The 28-day cured specimens have been subjected to 5, 10 and 14 cycles of wetting-drying in open ended system to study the durability characteristics of stabilised soils. It has been observed that fly ash, even in smaller amounts, is beneficial in combination with lime in improving the strength of soils. With an increase in the percentage of fly ash, keeping the amount of lime as a constant in a soil-lime-fly ash mixture, strength tends to increase and reaches a certain maximum value and thereafter it starts decreasing, but is always higher than that of the respective soil-lime mixture. For a particular lime percentage, maximum strength is obtained when the lime-fly ash ratio varies between 1:3 to 1:4 in soil-lime-fly ash mixture. With an increase in amount of lime and fly ash at a particular lime-fly ash ratio, strength of stabilised soil tends to increase. In soil-lime-fly as mixture, at a lime-fly ash ratio with high lime content, strenth may be more than that at near optimum lime-fly ash ration with lesser lime content. However, greater strength can be obtained by correctly adjusting the lime-fly ash ratio that can be obtained by increasing lime content in the stabiliser. When fly ash is added to soil-lime mixtures, durability of these mixrtures to wet-dry cycles increases, with stabilised soils showing more resistance in the optimum range of lime-fly ash ratio.

When lime is added to clayey soil, certain chemical reactions take place which attribute to the strength gain of the soil. The reactions are base exchange, flocculation carbonation and puzzolanic reactions. Puzzolanic reaction contribute to the major gain in strength. Therefore, amount of cementation produced in the mixture by lime is related to the reactivity and amount of the puzzolanic material existing in the soil. When lime is added to soils containing little reactive puzzolanic material, not only may the rate of gain in strength be slow, but also the increase in the strength may be slight. Moreover, lime stabilisation, is restricted to warm to moderate climates since lime stabilised soils are susceptible to break up under freezing and thawing. When a puzzolanic material such as fly ash is added to soil, its cementation with lime is assured and the reaction will take place rapidly. Fly ash is produced as a waste material from various thermal power stations. The vast quantity of fly ash needs huge sums of money for its disposal and also creates environmental problem. Normally, on addition of water, a mixture of lime and fly ash reacts with soil to form a new material with strength and durability properties superior to those exhibited by either of the materials reacting alone. A good deal of information is available on the stabilising effects of lime and lime-fly ash mixtures on granular soils. However, in the case of fine-grained soils, the information is sparse and in some cases contradictory. In the present work a study on strength and durability characteristics of lime and lime-fly ash stabilised Black-cotton soil has been done and basis of strength evaluation is the unconfined compressive strength test.

4.5.2 Materials Used

Black Cotton Soil: Black cotton soil used in the investigation was collected from Indore in Madhya Pradesh. The principal physical properties, engineering properties, and textual composition are reported in Table 4.1. The soil is classified under CH group as per IS:1498-1970. The soil is characterised by a high plasticity index of 28.45%.

Hydrated Lime: Lime varies widely in its quality when collected from different sources or collected in batches from the same source. In order to keep uniformity in the quality of lime, high calcium calcitic lime was used throughout the investigations The properties and chemical composition of the hydrated lime have been reported in Table 4.2 and Table 4.3.

Fly Ash: The fly ash used in the investigations was obtained from the Indra Prastha Thermal Power Plant at New Delhi. The chemical and physical analysis of fly ash are given in Table 4.4.

Table No. 4.1: Properties of Black Cotton Soil Samples

Liquid limit %	70
Plastic limit %	41.55
Plasticity index %	28.45
Specific gravity	2.67
Standard proctor density gm/cc	1.47
Optimum moisture content %	25.60
Free swell index %	84
Swelling pressure kN/m^2	187
Clay %	90.30
Classification	CH

Table No. 4.2 Properties of Hydrated Lime

Specific gravity	2.05
Normal consistency	43.50
Setting time Initial Final	 2 hours 25 min 45 hours
Soundness (Le- chateliers expansion mm)	1.8
Compressive strength at 28 days kN/m^2	2200

Table No. 4.3 Chemical Composition of Lime Used

Calcium hydroxide	Min 95%
Chloride	Max 0.01%
Sulphate	Max 0.2%
Aluminum, Iron and Insoluble matter	Max 1%
Arsenic	Max 0.0004 %
Lead	Max 0.001%

Table 4.4 Physico-Chemical Properties of Fly Ash

SiO_2%	65.4
Al_2O_3 %	22.1
Fe_2O_3%	8.0
CaO %	1.7
MgO%	1.9
Na_2O%	0.2
K_2O %	0.2
Loss on ignition %	0.3
Specific gravity	2.47
Percent finer 75 micron size	82

4.5.3 Sample Preparation

Soil samples have been prepared by varying lime percentage as 0,2,4,6,8, and 10% and fly ash percentage as 0,5,10,15, and 20%. The soil was first dried in the Sun and then pulverised with a rubber mallet to pass through I.S. 2.00 mm sieve. Infrared over-dried soil was dry-mixed by hand with various amount of lime and fly ash for 2-3 minutes. Sufficient quantity of water was then added to bring the moisture content to the desired optimum moisture content, light compaction and mixed again for about 5 minutes. Cylindrical samples were compacted by static compression in compaction frame in 3.81 cm diameter steel mould to a height of 7.62 cm. Mixtures were placed in moulds with the help of a scoop and funnel. Different comparative efforts were employed to fabricate samples of various soil-lime-fly ash proportions to the desired maximum dry density as per specifications. The specimens was extruded by means of a solid metal extruder. The specimens after extraction were cured at 90-100% relative humidity in closed dissectors partly filled with water at room temperature.

4.5.4 Experimental Procedure

At the end of each curing periods (7,14,28 days), the specimens were soaked in water for a period of 24 hours. The unconfined compressive strength of the specimens was determined in the triaxial cell (without any confining pressure) with the help of a motorised loading frame, and at a loading rate of 1.0 mm/min. The load at failure was recorded and compressive strength determined. The average of atleast three similar specimens was taken for reporting purposes. Duability of soil-lime-fly ash specimens has been tested on the basis of their unconfined compressive strength. Weathering has been simulated by cyclic action of wetting and drying. Wet-dry cycles were applied on 28-day cured and 24 hour soaked specimens. The main features of the wet-dry cycle are as follows :

(I) Drying temperature in oven maintained at 70 C ;

(II) The number of wet dry cycle are 5, 10 and 14 ;

(III) The sample is subjected to 16 hours of drying, followed by 8 hours wetting in water at room temperature constituting one cycle.

4.5.5 Test Results and Discussions

The results of unconfined compressive strength tests on Black-cotton soil samples with various percentages of lime and fly ash under different testing conditions have been summarised in Tables. Referring to Table 4.5 to 4.9. It is seen that as the percentage of fly ash is increased keeping amount of lime as constant, strength tends to increase and reaches a certain maximum value and thereafter it starts decreasing, but is always higher than that of respective soil-lime mixture. For a particular lime percentage, maximum strength is obtained when lime fly ash ratio varies between 1 : 3 to 1:4 as is seen in the tables and as can also been from fig. 4.3 and 4.4. It is observed that at a particular lime fly ash percentage, increase in amount of lime and fly ash in soil increases the strength. Initially, this increase is quite high, but as the total content increases, percentage increase is on the lower side. Greater strength can be obtained by correctly adjusting the lime fly ash ratio than can be obtained by increase in lime content in the stabilizer (lime and fly ash). It is observed that when fly ash is added to soil-lime, its durability to wet and dry cycle increases and in the optimum range of lime fly ash ratio (1:3 to 1:4), the soil shows more resistance to weathering. This may be attributed to that in conjunction with fly ash, cementation of lime is assured and reactions take place rapidly. Iso –Strength curves for 7 days cure and 28 days cure wet dry cycle, fig. 4.5 and 4.6 respectively, shows the proportions for the most economical mix for the specified unconfined compressive strength at a given curing period and wet dry cycle.

Table No. 4.5 Immersed Unconfined Compressive Strength (UCCS) (q_u) of Black Cotton Soil + 2% Lime and Fly Ash mixtures under different testing conditions

Soil lime + fly ash (lime : fly ash)	UCCS (kN/m^2) After curing (days)			UCCS (kN/m^2) After wetting drying cycles (days)		
	7	24	28	5	10	14
Soil lime + 0% fly ash	176	178	178	176	180	182
Soil lime + 5% fly ash (1:2.5)	316	402	626	689	717	891
Soil lime +10% fly ash (1:5)	472	519	817	872	919	1002
Soil lime + 15 % fly ash (1:7.5)	398	428	692	690	752	784

Table No. 4.6 Immersed Unconfined Compressive Strength (UCCS) (q_u) of Black Cotton Soil + 4% Lime and Fly Ash mixtures under different testing conditions

Soil lime + fly ash (lime : fly ash)	UCCS (kN/m^2) After curing (days)			UCCS (kN/m^2) After wetting drying cycles (days)		
	7	24	28	5	10	14
Soil lime + 0% fly ash	562	689	717	762	879	912
Soil lime + 5% fly ash (1:1.25)	572	667	895	902	989	1025
Soil lime +10% fly ash (1:2.5)	703	932	1150	1178	1241	1297
Soil lime + 15 % fly ash (1:3.75)	1010	1349	1629	1754	1810	1878
Soil lime + 20 % fly ash (1:5)	992	1217	1492	1521	1655	1704

Table No. 4.7 Immersed Unconfined Compressive Strength (UCCS) (q_u) of Black Cotton Soil + 6 % Lime and Fly Ash mixtures under different testing conditions

Soil lime + fly ash (lime : fly ash)	UCCS (kN/m^2) After curing (days)			UCCS (kN/m^2) After wetting drying cycles (days)		
	7	24	28	5	10	14
Soil lime + 0% fly ash	633	1012	1132	1167	1291	1350
Soil lime + 5% fly ash (1:0.8)	639	1010	1144	1210	1309	1380
Soil lime +10% fly ash (1:1.7)	641	1091	1217	1222	1350	1419
Soil lime + 15 % fly ash (1:2.5)	826	1122	1329	1366	1470	1511
Soil lime + 20 % fly ash (1:3.3)	1162	1474	1770	1825	1988	2010

Table No. 4.8 Immersed Unconfined Compressive Strength (UCCS) (q_u) of Black Cotton Soil + 8% Lime and Fly Ash mixtures under different testing conditions

Soil lime + fly ash (lime : fly ash)	UCCS (kN/m²) After curing (days)			UCCS (kN/m²) After wetting drying cycles (days)		
	7	24	28	5	10	14
Soil lime + 0% fly ash	1206	1887	2820	3041	3295	3510
Soil lime + 5% fly ash (1:0.6)	1211	1901	2835	2980	3245	3520
Soil lime +10% fly ash (1:1.2)	1252	1988	2867	3045	3288	3497
Soil lime + 15 % fly ash (1:1.66)	1325	2041	3000	3122	3256	3455
Soil lime + 20 % fly ash (1:2.5)	1371	2072	3010	3162	3302	3692

Table No. 4.9 Immersed Unconfined Compressive Strength (UCCS) (q_u) of Black Cotton Soil + 10% Lime and Fly Ash mixtures under different testing conditions

Soil lime + fly ash (lime : fly ash)	UCCS (kN/m²) After curing (days)			UCCS (kN/m²) After wetting drying cycles (days)		
	7	24	28	5	10	14
Soil lime + 0% fly ash	1092	1441	2226	2561	2802	3145
Soil lime + 5% fly ash (1:0.5)	1220	1942	2797	2856	3091	3210
Soil lime +10% fly ash (1:1)	1226	1970	2825	2888	3110	3376
Soil lime + 15 % fly ash (1:1.5)	1257	2010	2912	2910	3250	3420
Soil lime + 20 % fly ash (1:2)	1341	2125	3122	3155	3297	3478

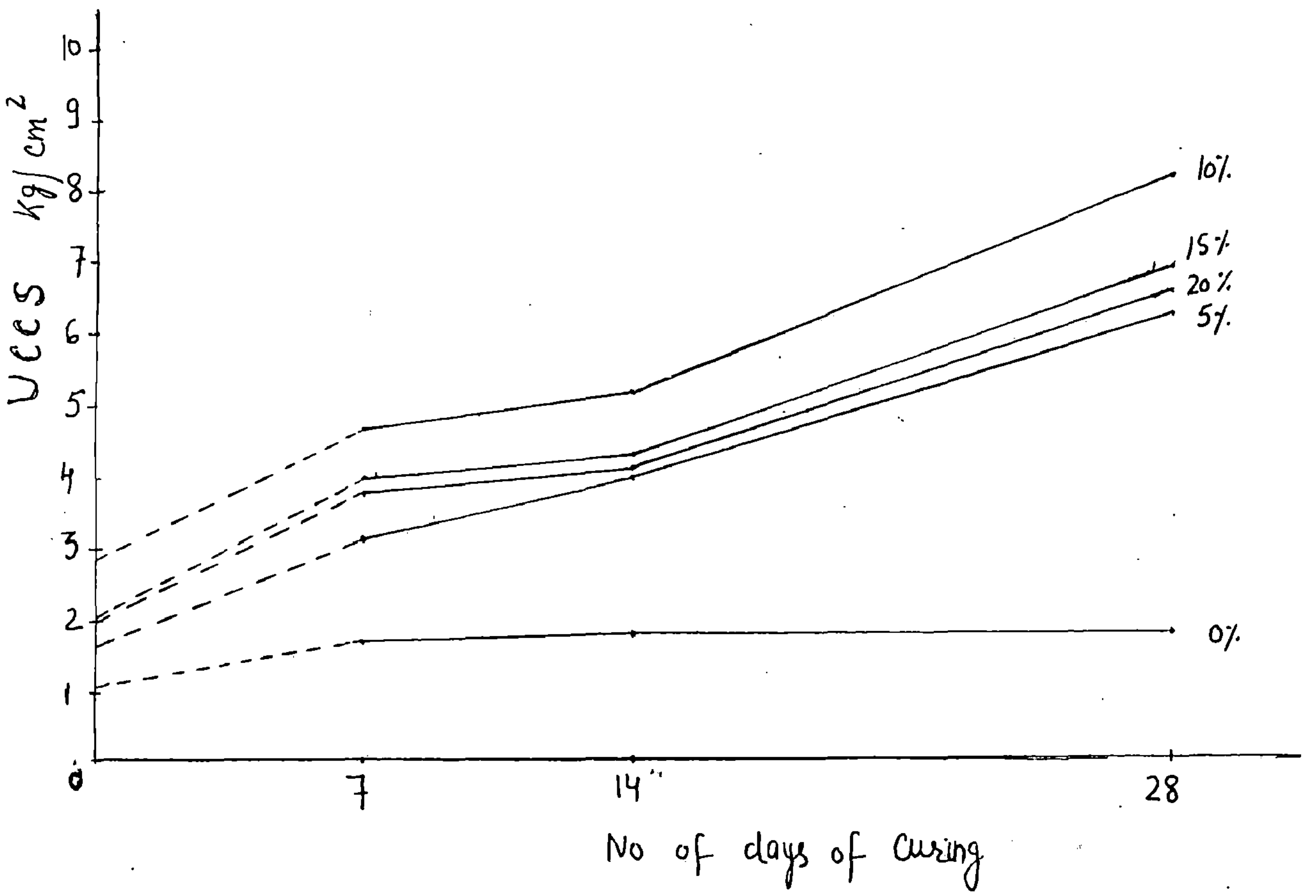


fig 4.3 Graph showing immersed UCS of Cure Soil + 2% lime & fly ash mixers. [Different %]

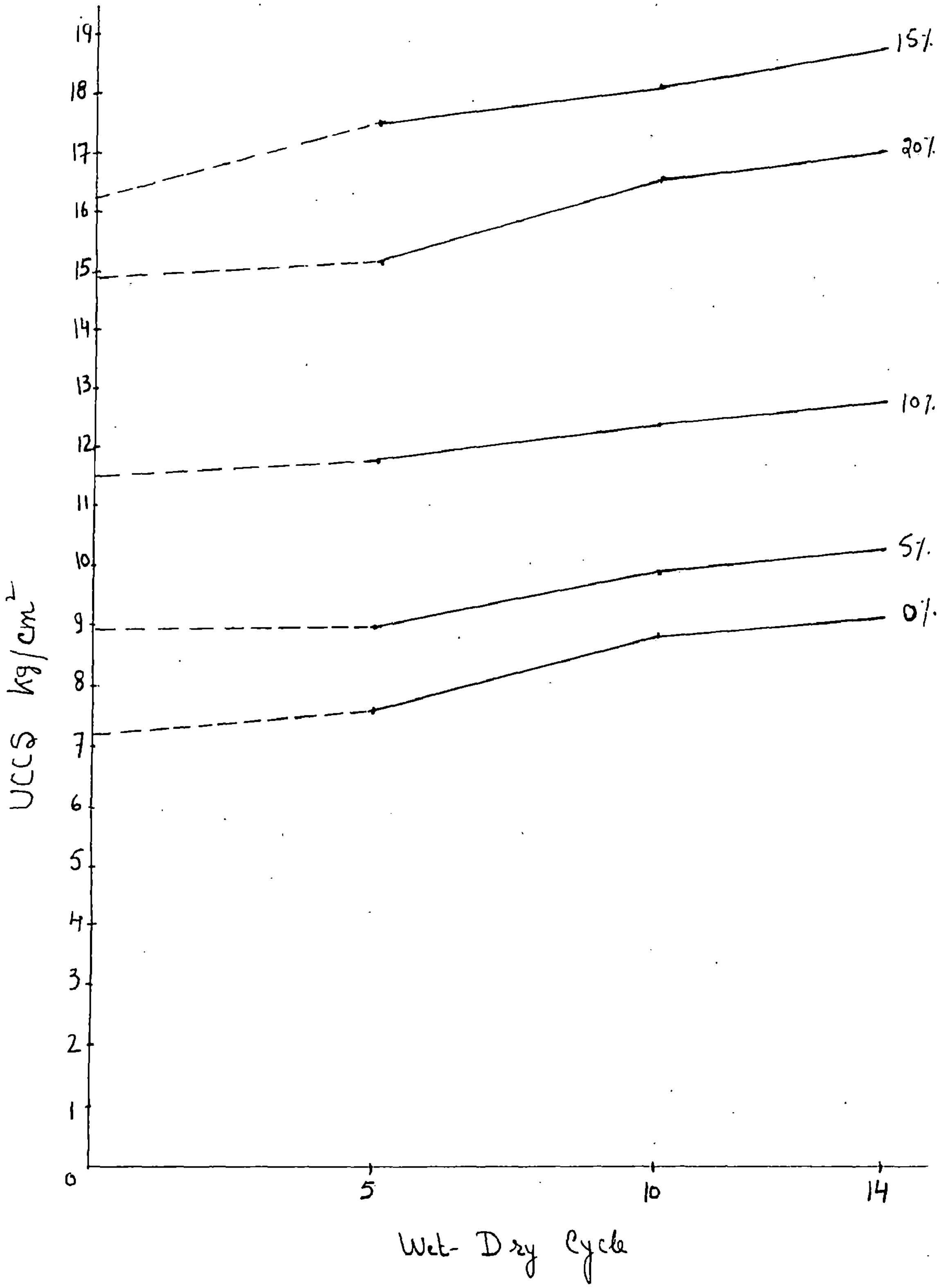


fig 4.4 Graph Showing immersed UCS of 28 days cure Soil + 4% lime & fly ash mixers [Different %]

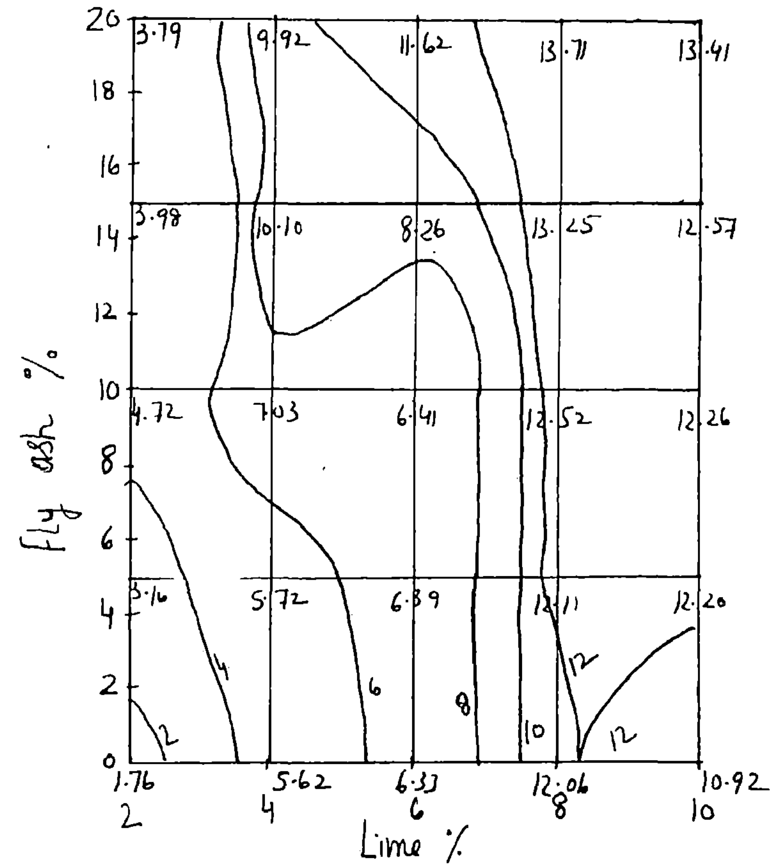


fig No 4.5 Graph showing 7 days Cure Iso-Strength Curve [kg/cm²]

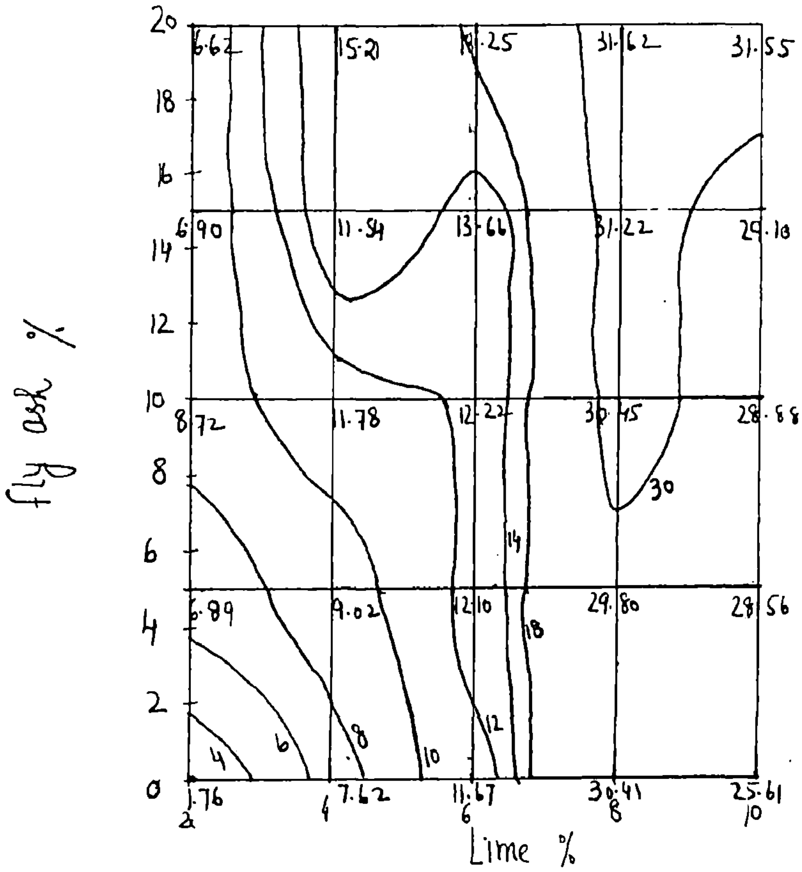


fig No. 4.6 Graph Showing 28 days Cure - 5 cycle Wet Dry Iso Strength curve [kg/cm²]

4.6 STABILISING SWELLING SOILS WITH CALCIUM CHLORIDE($CaCl_2$)

4.6.1 General

Calcium Chloride is a hygroscopic material and is quite suited for stabilization of swelling soils because it absorbs water from the atmosphere and prevents shrinkage cracks in expansive soils during the summer. The advantage of calcium chloride therefore is its moisture retaining property. It supplies cautions in exchange of the ions on the clay particles which absorbed the –ve charged clay particle reducing the intergranular electric repulsion which is chiefly responsible for reducing the swelling properties. Aggregation also immediately takes place & increases the strength.

4.6.2 Experimental Procedure

Swelling Soil (Bentonite) used for this experiment has been obtained from Indira Gandhi Nahar Pariyojana , Rajasthan. It is a highly swelling Index of 155%. The properties of Soil are given in table –4.10. These Index properties have been determined according to IS Codes.

Table: 4.10 Index Properties of the Swelling Soil (Bentonite)

Property	Test result
Specific gravity	2.67
Clay %	85.34
Liquid limit %	263
Plastic limit %	40
Plasticity index	223
Shrinkage limit %	6
Free swell index %	155
IS classification based on plasticity	CH
IS classification based on degree of expansion	High
Swelling pressure	375 kN/m^2

The optimum moisture content and the max dry unit weight of the Soil are 27.2% and 1.313gm/cc The effect of calcium chloride on plasticity, Free Swell index and unconfined compressive strength of the soil with different & of calcium chloride has been studied. All the samples are prepared at Saturation moisture contents (SMC) corresponding to dry unit weight. The % of calcium chloride used in the experiment i.e. 2,4 & 6.

4.6.3 Results and Discussions

4.6.3.1 Influence on plasticity characteristics

Influence of Calcium Chloride on Plasticity Characteristic the liquid and plastic limits, plasticity index (PI) & free swell index of the untreated soil (with 0% calcium chloride) with those of Soil treated with calcium chloride are shown in Table :4.11.

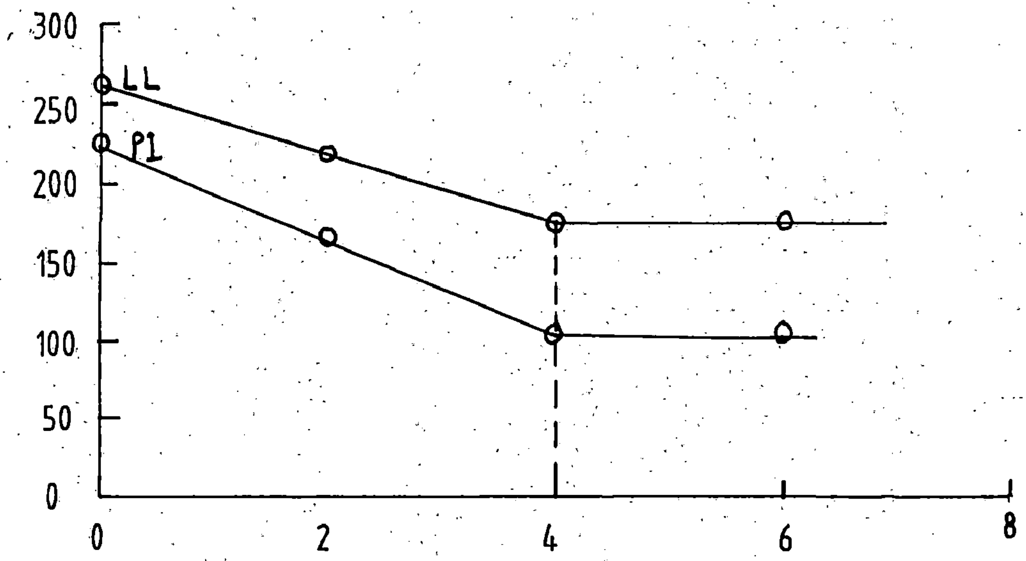
TABLE –4.11 Effect on Plasticity Characteristic & Free Swell Index

Index property	Untreated Soil	Soil treated with $CaCl_2$ at % of		
		2	4	6
Liquid limit %	263	220	173	173
Plastic limit %	40	55	70	71
Plasticity index %	223	165	103	102
Free swell index %	155	105	30	-

Considering the effect of calcium chloride on plasticity as shown in table 4.11, liquid limit decrease & plastic limit increases reducing the plasticity index of the soil considerably as can be seen from fig. 4.7. The reduction of plasticity index is max upto 4% of $CaCl_2$ & thereafter remains almost constant.

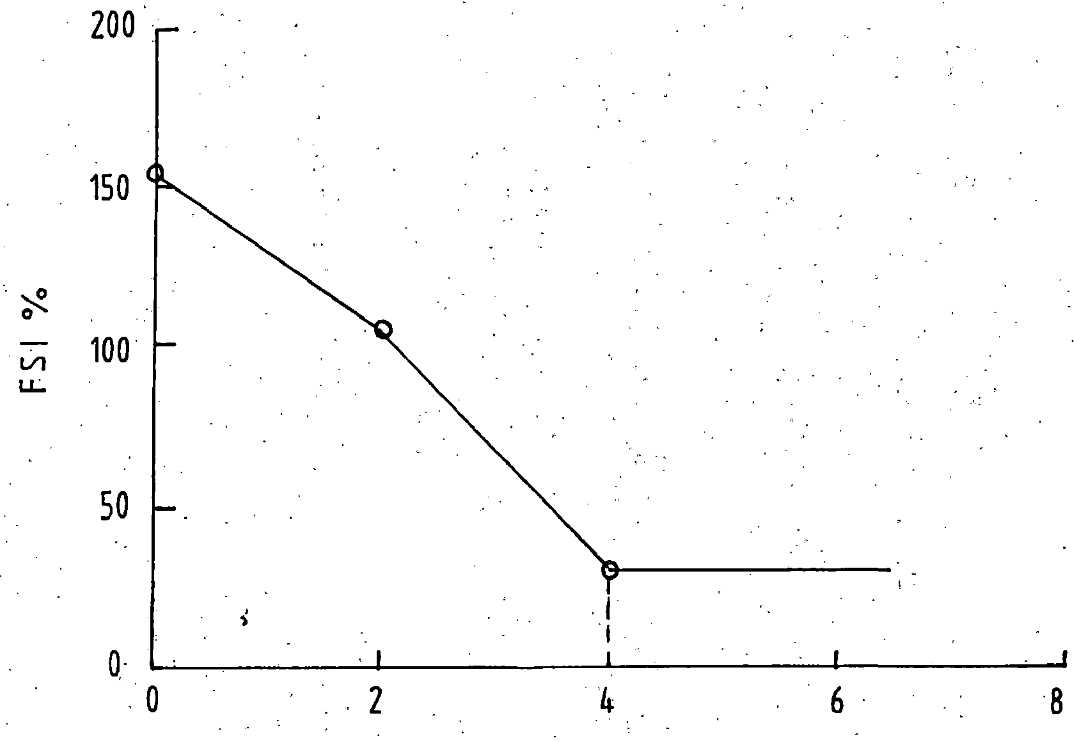
4.6.3.2 Influence on free swell index (FSI)

As seen from the table FSI has also decreased notably with the increase in the % of Calcium Chloride. At 4% of Cacl2. FSI has attained a lower value of just 30% rendering the soil swelling with the degree of Severity non-critical as can be seen from fig. 4.8.



% $CaCl_2$

Fig 4.7 Effect of calcium chloride on liquid limit & plasticity index



% $CaCl_2$

Fig 4.8 Effect of calcium chloride on Free Swell Index (FSI)

4.6.3.3 Influence on strength characteristic

The values of unconfined compressive strength (UCCS) the soil at different % of $Cacl_2$ is show in table –4.12

TABLE- 4.12 Effect on UCCS

% of $CaCl_2$	UCCS kN/m^2
0	202
2	270
4	294
6	230

As seen from the above table strength increases with the % of calcium chloride up to 4%. The strength increases only upto 4% of $CaCl_2$ and thereafter it decreases as can be seen from fig. 4.9 indicating that 4% may be taken as the optimum content of Calcium Chloride.

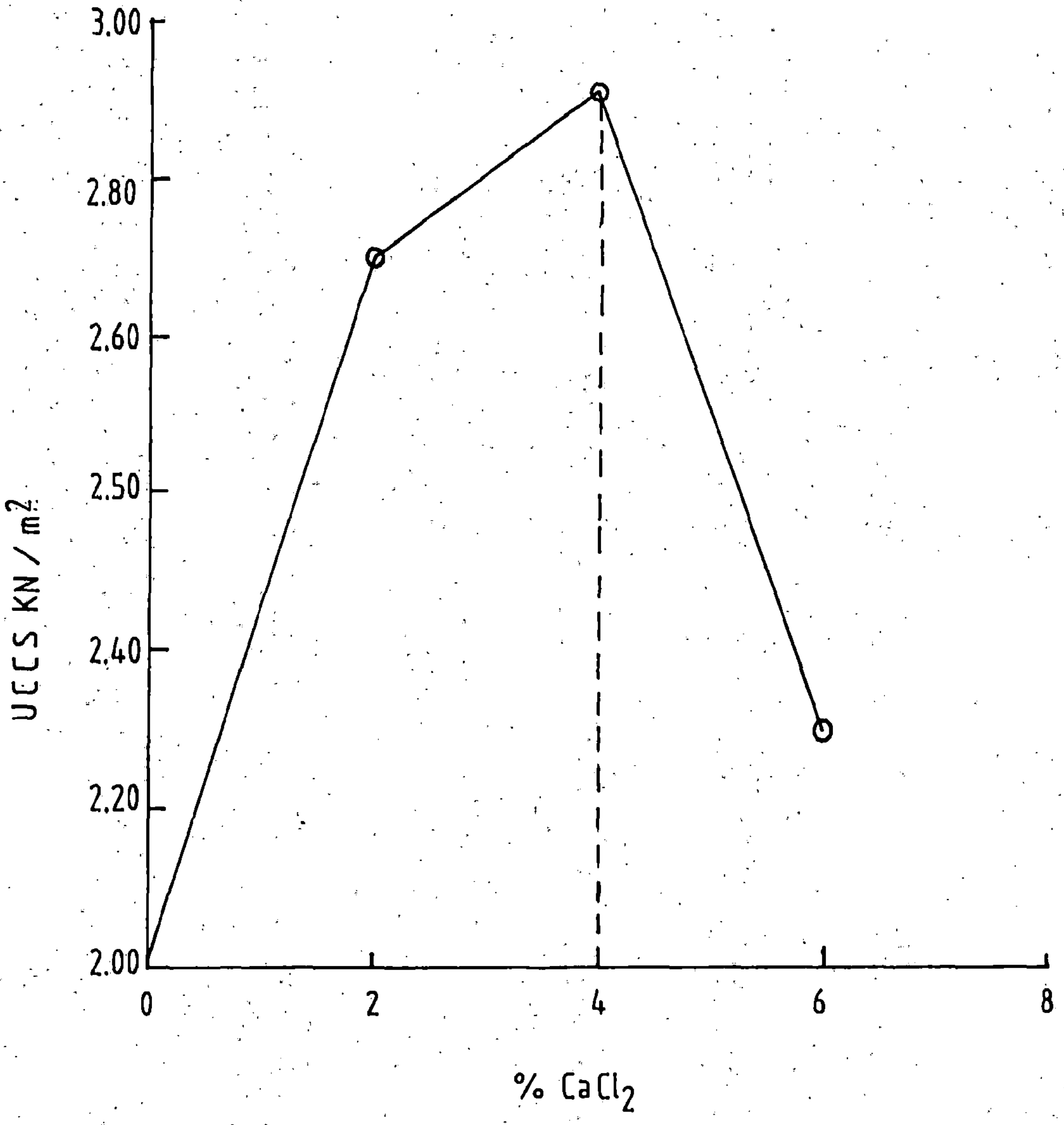


Fig 4.9 Effect of Calcium chloride on Unconfined Compressive Strength of Soil Sample

CONCLUSIONS

5.1 GENERAL

Expansive soils are notorious in behavior because of their swelling and abnormal volume change characteristics due to imbibing moisture. This leads to very low shear strength and bearing capacity. These soils are highly problematic when called upon for supporting foundation of structures related to Water Resources Development. These also cause problem when it is used as a material for other construction purposes.

On the basis of limited laboratory investigation data the following conclusions have been drawn

1. The values of liquid limit, shrinkage limit, plasticity index and other swelling properties indicate that the sample 1 and 3 may be categorized as highly expansive soil (Chandra Prakash 1988). Whereas the values for sample no. 2 are at boundary level indicates that the soil is not having very much swelling characteristics
2. From the consolidation test data it is observed that values of coefficient of permeability (k) are very low and of the order of 10^{-7} cm/sec indicating that the soils are highly impermeable. Stress –strain curve also show that the soils have been over consolidated upto a stress level of 200 kN/m^2
3. Triaxial test result shows very low shearing strength for all the soils tested.
4. The results of free soils index confirms that the sample no. 1 and 3 having the FSI 84% and 155% are highly expensive soils whereas the sample no. 2 having FSI less than 20% is having low degree of expensiveness. The swelling pressure intensities for sample 1 and 3 are 187 kN/m^2 and 375 kN/m^2
5. In order to stabilize the expansive soils, various test results have been performed and it has been found that

a. Stabilization with lime and fly ash

(i) For a particular soil + lime mixture, a small quantity of fly ash increases the cured strength of the mixture.

(ii) With the increase in the percentage of fly ash keeping amount of lime as constant in a soil – lime fly ash mixture, the strength tends to increase and reaches a certain maximum value there after it starts decreasing. For a particular lime percentage, maximum strength is obtained when the lime fly ash ratio ranges between 1:3 to 1:4 in a soil lime fly ash mixture.

b. Stabilization with Calcium Chloride ($CaCl_2$)

(i) Calcium Chloride reduces the plasticity characteristics of soiling soil and also reduces the free swell index indicating that it mitigates the problematic swelling characteristics of expansive soils

(ii) Unconfined compressive strength increases with the increase in the % of calcium chloride. 4% of $CaCl_2$ may be taken as the optimum content with respect to strength.

(iii) With regards to FSI and plasticity, 4% of $CaCl_2$ may be taken as optimum content

(iv) Due to the hygroscopic nature of the $CaCl_2$ the expensive soils treated with $CaCl_2$ will not form shrinkage cracks during summer. Hence it can be taken as better stabilizing agent for expensive soil.

5.2 SCOPE FOR FURTHER STUDIES

1 As seen from the mohr circles failure envelope that the soil is over consolidated even in the region of circle for effective stress of 1 kg/cm^2 [100 kN/m^2]. Therefore studies can be done even at effective stress less than 1kg/cm^2.

2 Behavior of the soil under cyclic loading conditions can be studied, as soil may be a part of foundation of any heavy machinery in Water Resources Structures.

3. During the soil stabilization with calcium chloride [$CaCl_2$] the optimum value of $CaCl_2$ has been found to be at 4 % in the range of 0, 2, 4 and 6 % of $CaCl_2$. Studies can be further made to find out the optimum value of $CaCl_2$ at 3% or less than 4% of $CaCl_2$.

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