NONDESTRUCTIVE EVALUATION AND STUDY OF VARIOUS PARAMETERS AFFECTING THE STRENGTH OF SOIL

THESIS

Submitted in partial fulfillment
of the requirements for the degree of

DOCTOR OF PHILOSOPHY

By

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PILANI (RAJASTHAN) INDIA

2001

Dedicated to,

Loving Parents
&
Wife Vani

BIRLA INSTITUTE OF TECHNOLOGY & SCIENCE
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CERTIFICATE

This is to certify that thesis entitled **NONDESTRUCTIVE EVALUATION AND STUDY OF VARIOUS PARAMETERS AFFECTING THE STRENGTH OF SOIL** and submitted by **Kamalesh Kumar** ID NO. **1995PHXF006** for award of Ph. D. degree of the Institute, embodies original work done by him under my supervision.

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ACKNOWLEDGEMENTS

I sincerely thank Prof. Ravi Prakash, Dean, Research and Consultancy division, BITS Pilani for supervising the thesis. His constant encouragement and guidance during the entire duration of thesis work and his invaluable suggestions helped me in completing the thesis.

Thanks are also due to Dr. S. Ghosal, Professor, Civil engineering, BITS for providing technical information pertinent to the thesis. I also thank Dr. Damodar Reddy, Ex. Chemistry faculty of the institute for his valuable guidance during the initial phase of thesis work.

My sincere thanks to Dr. Rajiv Gupta, Associate Professor, Civil engineering, BITS for his valuable suggestion and guidance regarding technical and software aspect of thesis work.

Sincere thanks are due to Professor S. Venkateswaran, Director, BITS, Prof. L. K. Maheshwari, Prof. K. E. Raman & Prof. V. S. Rao, Deputy directors, BITS, members of research board and members of Doctoral counseling committee for providing an environment conducive for completing the thesis work.

I sincerely thank anonymous reviewers of Journal of pure and applied ultrasonics and of Journal of Institution of Engineers (civil engineering) for accepting two research papers which were accepted for publication in their journals based on present experimental study.

Thanks are due to Prof. H. Y. Fang and Prof. S. Pamukcu, Professors, Civil engineering, Lehigh university, Bethlehem PA, for guiding in getting a topic suitable for the present thesis work based on current day practical applications during my graduate studies.

I also thank Prof. Yudhbir, Prof. M. R. Madhav and Prof. P. K. Basudhar, Professors, Civil engineering, IIT Kanpur, who were helpful in getting interest and selecting a topic of soil behavior for present thesis work during my undergraduate studies.

Valuable discussions with faculty members of Civil Eng., BITS during the entire duration of thesis work and the computational facilities of the information processing center of the institute are greatly appreciated.

My sincere thanks to Prof. M. M. S. Anand, librarian, BITS as well as to librarians at IIT Delhi and CSMRS Delhi for providing literature useful for the present thesis.

Sincere thanks to Mr. A. K. Soni and other non teaching staff of Civil Eng. and materials testing laboratory of the institute for their help while doing experiments related to the thesis work.

Finally I also thank my family members, wife Vani and Sons Vikhyat and Saksham for providing suitable environment at home for completing the thesis.

ABSTRACT

Shear strength behavior of soils have been discussed in detail in the present study. Shear strength of soils have two distinct components, one is called the cohesive component and the other one is called the frictional component. Literature study has been made to study the origin of these components of shear strength based on interparticle interactions. Factors affecting aforementioned components of shear strength have also been discussed. Effect of soil particle size composition, pore water content and pore water salt concentration onto its shear strength components have been discussed in the experimental framework.

Effect of reinforcing soils with fibres have also been discussed in detail. Literature study has been made to study different types of fibres commonly used as reinforcing agents in soils as well as failure mechanism of fibre reinforced soils. Important soil and fibre parameters affecting behavior of fibre reinforced soil have also been discussed. Effect of increasing weight fraction of E-glass fibres (with fibre orientation perpendicular to failure surface) onto cohesion and angle of internal friction of fibre reinforced soil has been discussed in the experimental framework. Strength deterioration of fibre reinforced soil composite with time also has been discussed in the experimental framework.

Ultrasonic testing of soils have also been conducted in the present study. Literature study has been made to understand ultrasonic testing technique as an useful testing technique.

Propagation and attenuation characteristics of ultrasonic pulses have been discussed in detail. Literature study also has been made to understand about different types of ultrasonic pulses and methods of measuring them. Effect of soil particle size composition onto ultrasonic pulse velocity through it has been studied in the experimental framework. Furthermore, based on experimental studies calibration curves have been developed to estimate cohesion, angle of internal friction and safe bearing capacity of soils by knowing ultrasonic pulse velocity through them.

Experimental observations of present study have been explained based on the information available in the literature. Finally, practical uses of present experimental study have also been discussed.

LIST OF SYMBOLS

SYMBOL	MEANING
a_a	Fraction of particle surface area in contact with air
a_m	Fraction of particle surface area in contact with other minerals
a_w	Fraction of particle surface area in contact with water
A	Long range attractive force
A_0	Initial amplitude
A_t	Transmitted signal amplitude
A_{area}	Cross sectional area of shear box
B	Width of strip footing
c	Cohesion intercept
c'	Effective cohesion intercept
C	Composition
D_f	Depth of embedment of footing from ground level
e	Void ratio
E	Young's modulus of elasticity
H	Stress history
N_c	Terzaghi's bearing capacity factor for general shear failure for cohesion
N'_c	Terzaghi's bearing capacity factor for local shear failure for cohesion

SYMBOL	MEANING
N_q	Terzaghi's bearing capacity factor for general shear failure for effective surcharge
N'_q	Terzaghi's bearing capacity factor for local shear failure for effective surcharge
N_γ	Terzaghi's bearing capacity factor for general shear failure for depth
N'_γ	Terzaghi's bearing capacity factor for local shear failure for depth
P_H	Applied horizontal load
P_V	Applied Vertical load
q	Effective surcharge at the base level of footing
q_d	Ultimate bearing capacity
R	Long range repulsive force
S	Structure
SBC	Safe bearing capacity
t	Sample thickness
T	Temperature
u_a	Pore air pressure
u_w	Pore water pressure
V	Ultrasonic pulse velocity
V_{app}	Apparent ultrasonic velocity
w	Water content
σ	External force per unit area
σ_m	Mineral to mineral contact stress
ρ	Density

SYMBOL	MEANING
ν	Poisson's ratio of the material
α	Ultrasonic attenuation
τ	Shear stress
τ_{ff}	Shear stress on failure plane at failure
τ'_{ff}	Effective shear stress on failure plane at failure
σ_{ff}	Normal stress on failure plane at failure
σ'_{ff}	Effective normal stress on failure plane at failure
ϕ	Angle of internal friction
ϕ'	Effective angle of internal friction
ϕ'_{mod}	Modified angle of internal friction
ε	Strain
ε'	Strain rate
γ	Total unit weight

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1. INTRODUCTION

Strength related properties of soil are of great importance to soil scientists. When construction work is done or foundations are laid; soil should have sufficient strength to support the incoming load (Bolton, 1986). It is necessary for the soil scientist to have an idea of soil strength and also to know the parameter which affect soil strength. Some of the properties which are useful to a soil scientist are cohesion, angle of internal friction, compressive strength, shear strength, tensile strength etc (Crawford, 1963).

These strength related properties of soil at the macro level arise due to interaction between soil particles at the micro level (Sridharan and Venkatapparao, 1979). There are varieties of factors related to soil system and surrounding environment, which affect the soil strength. This is basically due to interparticle interaction between soil particles.

Non-destructive testing (NDT) techniques have become an important tool for studying the properties of various materials being used for civil engineering applications (Xiutang et al, 1996). Generally certain parameters, which can be measured using nondestructive testing techniques, are experimentally obtained and correlated with the strength of the material. This is because of the fact that these parameters are dependent upon material properties. Ultrasonic pulse velocity testing is one of the most important non-destructive testing techniques available. Ultrasonic response from soil samples obtained from different locations, having varied soil properties and strength has been studied in detail by Chandra et al, (1991).

1.1 SOIL STRENGTH BEHAVIOR

Soil is used as an engineering material for variety of applications. Properties such as bearing capacity, shear strength of the soil, compressive strength and cohesion are of great practical importance to soil scientists (Venkatapparao and Moondra, 1976). Soil particles are particulate in nature, which distinguishes them from solids and liquids. They come in a large range of particle size, ranging from coarse sand (2mm size) to clay colloids (10^{-6} mm size). Clays are distinguished from sands mainly due to particle size and specific surface area associated with the smallest particle unit for both. ($800x10^{6}$ mm^2/gm for a typical clay mineral as compared to $2x10^{3}$ mm^2/gm for a typical sand particle). (Nagaraj, 1995).

Sand particles are more bulky than clay particles. These two soil systems have very different strength behaviour. The strength of sand as well as that of soil gets affected due to a variety of factors. Knowledge of these factors that affect strength and their proper explanation based on interactions at micro-level is very important to soil and foundation engineer (Ramanasastry and Ganeshkumar, 1989).

Sand particles are isometric in shape and exist as individual units. However clay particles are more like a continuous unit. At macro level, it is not easy to single out an individual unit of clay. They are hydrated aluminum silicates and hydrous oxides of aluminum, magnesium and iron in crystalline form.

INTERPARTICLE INTERACTIONS

Between two units of matter, there are always interacting forces present. They can either be attractive or repulsive. Same phenomena are observed between individual sand and clay units. The magnitude of a particular type of interaction changes across the sandy and clayey type of particle due to change in soil microstructure.

Interacting forces (both attractive and repulsive) acting between two particle units can be broadly classified into:

(a) Short range forces

(b) Long range forces

Action radius of short-range forces is very small (few Angstroms). Forces arising due to mineral contact, mass, homopolar and hetropolar forces, hydrogen bonding and born repulsive forces come under this category.

Long range forces act over a long distance (several hundred Angstroms) and they are van der Waals attractive forces and Coulomb repulsive forces. These forces originate between particles of colloidal dimensions (clays in suspension for example). Long range repulsive forces are dominant over long range attractive forces (Nagaraj, 1995).

Short-range forces are dominant in the case of sands. These forces originate due to actual mineral contact between particles and due to particle mass. Long-range action radius is much larger than short-range action radius when few particles come in contact at rough surfaces. Due to this, the long-range forces are not important in case of sands. Frictional strength results due to short-range interaction (Bolton, 1986).

Long range forces are dominant in case of clays. Smoother particle surface for the case of clay spreads the mineral contact points to a large area leading to reduced short-range force effect. Long range repulsive forces are dominant over long range attractive forces, which keeps fine grained stable particle units separated and thus preventing direct physical contact.

PORE FLUID EFFECT

Sand particles have low surface activity. Hence effect of pore fluid on sand particle interaction is minimal. Physicochemical reaction between sand and pore fluid is low. As a result short range forces arising due to mineral contact does not get greatly affected and frictional response of the sand, more or less remains same. Generally presence of pore fluid facilitates presence of hydrated ions or contaminants. However, this phenomenon is not found in sands due to their low surface activity. In the presence of pore fluids, interaction between clay particles gets modified in the way described below:

(i) Clay particles carry a net negative charge. When clay particle is placed in water, positively charged cations swarm around the clay particle. These positively charged cations are present due to dipolar nature of water, as well as free positive ions present in it. When cations swarm around clay particle, negative surface charge on clay particle neutralizes. Concentration of cations is maximum at clay surface. It decreases while going away from surface and attains concentration corresponding to neutral water at some distance away from clay surface. The distance to which it takes place is called double layer thickness (Lambe, 1958). The charged surface and strongly held cations close to the surface form two layers.

(ii) Double layer thickness is greater than the micro roughness of surface of clay particles under aqueous conditions. This prevents the mineral to mineral contact in clay particles making short range forces insignificant. Also long range repulsive forces are dominant over long range attractive forces. This keeps stable particle units separated preventing direct physical contact.

(iii) Long range repulsive forces are sensitive to pore fluid properties. Hence by varying pore fluid characteristic, long range repulsive forces between clay particles can be varied. This phenomena is not pronounced in dilute suspension because distance between clay particles is much larger than the action radius of long range forces under such conditions.

(iv) Short-range forces are significant under edge to face interaction conditions. Increased ion concentration of pore fluid also increases short range force effect. These result in depression of double layer. Long range forces are also present. Hence the final response of clay under pore fluid presence depends on clay microstructure, clay concentration, pore fluid characteristics and ion concentration of pore fluid (Bansal and Ghuman, 1997).

EXTERNAL AND INTERPARTICLE STRESSES

Soil particles are idealized as parallel plates with inter-granular friction at the point of contact. When all external and internal forces are combined, following expression relating internal stresses with total external force per unit area has been suggested (Sridharan, 1991):

$$\sigma = \sigma_m a_m + u_w a_w + (R-A) + u_a a_a \tag{1.1}$$

This expression provides the most general relation under partially saturated conditions. σ_m is mineral to mineral contact stress, u_w is pore water pressure and u_a is pore air pressure. Fraction of particle surface area in contact with other minerals is denoted by a_m. Also, a_w refers to that it is with respect to water and similarly a_a refers to that it is with respect to air. **R** is long range repulsive force and **A** is long range attractive force. This general relation gets modified for dry and saturated conditions. Also significance of a particular term on the right hand side of the expression changes with soil types as well as with mineral to mineral contact between soil particles.

For highly plastic dispersed clay in fully saturated state, long range forces are far more significant than mineral to mineral contact short range forces as has already been discussed. Also long range repulsive forces are dominant over long range attractive forces. Hence **(R-A)** contribution to strength is more significant compared to other contributions in such type of clays.

For the coarse grained soils, mineral to mineral contact short range forces are the most dominant at the particle level. This is due to existing micro-roughness of particle surfaces at that level. As a result $\sigma_m a_m$ term is the most important contributor to external stress. Furthermore, soil strength is dependent on external stress.

Even under actual field conditions, a mixture of coarse and fine-grained soils are found. As a result both **(R-A)** and $\sigma_m a_m$ term will play an important role in determining strength. The contribution of a particular term will become large or small based on fraction of coarse and fine grained soil in a given soil mixture.

Pore fluid, its dielectric constant, presence of free ions and other pollutants in it also affect u_w, $\sigma_m a_m$, as well as **(R-A)**. They in turn result in alteration in strength related properties both for coarse and fine-grained soils. In the present investigation, an experimental study was undertaken to study the effect of water content and electrolyte concentration of pore fluid on soil strength.

1.2 FIBRE REINFORCED SOIL

Soil particles interact at the micro-level in the soil matrix. This interaction gets modified under the presence of pore fluid. As a result strength of bulk soil system changes. From engineering point of view it is always desirable to enhance the strength of soil system by adding external agents into it. Reinforcing fibres have been successfully and effectively used in concrete and cement structures to obtain desirable strength related properties (Zhao and Michalowski, 1996). Besides cement and concrete, these fibres can also be used as reinforcement in soils to enhance their strength.

Variety of natural fibres have been used as strength enhancing agents in the soil matrix. Banana, jute and coconut fibres are some of the important ones. Lately variety of synthetic fibres have also been developed to act as reinforcing agents for the soil matrix. These reinforcements are classified as follows (Sonobe et al, 1997):

(A) Classification by Fibre Type

(i) Inorganic

(ii) Organic

- Inorganic
  - Glass fibres
    - Alkali-resistant glass
    - E glass
  - Carbon fibres
    - PAN-based
    - Pitch-based

- Organic
  - Aramid fibres
  - Polyvinyl alcohol (vinylon) fibres
  - Others

(B) Classification by Binders

(i) Organic

(ii)Inorganic

- Organic
  - Epoxy
  - Vinyl ester
  - Unsaturated polyester
  - Others

Inorganic (Specialised cement etc.)

(C) Classification by Shape

- FRP reinforcement
  - Bar type
    - Linear
      - Round bar
        - Surface treatment
          - Fibre-wound
          - Sanded
          - Others
        - W/O surface treatment
      - Deformed
      - Square bar
        - Surface treatment
        - W/O treatment
    - Stranded
    - Braided
      - Surface treatment (Sanded)
      - W/O treatment
  - Planner/three-dimensional reinforcement

Soil matrix has low modulus. It is reinforced with high strength and high modulus fibres. Plastic flow of soil matrix under stress transfers load to the fibres. This results in high strength, high modulus composite. The principle constituents that influence strength and stiffness of the composites are the reinforcing fibres, the soil matrix and the interface. Each of these individual phases has to perform certain essential functional requirements based on their mechanical properties so that the system as a whole will perform satisfactorily (Leung, 1996).

Following are the desired functional requirements of the fibres, which are used as reinforcing agents in the soil matrix:

~ Fibres should have high modulus of elasticity in order to give efficient reinforcement.

~ Fibres should have high ultimate strength.

~ Variation of strength between individual fibres should be low.

~ Fibres should be stable and retain their strength during handling and fabrication.

~ Diameter and surface of fibres should be uniform.

Matrix in which fibres are reinforced should satisfy following functions:

~ It should bind together the fibres.

~ It should protect their surface from damage during handling and fabrication.

~ It should disperse the fibres and separate them so as to avoid any catastrophic failure.

~ Matrix should transfer stress to fibres efficiently by adhesion and friction.

~ Matrix should be chemically and thermally compatible with fibres over a long period.

Also interface between fibre and matrix should provide adequate physicochemical stable bonding between the two. This will enhance strength of soil matrix (Freitag, 1986).

In fibre reinforced soil system, certain factors which influence the mechanical properties such as strength and stiffness, are given below:

Mechanical properties of fibre and matrix

~ Fibre volume fraction of the composite.

~ Degree of fibre matrix interface adhesion.

~ Fibre cross section.

~ Fibre orientation within the matrix.

GLASS FIBRE REINFORCEMENTS

Variety of reinforcements could be added to the soil system to enhance its strength. These reinforcements are classified as ideally extensible & inextensible. Glass fibres are extensible due to their low modulus. Maximum force in fibre is controlled by deformation in soil under these conditions. They are broadly divided as E-glass and S-glass fibres.

E-glass is available as continuous filament, chopped staple, and random mats suitable for most methods of resin impregnation and composite formation. S-glass, originally developed for aircraft components and missile casings, has the highest tensile strength of all fibres in use. However, the compositional difference and higher manufacturing cost make it more expensive than E-glass. A lower-cost version of S-glass, called S-2 glass, has been made available in recent years. Although S-2 glass is manufactured with less stringent non-military specifications, its tensile strength and modulus are similar to those of S-glass. S-glass is primarily available as rovings and yarn, and with a limited range of surface treatments (Schwartz, 1997).

Adding glass fibres into matrix inhibits crack propagation and provides ductility. It also increases tensile strength and fracture energy. Presence of glass fibres holds the matrix together. Glass fibres are stiffer than matrix. Hence small amount of fibre added is adequate to provide sufficient improvement in properties.

Properties of soil matrix reinforced with glass fibres is influenced by a variety of factors. It includes volume fraction of glass fibres, orientation, length and manufacturing methods. Type and density of soil matrix, as well as moisture content also play an important role (Mandal, 1987).

As commercial reinforcements, glass fibres are produced as roving, chopped strands, mats, fabrics and woven rovings. Glass-fibre mat is a blanket of chopped strand or of continuous strands laid down as a continuous thin flat sheet (Schwartz, 1984). In the present investigation E-glass fibres in the form of mat were used. Fibres of required length were obtained from that mat. They were then used as reinforcements in soil.

E-glass fibres are less durable as compared to S-glass fibres. However E-glass fibres being more economical, are extensively used. For the present study also E-glass fibres were used.

1.3 ULTRASONIC TESTING TECHNIQUES

Strength of soil is an important soil property. It depends on the characteristics of soil system and properties of surrounding environment. Conventional method of testing involves finding out properties of soil and then determining soil strength. These conventional methods involve disturbance of soil sample, which in turn puts restriction on to the usefulness of the results.

Several non-destructive testing (NDT) techniques, have been developed to test materials of civil engineering importance. In these methods of testing, certain non destructive parameters of the material are determined experimentally. Then a correlation is drawn between those measured non-destructive parameters and engineering properties. Soil strength is one such very important property. During the testing, sample is not destructed mechanically. This makes testing convenient to perform and reliable information is obtained. It also allows performing in-situ testing without disturbing the sample.

Wave based non-destructive tests are a special class of non-destructive testing technique. In these methods information about elastic wave travel through the media helps to understand the characteristics of the media. When elastic waves travel through soil samples, the characteristics of primary and secondary elastic waves changes with changes in properties of the soil such as elastic moduli, density, moisture content, void ratio, porosity, degree of saturation and particle size composition. Hence variation in any one of these parameters can be correlated with the changes in the behaviour of elastic waves provided other parameters remain unaltered. Strength of soil is also dependent on the above mentioned characteristics of soil. Hence a correlation can be developed between wave characteristics and strength of the soil.

Ultrasonic pulse velocity testing is a long established non-destructive testing method. It involves determination of velocity of longitudinal waves through the sample. Velocity measurement is then correlated with properties of sample such as strength of the sample. In the present research study use of ultrasonic technique has been made for testing of soils. This method involves determination of longitudinal (compressional) wave velocity through sample. This can be achieved by measuring time taken by a pulse to travel a measured distance in the sample. Transducers are placed in contact with the sample and low frequency transducers are used for this purpose. Frequency used was 150kHz which is very small as compared to MHz frequencies used for NDT of metallic materials.

Measurement can be done using through transmission technique. In this method transmitting and receiving transducers are placed on the opposite faces of the sample. The axes of the transducers are aligned. Measurement can also be done by pulse echo or impact echo techniques. Here transmitting and receiving transducers are placed on the same side of the sample. In impact echo technique, elastic pulses are generated from an impact source placed on the surface. However in pulse echo technique ultrasonic pulses are generated by a piezo-electric transducer, placed on the surface. In both the methods receiving transducers are placed at some distance on the same side of the surface to receive the pulse.

Velocity of these pulses depends on the density and elastic properties of the sample. Generally quality of material is related to its elastic stiffness. Hence measurement of ultrasonic pulse velocity in such materials can often be used to indicate their quality as well as to evaluate their elastic properties (Brandt, 1955). These concepts have been applied to the soil samples used in the present research work.

The pulse velocity is determined by using the single equation:

Pulse velocity=path length/transit time (1.2)

This single equation can be applied to transmission of pulses through material of any shape or size. Only restriction being that the least lateral dimension (dimension measured perpendicular to the path of pulses) should not be less than the pulse amplitude.

Ultrasonic pulse velocity 'V' for an elastic homogenous material is given by:

$$V^2=E\ (1-\nu)/\rho\ (1+\nu)(1-2\rho) \tag{1.3}$$

Where,

E=Young's modulus of elasticity

ρ=Density

ν Poisson's ratio of the material

E, ρ as well as ν are dependent on the microstructure of the material. They also get altered due to interaction of particles at the micro level within the material. These factors also affect strength related properties of the material. Hence pulse velocity can be correlated with sample strength directly or by incorporating some more parameters of the material (Patterson, 1956).

The pulse velocity is not affected by the frequency of the pulse. As a result the wavelength of the pulse vibrations is inversely proportional to its frequency. Thus pulse velocity will generally depend only on the properties of materials. Measurement of this velocity enables an assessment to be made of the conditions of the material. There is considerable amount of attenuation of the pulses when they pass through materials of civil engineering importance. Hence frequency used for civil engineering materials is in kHz range and not in MHz range.

For assessing quality of materials from ultrasonic pulse velocity measurement, measurement should be of high accuracy. Path length and transit time should each be measured to an accuracy of about $\pm$ 1%. Time taken for earliest part of pulse to reach receiving transducer from the time it leaves transmitting transducer is measured. While doing measurement, transducers are placed at suitable points on the surface of the material. Depending on relative placement of transducers on the surface of the specimen, transmission could be either direct, indirect or semi-direct. Direct transmission is most adequate. In this technique longitudinal pulses leaving transmitter probe are propagated mainly in the direction normal to the transducer face.

Attenuation characteristics of the sample can also be determined by using the amplitude of received pulses. Following relation is used for calculating attenuation in dB/mm (Ensminger, 1973) :

$$\alpha \text{ (dB/mm)} = -20/t \log_{10}(A_t/A_0) \qquad (1.4)$$

Where,

A_0= Initial amplitude.

A_t= Transmitted signal amplitude.

t= Sample thickness (in mm).

α= Ultrasonic attenuation.

Pulse transmission through sample gets affected due to variety of reasons:

~ Presence of voids in the transmission path.

~ Non homogenity of the sample.

~ Moisture content of the sample.

~ Density of the sample.

~ Degree of compaction.

~ Mix proportion of the sample.

Thus with change in these properties of the sample, ultrasonic transmission velocity also changes. These relationship have been studied for soil samples collected from different locations, in the present research.

1.4 SCOPE OF PRESENT WORK

Present research is aimed towards study of local soil behavior. This will help in understanding the engineering properties of soil. Study has been conducted on the experimental framework. Experimental results obtained have been explained based on conceptual facts already known. Computing facilities have been used wherever necessary for data analysis.

Locally available soils of sandy and silty clayey type have been collected. They were subjected to sieve analysis as per specifications. This helps to separate particles of specific size in both the type of soils. Particle size distribution of these two soils was obtained. Specific size particles from both type of soils were taken. Soil mixture containing a given fraction of each was prepared. Their strength related properties were also determined. Strength of soil mix are dependent on its important physical characteristics. Hence some of the important physical characteristics affecting soil strength were investigated. Soil samples collected from different locations but having similar physical properties are expected to show similar strength behavior.

Soil mixture containing varied combination of sand and silty clay (of specific particle size) were prepared. They were subjected to direct shear testing. This helps to determine variation of direct shear parameters when composition of sand and silty clay in the soil mixture changes. Water content and all other parameters were kept constant during testing. Results thus obtained provide the much needed information about the optimum combination of sand and silty clay to get maximum shear strength parameters.

Effect of pore fluid amount as well as variation of salt concentration in pore fluid was also studied. Direct shear strength parameters were determined when the moisture content of the samples was changed keeping all other parameters constant. Similarly the effect of pore fluid salt concentration was also studied. Thus salt concentration and moisture content corresponding to optimum strength parameter were obtained.

Natural as well as synthetic fibres have been used as reinforcements in materials of civil engineering importance. This enhances strength related characteristics. Glass fibres were used as reinforcing agents and an effort was made to study as to how does this reinforcement affects the physical characteristics of the soil. Effect of fibre content in soil sample on the direct shear parameters was also studied.

Detailed study of ultrasonic testing of soil samples was conducted. Variation of ultraonic transmission velocity was studied when the particle size composition of the soil mixture was changed. Other parameters were kept constant during testing. Results thus obtained could also be used for soil samples collected from other locations, having similar physical properties.

Ultrasonic testing was also done on variety of soil samples collected from varied locations. All these samples had varied in-situ density, moisture content and void ratio. Safe bearing capacity of these samples was also different. Safe bearing capacity reported in the investigation corresponds to soil samples at 2m depth from ground surface. It is determined for strip footing under local shear failure conditions. Results of ultrasonic transmission velocity through these samples was combined with certain soil properties to get a parameter. This parameter indicates a definite pattern of variation with respect to bearing capacity of soil samples. This correlation can be used to approximate bearing capacity of soil samples by knowing its certain in-situ properties as well as ultrasonic transmission velocity.

2. LITERATURE REVIEW

2.1 SHEAR STRENGTH OF SOILS

Soil for a geotechnical engineer is the top thickness of the earth's crust. This is accessible and feasible for practical utilization in geotechnical enginnering. In the most general sense, soil refers to the uncemented particulate material. However shale and sensitive soils have some degree of cementation. Soil is particulate system in the sense that the discrete particles are not bonded together as the crystal in a metal. At the same time, the individual molecules in a particle are not free to move as the molecules in a fluid. Particulate materials are composed of solid particles within the gaseous or liquid phase. They exhibit dilatency or contractancy and are sensitive to hydrostatic stresses. Due to these three phases present in a soil system, soil behaviour is quite complex. Consequently engineering behaviour of soils changes under drained and undrained conditions. Similarly engineering behaviour of saturated soil is different than engineering behaviour of partially saturated soil (Nagaraj et al, 1990).

Apart from three phases which are present in a soil system, there are certain other parameters also which significantly affect engineering behaviour of soils. Particle size composition, pore water content, salt concentration of pore water, type of mineral present and soil structure are some of the significant influencing parameters (Venkatapparao and Moondra, 1976)

Shear strength of soils is one of the significant engineering properties. Engineers are interested in knowing about it because it helps in engineering design. Important practical problems like stability, bearing capacity, in-situ stress etc. depend on soil strength. At micro-level shear strength arises due to interparticle interactions between soil particles. These interactions can either be attractive or repulsive.

Shear strength of a soil is conventionally defined in terms of the two components, cohesion c, and angle of internal friction ϕ. Variety of factors cause chemical cementation between soil particles (Nagaraj et al, 1991a). This results in cohesion. Similarly friction arises due to actual mineral contact between soil particles. These two parameters of cohesion and angle of internal friction are not constant for a given soil system. They are influenced by several factors.

Attempts have been made in the past to explain soil strength and load deformation behaviour of soils by a number of investigators and most of them are emperical studies. Mohr-Coulomb relation is the most widely used relationship. It is empirical relation to explain strength behaviour of soil system (Mitchell, 1976). According to this relation:

$$\tau_{ff} = c + \sigma_{ff} \tan\phi \qquad (2.1)$$

$$\tau'_{ff} = c' + \sigma'_{ff} \tan\phi' \qquad (2.2)$$

Soil system when subjected to external loading fails along a failure plane. τ_{ff} and σ_{ff} indicate shear and normal stresses acting on failure plane at the time of failure. **c** is cohesion intercept and ϕ is angle of internal friction. From equation (2.1) it is clear that there is a critical combination of τ_{ff} and σ_{ff} at failure. τ_{ff} or σ_{ff} alone do not cause failure. σ_{ff} in equation (2.1) is total normal stress at failure on failure plane, whereas σ'_{ff} in equation (2.2) is effective normal stress at failure on failure plane. τ_{ff} and τ'_{ff} are similarly total and effective failure shear stresses respectively. Effective stresses are obtained by subtracting pore water pressure at failure from total stresses. Again, **c** and ϕ are total stress parameters, whereas **c'** and ϕ' are effective stress parameters

Experience and experimental observations however indicate that Mohr-Coulomb relation as explained above is not able to completely explain the soil strength behaviour. This is because the strength of soil system depends on other factors as well. The complete functional relationship for shear strength takes the form given below (Mitchell, 1976):

$$\text{Shear strength} = F(e, \phi, C, \sigma', c', H, T, \varepsilon, \varepsilon', S) \tag{2.3}$$

This relationship is applicable to a general type of in-situ soil, where pore fluid is also present. In this relationship, **e** represents void ratio, **C** represents composition, **H** denotes stress history, **T** represents temperature, ε represents strain, ε' represents strain rate and **S** represents structure. All the parameters in the above equation are not necessarily independent. By conducting specific type of test certain parameters can be controlled to some extent. Cohesion and angle of internal friction values thus obtained are specific to that type of test even though soil system, drainage condition, rate of loading, range of confining pressure and stress history are kept constant. Specific tests in which certain parameters can be controlled to some extent are direct shear, triaxial compression, triaxial extension, simple shear etc. As a result, a variety of values for angle of internal friction and cohesion may be obtained.

From the practical viewpoint, it is always desirable to adopt comparatively simple laboratory tests to measure the properties needed for soil strength analysis (Khosla and Wu, 1976). In view of this, in the present study direct shear testing method was adopted to test soil samples having different particle size composition, water content and pore water salt concentration. The direct shear test is easy to perform and therefore a number of soil samples can be quickly tested (Das and Prakashi, 1990). This helps to establish a relationship showing the effect of particle size composition, water content, pore water salt concentration etc. on shear strength of the sample.

Depending on the degree of saturation and the degree of cementation, soil system is classified into three broad categories:

(a)Saturated uncemented.

(b)Saturated cemented.

(c)Partly saturated.

These three soil systems have very different shear strength characteristics.

Saturated uncemented soils are two phase system. All the void spaces are filled up with water. Soil particles present in such system are coarse grained. Chemical cementation between soil particles is absent. Hence shear strength of soil arises due to actual physical contact between soil particles. Interactions arising due to actual physical contact between soil particles is not very sensitive to pore fluid characteristics. Hence although all the voids are filled up with water, pore fluid characteristics have a little effect on shear strength in such type of soils. Shear strength is a function of applied normal stress (Nagaraj et al, 1991a).

In cemented saturated soils, often environmental factors cause chemical bonding between soil particles. This is an additional complexity in soils. These cementation bonds markedly influence the behaviour of such soils. In general, cemented soils exhibit very low compressibility and high strength. This is nearly rigid non particulate response within the yield point. Beyond yield point it shows high compressibility and higher shear strength compared to that of uncemented soils. It is not easy to explain the behaviour of such soils because it has not been possible to uniquely relate the state of cemented soils to the effective stresses. In other words, the effective stress responsible for shear strength is not clearly understood. It changes from soil to soil (Nagaraj et al, 1991b). Cementation bond in these soils is the major cause of shear strength. However this bond is quite sensitive to pore fluid characteristics under saturated conditions. Hence shear strength is also sensitive to pore fluid characteristics.

Most of the natural soils above water table and invariably all the compacted soils in their initial condition are in a state of partial saturation. Partly saturated soils are found in places where there is annual excess of evaporation potential over saturation. The mechanics of shear strength behaviour of partially saturated soils is of major concern to geotechnical engineers due to their practical use. There has been extensive research to understand these behaviours. In general, attempt has been made to arrive at the effective stresses which control the shear strength behavior. However, still unanimity in understanding has not been achieved as to what constitutes the effective stresses in these soils (Nagaraj et al, 1991c). These kind of soils are a mixture of cemented and uncemented soils. Moreover it is a three phase system under partially saturated state. Hence shear strength behaviour changes from soil to soil. Present study has been conducted on partially saturated soils.

Shear strength of partially saturated soils also depends on the index properties of soil. These include particle size characteristics, atterberg limits, specific gravity and maximum & minimum dry density (Lee and Singh, 1971; Wesley, 1977). Soils in general are classified as gravel, sand, silt and clay. Soil of particle size greater than 2mm are taken as gravel. If the particle size is in between 0.06mm and 2mm, it is taken as sand. Soils of particle size in between 0.002mm and 0.06mm is taken as silt. Soils having particles smaller than 0.002mm are taken as clay (IS 1498:1970).

In the case of large particles, e.g. in the sand range, the compressibility and shear strength of soil is controlled by the rigidity and strength of individual particles, their packing and coefficient of friction between the particles (Olson, 1974). For fine grained soils, double layer interaction between soil particles plays an important role in modifying its shear strength (Olson, 1963).

FACTORS INFLUENCING SHEAR STRENGTH OF SOILS

At the particle level, interparticle interactions between soil particles are responsible for shear strength. Forces arising due to actual particle contact as well as the interparticle attraction and repulsion play important role in deciding the level of shear strength. Nature of interparticle forces, spacing of soil particles, orientation of particles, applied stresses and characteristics of soil water system are some of the important factors that affect shear strength of the soil.

For granular soils, frictional resistance is the main cause of shear strength. It arises due to actual physical contact between the particles (Nagaraj, 1995). In clays also localized contacts at the micro-level contribute to shear strength. However in view of smooth particle surface, interparticle forces of interaction become the predominant cause for shear strength of clay (Calladine, 1971). If the physical contact between soil particles increases, it will increase the frictional strength of soil. Interparticle forces are function of particle displacement. An increase in interparticle attraction increases shear strength, whereas an increase in interparticle repulsion decreases shear strength.

Spacing of particles also has influence on shear strength. At the closer spacing all the interacting forces are greater. If two soil particles are placed some distance apart, the effect of a reduction in spacing could result in either an increase in attraction or may cause repulsion. Shear strength would enhance, if the net result is increase in attraction. For sandy soils, increasing the externally applied intergranular pressure reduces the particle spacing. This increases the shear strength. Hence denser the sand and closer the particle spacing; greater would be the shear strength (Anandrajah et al, 1995). For clays, interparticle attraction as well as repulsion are the dominant interactions. Attraction is due to van der Waals forces whereas repulsion is due to double layer interaction. At closer spacing however, attractive forces are dominant. Consequently shear strength of the clay increases (Nagaraj et al, 1990).

For a given particle spacing, at micro-level, the more nearly parallel the adjacent particles are, the weaker the soil would be. Net attractive force for a parallel orientation of particles is lower compared to the orientation in which particles are inclined. Consequently shear stress required to slide particles relative to each other is lower when particles are oriented nearly parallel. The orientation in which soil particles are parallel or nearly parallel takes place when soil particles are in dispersed state. If soil particles are flocculated with salt type flocculation then also this type of orientation is favored. Under non-salt flocculation orientation edge of one particle comes closer to face of another particle. Consequently edge to face attraction becomes significant. Hence shear strength increases.

A change in externally applied intergranular pressure results in a change in spacing and/or orientation. This alters the electrical forces between particles. Consequently shear strength changes. For particles in contact with each other, a change in the intergranular pressure affects the contact pressure between particles and thus the shear strength. Externally applied stresses also induce stress in pore fluid. The stress in the pore fluid is not a primary variable of shear strength. However it is a factor which helps in determining the intergranular stresses between the particles. In soils under fully saturated state or under high degree of saturation, negative water tension due to stress in water causes intergranular pressure of equal magnitude. As a result attractive tendencies are increased. The stress in pore fluid mostly comes from hydrostatic pressure.

Variables as well as characteristics of soil water system also have effect on interparticle forces between particles. Any expansion of the double layer increases the interparticle repulsive force between adjacent particles. This will cause reduction in shear strength. The attractive forces are not greatly affected due to soil-water system characteristics. Percolating waters can however deposit cementing agents such as carbonates and iron oxides. This will enhance attractive tendencies (Lambe, 1958).

2.2 REINFORCED SOIL

Geotechnical engineers are interested in effective utilization of poorest of poor soils. Consequently, their efforts are directed to develop technically viable and economically feasible methods to improve the mechanical properties of soil. This will suit the requirements of engineering structures. Various soil improvement methods have been developed to improve soil properties. Among recent developments is the introduction of reinforcements in the form of sheets, strips, discrete fibres etc. Reinforcements in the form of sheets and strips does strengthening of soil at "macro-scale". Similarly reinforcements in the form of discrete fibres does strengthening at the "meso-scale". When randomly distributed discrete fibres are added to the soil, the soil-fibre mix is termed as "ply soil" (Ranjan and Charan, 1998).

The use of reinforced soil as a civil engineering material has gained widespread application. Thus earthwalls, embankments, abutments, etc. are routinely being constructed with this material. Reinforcement of ground for slopes, foundation beds, pavements, etc. is another application of reinforcement principle. There have been extensive and varied application of reinforced soil. However design procedures are still empirical or semi-empirical due to inadequate understanding of the behaviour of reinforced soil.

The reinforcing elements vary from almost inextensible steel or galvanised iron strips, through moderately stiff geogrids to highly extensible geotextiles. The improvement in the overall behaviour of the composite is achieved through the interaction between soil and the reinforcement. There is difference in the moduli of elasticity between soil and reinforcement and this results in differential deformation between soil and the reinforcement. Consequently shear stresses are mobilized at the interfaces between the soil and the reinforcement. The effect of the shear stress is to generate tensile stress in the reinforcement (Madhav, 1992).

The fibre reinforcement falls under the category of ideally extensible inclusions. The deformation under load and the consequent failure mechanism of fibre reinforced soil is different from that of soil reinforced with inextensible reinforcements. In case of extensible fibre reinforcement, the maximum force in the fibre is controlled by the deformation in the soil. This type of fibre reinforced soil exhibits an enhanced load carrying capacity. This capacity is higher than the peak shear strength of the soil alone. Furthermore, it also exhibits greater residual strength and ductility as compared to soil alone or soil reinforced with inextensible reinforcements (Ranjan et al, 1994).

Even though the strength properties of materials currently being used for civil engineering applications are reasonably good, their stiffness, in terms of their modulus of elasticity, is low. For engineering applications, both strength and stiffness are important. Therefore reinforcement becomes necessary. The most commonly used reinforcements are in particulate or fibre form. In fibre composites, fibres have high strength and high modulus. They are embedded in the low modulus matrix (Andersland and Al-Khafaji, 1981) such as soil. The fibrous reinforcement is oriented in such a way, so as to provide maximum strength and stiffness in the desired direction.

A unidirectional fibre composite is highly anisotropic. Whereas, stiffness and strength along fibre direction are very high, stiffnesses and strength across the fibre direction are much lower. Consequently properties of composite depend on the orientation of fibres (Cox, 1952).

The low values of stiffness and strength in the transverse direction provide the motivation to opt for laminate construction. Laminate consists of thin unidirectional layer with different reinforcement directions. The layer thickness, fibre directions, choice of fibres, etc. are selected to minimize the weight and price for achieving a particular strength and stiffness (Hashin, 1983).

Matrix is effective in transferring stresses from one fibre to another fibre. As a result, applied load is evenly distributed. Furthermore, as fibres are much stronger than the matrix, usually the cracks initiate in the matrix (Aryciw and Irsyam, 1991). The main factors controlling the theoretical performance of the composite material are the physical properties of the fibres and the matrix. The strength of the bond between the fibres and matrix also plays a very important role in deciding the overall strength of the composite. Important properties of fibres which are used as reinforcing agents in materials of civil engineering importance are listed in table 2.1.

TABLE 2.1 Properties of Selected Commercial Reinforcing Fibres (Schwartz, 1997)

Fibre	Typical Diameter (μm)[a]	Specific Gravity	Tensile Modulus (GPa)	Tensile Strength (GPa)	Strain to Failure (%)	Coefficient of Thermal Expansion ($\cdot 10^{-6}/^{0}c$)[b]	Poisson's Ratio
Glass							
E-glass	10	2.54	72.4	3.45	4.8	5	0.2
S-glass	10	2.49	86.9	4.30	5.0	2.9	0.22
PAN carbon							
T-300[c]	7	1.76	231	3.65	1.4	-0.6, 7-12	0.2
AS-1[d]	8	1.80	228	3.10	1.32		
AS-4[d]	7	1.80	248	4.07	1.65		
T-40[c]	5.1	1.81	290	5.65	1.8	-0.75	
IM-7[d]	5	1.78	301	5.31	1.81		
HMS-4[d]	8	1.80	345	2.48	0.7		
GY-70[e]	8.4	1.96	483	1.52	0.38		
Pitch carbon							
P-55[c]	10	2.0	380	1.90	0.5	-1.3	
P-100[c]	10	2.15	758	2.41	0.32	-1.45	
Aramid							
Kevlar-49[f]	11.9	1.45	131	3.62	2.8	-2, 59	0.35
Kevlar-149[f]		1.47	179	3.45	1.9		
Technora[g]		1.39	70	3.0	4.4	-6	
Extended-chain polyethylene							
Spectra-900	38	0.97	117	2.59	3.5		
Spectra-1000	27	0.97	172	3.0	2.7		
Boron	140	2.7	393	3.1	0.79	5	0.2
SiC							
Mono-filament[h]	140	3.08	400	3.44	0.86	1.5	
Nicalon (multi-filament)[i]	14.5	2.55	196	2.75	1.4		
Al_2O_3							
Fibre-FP[f]	20	3.95	379	1.90	0.4	8.3	
Al_2O_3-SiO_2[j]							
Fibrefrax (discon-tenuous)	2-12	2.73	103	1.03-1.72			

[a] 1 μm = 0.0000393 in.
[b] 1 m/m per ^{0}C = 0.556 in./in. per ^{0}F.
[c] Amoco.
[d] Hercules.
[e] BASF.
[f] Dupont.
[g] Teijin.
[h] Allied-Signal.
[i] Nippon Carbon.
[j] Carborundum.

Fibres elongation at break are two or three orders of magnitude greater than the strain at failure of the matrix. Consequently, the matrix fails long before the fibre strength is approached. Modulus of elasticity of fibres is many times higher than that of soil matrix. However, if the fibre volume fraction is low in the composite, modulus of elasticity of the composite will not be greatly different from that of the matrix. By placing flexible reinforcements in the soil, shear strength of soil may be enhanced. In addition, they also provide protection from erosion and mechanical damage (Patel and Kulkarni, 1990).

Natural as well as synthetic fibres have been used as reinforcing agents in the civil engineering applications. There is interaction between the soil and reinforcement due to soil reinforcement surface friction. Resultant interaction transmits the stresses in soil mass to the reinforcement resulting in improved engineering properties (Mandal, 1987). However synthetic fibres are preferred over natural fibres as reinforcing agents in the soil matrix. Synthetic fibres are more efficient, although total cost involved is higher compared to natural fibres.

The mode of placement of fibres in soil mass is also an important aspect. The fibres may be placed along certain preferred direction. They can even be randomly distributed in soil mass. In oriented fibre reinforced soil, increase in shear strength of reinforced soil depends on concentration, modulus and initial orientation of fibres. Confining stress also plays an important role. Strength increase is directly proportional to weight fraction of fibres. Fibres must be long and rough enough to avoid pull out under the confining stress.

The inclusion of discrete fibres results in significant increase in shear strength of soil when fibres are randomly distributed. Randomly distributed fibres can be mixed with ease, it maintains strength isotropy and there is no specific plane of weakness. They are effectively used as ground improvement techniques, with respect to embankment, subgrade etc. These randomly distributed fibres interlock soil particles or soil particle groups in a unitary coherent matrix. Essentially discrete fibres are simply added and mixed with the soil, much the same way as cement, lime or other additives are added (Maher and Gray, 1990).

The length and volume fraction of fibres in the soil matrix also play an important role in modifying strength related matrix properties. Strength of the composite generally increases with increase in length of fibre filaments. Thus largest strength and strain capacities can be achieved in short term by high volumes of the long fibres (Gray and Al- Refeai, 1986). Thus length, volume fraction and orientation of the fibres in the soil matrix characterize the soil properties. Even in metallic composites, modulus of elasticity of the composite increases with increasing fibre volume fraction (Whittaker and Patten, 1983).

Zhao and Michalowski (1995), have suggested two methods for the analysis of reinforced soil structures: the structural approach and the continuum approach. In structural approach, soil and reinforcements are considered as two seperate structural constituents. Soil reinforced with unidirectionally placed bars, strips, geosynthetic sheets and geogrids are best described by this structural approach.

In continuum approach, macroscopic failure criterian for the reinforced soil is used. This approach finds wider application when continuous filaments or fibres are used as reinforcing agents. This approach assumes that in composites, the size of fibres or filaments relative to the grain size of the soil matrix plays an important role in determining the type of interaction. If the diameter of the fibre is an order of magnitude smaller than the grain size, the flexible fibres may be accommodated in a three dimensional grain assembly entirely by the pore spaces even if the fiber aspect ratio (fibre length to diameter ratio) is large. In such a case little or no load can be transferred to the fibres since the fibres will slip in the process of matrix deformation. However fibres become effective when soil grain size becomes small compared to fibre diameter.

An effect known as belt friction effect refers to a condition where the force on the filaments is induced due to deformation of matrix. A tensile force thus induced does not relax due to slippage because of the serpentine deposition of the filaments in the matrix. However during a deformation process, only a portion of the fibre filaments is subjected to extension (stretching), whereas the remaining part generally kinks because of the inability of fibre filaments to carry compressive load. Also portions of fibre filaments at transition from the extension to compression regime do slip. It is the portion of fibre filaments subjected to extension that contributes foremost to the composite strength (Michalowski, 1997).

In the present investigation diameter of glass fibres was in the range of 15-20 μm. Only silty clay soil particles retained on pan will have some soil grains smaller than that size. Most of the soil particles in the soil matrix have particle size larger than that value. Hence the effect of fibre addition, in enhancing soil matrix strength, is largely due to belt friction effect explained above. Effect due to frictional interaction between soil matrix and fibres is comparatively smaller.

Glass fibres have been used extensively as reinforcing agents in the soil matrix. The physical performance of the fibres in this matrix as well as the performance of the composite is critically dependent on the chemical and physical microstructure of the soil. The interfacial region is important not only on the external surface of the fibre bundle but also within the fibre bundle.

As reinforcement for soils, glass fibres are generally used in short lengths. Sometimes glass fibres are also used as woven mats or as continuous rovings. Glass fibres have a number of important properties which make them potentially attractive for engineering applications. Glass is made of stable oxides (thus free of oxidation problem), has high strength and can withstand thermal shocks and vibration. It has low stretchability and is largely resistant to chemical action (Gupta et al, 1989).

2.3 ULTRASONIC TESTING

Ultrasonics, which is a branch of acoustics, deals with vibratory waves at frequencies above those within the hearing range of the average person. These frequencies are above 20 kHz. Ultrasonic waves are stress waves and for this reason they can exist only within mass media. They are transmitted from one mass to another by direct and intimate contact between the masses. In this respect, they differ from light and other forms of electromagnetic radiation which travel freely through vacuum. However, these two forms of energy obey similar laws of propagation. Ultrasonic waves also are termed elastic waves. It is the elastic property of the medium which is responsible for the sustained vibrations required for ultrasonic wave propagation.

Application of ultrasonics in general is divided into two broad categories: Low intensity and high intensity. Low intensity applications are those wherein the primary purpose is to transmit energy through a medium. Here, the objective may be to learn something about the medium or to pass information through the medium. The objective is never to change the state of the medium. Typical low intensity applications are nondestructive testing of materials or devices, measurement of the elastic properties of materials and medical diagnosis. Marine applications such as depth sounding, echo ranging, communication and submarine detection may also be included in this category.

High intensity applications are those wherein the purpose is to produce an effect on a medium, or its contents, through which the wave propagates. Typical high intensity applications of ultrasonics are medical therapy, atomization of liquids, machining of brittle materials, cleaning and welding of plastics as well as metals. Homogenization or mixing of materials is also included in this category.

An ultrasonic transmitter is an instrument designed to generate the disturbance from which the ultrasonic energy emanates. Consequently, any device capable of generating ultrasound is an ultrasonic transmitter. The ultrasonic transmitters most frequently used are either piezoelectric devices or magnetostrictive devices. Piezoelectric transducers may be used throughout the ultrasonic range of frequencies. However magnetostrictive transducers are useful for generating high intensity ultrasonic energy having frequencies approximately 50 kHz. The device that detects the ultrasound is called an ultrasonic receiver. The transducers most often used as receivers of ultrasonic energy are the piezoelectric types. Magnetostrictive devices are used occasionally (Ensminger, 1973).

Ultrasonic waves are classified on the basis of the mode of vibration of the particle of the medium with respect to the direction of propagation of the waves. They are longitudinal, transverse and surface waves.

Longitudinal wave is the most common form of ultrasonic wave transmission. In this type of ultrasonic wave, alternate compression and rarefaction zones are produced by the vibration of the particles parallel to the direction of the propagation of the wave. Particle oscillations are in the longitudinal direction in longitudinal waves. Because of its easy generation and reception, this type of ultrasonic wave is most widely used in ultrasonic testing. This type of wave can propagate in solids, liquids and gases.

Transverse waves are also called shear waves. In this type of ultrasonic wave, the direction of particle displacement is at right angles to the direction of wave propagation. For such wave to travel through a material, each particle of the material should be strongly bound to its neighbors. In liquids and gases, adjacent particles are not strongly bound to each other. Consequently, vibration of one particle doesn't cause sufficient vibration of adjacent particles. As a result transverse waves are not able to propagate in liquids and gases. For all practical purposes, transverse waves can propagate only in solids. Usually shear wave velocity is approximately one half of longitudinal wave velocity in the same material.

Surface waves can travel only along a surface bounded on one side by strong elastic forces of the solid and on the other by nearly non-existent elastic forces between gas molecules. These waves have a velocity approximately 90% that of shear waves in the same material. They can propagate only in a region no thicker than about one wavelength beneath the surface of material. In surface waves, particle vibration follows an elliptical orbit. Major axis of ellipse is perpendicular to the surface along which ultrasonic waves are propagating. These waves are not as frequently used as longitudinal or transverse waves for ultrasonic testing. However, these waves find some application in locating flaws along the material surface (Baldev Raj et al, 1997).

As ultrasonic wave propagates through a medium, its amplitude decreases or it attenuates. In general, an increase in the attenuation of ultrasound in a material is an indication of possible degradation or loss of strength of material. In practice several factors contribute to wave attenuation. In a material with very coarse grains compared to the wavelength of incoming ultrasonic pulse, attenuation primarily takes place due to geometric division of the ultrasonic pulse. On an oblique grain boundary, the pulse is split into various reflected and transmitted pulse types. This process repeats itself for each pulse at the next grain boundary. Thus, the original ultrasonic pulse is constantly divided into partial pulses as it propagates through the medium. Consequently attenuation of pulse takes place.

If the grain size in a material is smaller than the wavelength of incoming ultrasonic pulse, attenuation of pulse takes place primarily due to scattering. If the grain size is 1/1000th to 1/100th of the wavelength, scattering for all practical purposes is negligible. Consequently attenuation effect is not present. However attenuation effect increases rapidly as the grain size increases. The effect of attenuation is felt at grain sizes from $1/10^{th}$ to full value of the wavelength. Effect of attenuation is of such an extent that it makes testing impossible. Consequently in such materials frequency of ultrasonic pulse should be such that the corresponding wavelength of incoming ultrasonic pulse is at least 10 times more than the average grain size.

Absorption is another major cause of attenuation. Absorption is a direct conversion of ultrasonic energy into heat. Absorption arises due to braking effect of the oscillations of the particles. Consequently a rapid oscillation loses more energy than a slow oscillation. As a result absorption usually increases with frequency (Krautkramer and Krautkramer, 1969).

In pulse-echo method, ultrasonic pulses are generated with the help of piezoelectric element in the probe head. These pulses are then transmitted into the material under test. Consequently transmitters act as emitters of ultrasonic pulses. A defect or discontinuity within the material along pulse path causes reflection of ultrasonic pulse back to the transducer. Hence same transducer also acts as receiver of reflected ultrasonic pulse. Reflected ultrasonic pulse (i.e. echo) is converted by the transducer into electrical signal. The echo amplitude and time of travel through the material is indicated on the screen of flaw detector. By knowing ultrasonic pulse velocity through the material and the time of travel, the distance of defect or discontinuity from the test surface can be evaluated. Ultrasonic pulse is also reflected from the opposite face of the material. This can be used to determine material thickness if pulse velocity and travel time is known.

Pulse-echo method can also be used with two transducer arrangement. In such arrangement, one transducer transmits the ultrasonic pulse. The echoes reflected from the backwall, defects and discontinuities are received by another transducer. This transducer is called receiver. Both the transducers are placed on the same side of the test specimen.

In through transmission method two separate transducers are used. Testing requires access to both sides of the test specimen. One unit of transducer acts as transmitter and the other unit acts as receiver. The beam from the transmitter travels through the material. It is received by receiver placed on the opposite surface. Presence of defect or discontinuity in the path of the beam causes reduction of the ultrasonic energy reaching the receiving transducer. However exact size and location of defect cannot be known using this method.

To help efficient transfer of ultrasonic energy from transmitting transducer to test specimen, an intermediate medium is generally used. This is generally in the form of gel. This intermediate medium is called acoustic couplant. Couplant also helps in making perfect contact. Heavy loss of acoustic energy takes place if imperfect acoustic contact is there. The couplant should be able to wet both the test surface and transducer face. For rough surfaces, more viscous couplant should be used. Couplant should not be corrosive or toxic. It should be homogeneous, free from air bubbles and solid particles (Bindal, 1999).

Propagation and attenuation of ultrasonic pulses through the specimen is dependent on the specimen's micro-structure. Quality of specimen is also dependent upon its micro-structure. Consequently propagation and attenuation of ultrasonic pulses through the specimen can be used to assess quality of the specimen. These concepts have also been used in materials of civil engineering importance. Enough experimental research has not been conducted in the area of ultrasonic testing of soils. It has been reported that longitudinal and shear wave velocities in soils depend on its minerology, porosity, fluid content and degree of consolidation (Blangy et al, 1993) As far as use of ultrasonic testing of concrete is concerned, a number of authors have studied ultrasonic characteristics of concrete (Carino et al, 1986; Elvery, 1971; Ismail et al, 1996; Knab et al, 1983; Malhotra and Carette, 1980; Narayanan and Ramaswamy, 1976; Niyogi and Mukhopadhyay, 1977; Wu et al, 1996).

2.4 BEARING CAPACITY OF SOILS

Information about bearing capacity of soils is used in foundation design. Foundation is the lowest part of the structure. This part of the structure is in direct contact with surrounding soil of the ground. Loads from the upper part of structure (superstructure) is transmitted to the foundations. Maximum amount of load which can be transmitted to the foundation from the superstructure depends upon the load bearing capacity of surrounding soil. If the applied load is more than this maximum value, failure of surrounding soil takes place.

Foundations are subdivided as shallow foundations and deep foundations. Shallow foundations are located below ground surface at a shallow depth. Depth of shallow foundation below ground surface is less than or atmost equal to width of foundation. These kind of foundations are used when load due to superstructure are small. Furthermore soil immediately below and in surrounding region of foundation should have sufficient shear strength.

When load due to superstructure is very high (typically due to multistorey buildings), deep foundations are used. These foundations extend to great depth from ground surface. Certain soil deposits have low shear strength close to the ground surface. High strength soil layer is available only at deeper depth. Under these conditions also deep foundations are used. Soil having low shear strength, is associated with low load bearing capacity and vice-versa. As the deeper soils have higher shear strength, they have high load bearing capacity and therefore the concept of deep foundations.

The design of foundations must satisfy certain requirements. Firstly, complete failure of the foundation must be avoided with an adequate margin of safety. Secondly, the total and relative settlements of the foundation must be kept in limits. This limit should be tolerated by the superstructure. The ultimate bearing capacity of a foundation is defined as the maximum load coming from the superstructure that surrounding soil can sustain before failure of soil takes place. Sometimes load-settlement curve of soil doesnot exhibit a peak load corresponding to failure. In such soils bearing capacity is taken as the load at which the curve passes into a steep and fairly straight tangent (Meyerhof, 1951).

The failure of foundation when the bearing capacity of soil is exceeded usually takes place in four stages. The first stage involves a downward movement of soil beneath the foundation. The second stage is described by a localized cracking of soil around the perimeter of foundation. Stage three involves formation of a cone shaped wedge of soil beneath the footing. This forces the soil downward and outward. Finally a failure surface develops.

Several failure mechanisms have been suggested for shallow foundations. These include general, local and punching shear failures. General shear failure of foundation takes place when the underlying soil is dense and compacted. Such soils develop relatively high resistance to foundation penetration under external load. Beyond the critical load, there is sharp decrease in resistance to foundation penetration. General shear failure occurs when sufficiently large region of soil beneath the foundation is stressed to its yield condition. Consequently flow of soil away from the foundation from below the foundation takes place. Region of soil which is stressed to yield condition extends to 2.5 times the width of foundation on either side of foundation in the horizontal direction. Similarly in vertical direction this region extends two times the width of foundation (Saran and Agarwal, 1974).

Local shear failure of foundation takes place if underlying soil is of intermediate density. Resistance to foundation penetration under external loading is small compared to general shear failure condition. Under local shear failure, only localized region of soil close to foundation is stressed to its yield condition. Punching shear failure of foundation takes place if underlying soil is in loose state. Such soils have very little resistance to foundation penetration under external load (Larkin, 1968). Consequently depending on the density of the soil, general, local or punching mode of shear failure takes place for shallow foundations.

Bearing capacity of foundations depend on the mechanical properties of the soil. Important mechanical properties of soil affecting its bearing capacity are soil density, soil shear strength and soil deformation characteristics. Water conditions below the ground surface and physical characteristics of the foundation also affect bearing capacity of foundations. Important physical characteristics of the foundation affecting its bearing capacity are its size, shape, roughness and depth of embedment below ground surface (Meyerhof, 1955).

Shallow foundations are subdivided based on the geometrical shape of their base. Geometrical shape of the base of shallow foundation could be in the form of continuous strip, rectangle, square or circle. Shallow foundations in the form of continuous strip are most widely used. Following equations are used to determine ultimate bearing capacity of shallow foundations if geometrical shape of foundation base is continuous strip (IS : 6403 - 1971):

$$q_d = cN_c + qN_q + 0.5B\gamma N_\gamma \qquad \text{(Under general shear failure)} \qquad (2.4)$$

$$q_d = \frac{2}{3}cN'_c + qN'_q + 0.5B\gamma N'_\gamma \qquad \text{(Under local shear failure)} \qquad (2.5)$$

Where,

q_d = Ultimate bearing capacity in Kg/cm^2.

c = Cohesion of the soil in Kg/cm^2.

q = Effective surcharge at the base level of the footing in Kg/cm^2 ($=\gamma D_f$).

γ = Total unit weight of soil in Kg/cm^3.

D_f = Depth of embedment of footing from ground level in cm.

B = Width of strip footing in cm.

N_c, N_q and N_γ in equation (2.4) are Terzaghi's bearing capacity factors of the soil for general shear failure. They are dependent on the angle of internal friction of soil and can be obtained from table 2.2 given below. N'_c, N'_q and N'_γ in equation (2.5) are Terzaghi's bearing capacity factors of the soil for local shear failure. To determine N'_c, N'_q and N'_γ in equation (2.5), modified angle of internal friction ϕ'_{mod} is obtained using:

$$\phi'_{mod} = \tan^{-1} 2/3 \tan\phi \qquad (2.6)$$

Bearing capacity factors corresponding to this modified angle of internal friction is also obtained from table 2.2. These values correspond to N'_c, N'_q and N'_γ.

TABLE 2.2 Bearing Capacity Factors of Terzaghi (Murthy, 1996)

ϕ (deg.)	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	1.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
50	347.5	415.1	1153.0

Safe bearing capacity is determined by dividing ultimate bearing capacity by an appropriate factor of safety. In present investigation a factor of safety of three has been used to determine safe bearing capacity. Safe bearing capacity has been determined using equation (2.5) taking foundation width as 200cm and depth of foundation embedment below ground level also as 200cm. Safe bearing capacity thus obtained has been correlated with ultrasonic pulse velocity through soils. Study has been based on soil samples collected from different locations.

3. EXPERIMENTAL WORK

3.1 SHEAR STRENGTH BEHAVIOUR OF SOILS

SOIL USED

Locally available soil was used as the main experimental material. One of the reason for selecting local soil was to better understand its behaviour in view of major expansion plan being undertaken at BITS Pilani. Buildings are proposed and are under construction for new library complex, additional hostel complex, additional residential accommodation for faculty, new class rooms etc. Experimental work required coarse-grained soil as well as fine-grained soil. Coarse-grained soil was collected from desert stretch located some distance from BITS campus. Soil from this location was predominantly coarse grained. Amount of fine grains in the soil was very small. Soil sample had an in-situ moisture content of 4 to 5%. It was oven dried for 24 hours before using it for experimental work.

Experimental work also required fine grained soil. This soil was available locally close to the BITS campus at a depth of 12 to 15 meters. It was collected in the month of April from a deep ditch excavated at that location. Water table in the area is approximately at a depth of 150 meters and the in-situ water content of the soil was 6%. Liquid limit and plastic limit of sample was 26% and 16% respectively. Soil had an in-situ density of 1.61 gm/cm^3 and specific gravity of 2.63.

Sieve analysis of both the soil samples were made. Particle size distribution obtained after doing the sieve analysis of both the samples have been indicated in Table 3.1.

TABLE 3.1 Particle size distribution of local sand and silty clay

Particle Size	2.36mm	1.18mm	600μm	300μm	150μm	75μm
% finer (sand)	100	100	100	100	73.72	4.87
% finer (silty clay)	100	100	100	100	62.57	24.29

After doing sieve analysis, both the soils were found to be retained on 150μ sieve, 75μ sieve and on pan. Sandy soil retained on 150μ sieve and silty clayey soil retained on 75μ sieve as well as on pan was used for further studies. Coarse grained soil retained on 150μ sieve has been classified as sandy. Similalrly fine grained soil retained on 75μ sieve as well as on pan have been classified as silty clay. This classification is based on dispersion test. Particle size distribution curve of these two soils is also plotted in Figure 3.1.

For dispersion test, a spoonful of oven dried soil sample is allowed to settle in a jar containing water. Time for full settlement of sample in the jar is noted and based on time for settlement of the soil, it is classified in different catagories as shown in Table 3.2. Time taken for coarse grained soil retained on 150μ sieve for full settlement was noted as 35 seconds. Some part of fine grained soil which was retained on 75μ sieve and also on pan, got fully settled in 1 hour and remaining part remained in suspension for more than one hour. Hence coarse grained soil (retained on 150μ sieve) can be classified as sandy and fine grained soil (retained on 75μ sieve and on pan) can be classified as silty clay.

TABLE 3.2 Results of dispersion test (Sehgal, 1984)

Time for full settlement	Soil classification
Within a minute	Sandy
15 minutes to 1 hour	Silty
In suspension for hours	Clayey

Specific combinations of sandy and silty clayey soil samples have been used in the present investigation. Each combination will thus have unique particle size composition. Important soil properties for each combination of sandy and silty clayey soil is given in Table 3.3 given on page 55. These important soil properties include liquid limit, plastic limit, maximum and minimum dry density as well as specific gravity.

PORE WATER USED

Tap water available in the soil mechanics laboratory was used in all the experiments conducted.

TEST PROCEDURE

Direct shear testing was conducted to find out cohesion and angle of internal friction of soil. In the direct shear testing, the soil is forced to fail along a predetermined failure surface. This failure surface is horizontal. Thus testing is conducted on idealized condition. On this failure surface, there are two stresses acting, a normal stress due to applied vertical load P_V and a shearing stress due to applied horizontal load P_H. Thus $\sigma = P_V/A_{area}$ and $\tau = P_H/A_{area}$ at any stage of loading. A_{area} is the cross sectional area of the shear box. At failure, equation (2.1) is followed. At failure $\sigma = \sigma_{ff}$ and $\tau = \tau_{ff}$.

Direct shear testing makes use of shear box. Shear box has separate upper and lower parts. Shear box was assembled by keeping grid plate at the bottom and by putting shear pins to combine the upper and lower part of shear box. Uniform mixing of soil and pore water was done by hand mixing. Afterwards it was placed in the shear box by doing appropriate compaction. Soil sample was compacted in the box about 5mm from the top and then top grid plate and loading block were placed above soil. The serrations of upper and lower grid plate were placed at right angles to the direction of shear.

Shear box assembly was placed on the load frame. Lower part of shear box was against the loading jack. Upper part of shear box was against the horizontal load dial (proving ring). Horizontal load dial reading was set to zero.

Loading yoke was placed on top of loading block. A known vertical (normal) load was kept on the hanger of loading yoke. Shear box pins were removed. Horizontal loading was done at constant rate. Reading of horizontal load dial was taken at failure. At failure horizontal load dial reaches a peak and remains constant at peak for some time. Afterwards dial reading slightly decreases. Peak dial reading is taken as failure reading. Peak horizontal dial reading was multiplied by proving ring calibration to get horizontal load at failure. Proving ring calibration used in the present study is 2.16574 Newtons per smallest division of proving ring.

Same procedure was repeated for four more other normal loads. Failure shear load for each normal load was found. Normal load was divided by cross sectional area of the shear box to obtain normal stress. Corresponding failure shear stress was obtained by dividing horizontal load at failure with cross sectional area of shear box. Cross sectional area of shear box used in present study is 36 cm^2. Straight line of best fit was drawn through data points of normal stress and corresponding failure shear stress. These are plotted on a linear graph paper choosing same scale for normal and failure shear stress on X and Y axis respectively. X axis shows variation of normal stress. Y axis shows variation of failure shear stress. Intercept of this line with failure shear axis gives cohesion of soil. Slope of this line gives the angle of internal friction for the soil sample tested.

Following properties of soil system were changed while conducting the direct shear testing:

(i) Particle size composition

(ii) Water content

(iii) Salt concentration of pore water

Five different particle size composition of soil were used while doing the direct shear testing:

(i) S_{150}=90% by weight, C_{75}=5% by weight, C_p = 5% by weight.

(ii) S_{150} = 70% by weight, C_{75} = 15% by weight, C_p = 15% by weight.

(iii) S_{150} = 50% by weight, C_{75} = 25% by weight, C_p = 25% by weight.

(iv) S_{150} = 30% by weight, C_{75} = 35% by weight, C_p = 35% by weight.

(v) S_{150} = 10% by weight, C_{75} = 45% by weight, C_p = 45% by weight.

S_{150} refers to sand retained on 150μ sieve, **C_{75}** refers to silty clay retained on 75μ sieve and **C_p** refers to silty clay retained on pan. Gradation curve of these soil mix is shown in Figure 3.2 on page 54. Water content of the soil mixture while doing the testing was kept constant at 10%. Salt concentration of pore water was kept zero in all the set of tests. Cohesion and angle of internal friction for each of the above five set of particle size composition were found as mentioned earlier. Particle size composition was the only property of the soil sample which changed for different sets of experiments. Consequently change in cohesion and angle of internal friction values across five sets was correlated with change in particle size composition of the soil mix. Table 3.4 lists results of direct shear testing for the above mentioned particle size composition. Corresponding plots have also been prepared to get cohesion and angle of internal friction values in Figures 3.3, 3.4 and 3.5. Cohesion and angle of internal friction obtained for each composition is shown in Table 3.5. Variation in cohesion and angle of internal friction with particle size composition is also shown in Figures 3.6 and 3.7.

Following procedure was used to observe the effect of water content variation. Five different water content values were used (0%, 6%, 10%, 15% & 20%) while doing the testing.

Soil particle size was kept constant for each set of testing. S_{150} was taken as 50% by weight, C_{75} as 25% by weight and C_p also as 25% by weight for each set of test. Salt concentration of pore water was zero. Thus variations in cohesion and angle of internal friction was due to variation in moisture content of the soil. Table 3.6 lists results of direct shear testing for the above mentioned pore water content when particle size composition of the soil was constant. The variations of failure shear stress with respect to normal stress due to changes in water content are plotted in Figures 3.8, 3.9 and 3.10. Variation in cohesion with pore water content is shown in Figure 3.11. It is shown in a tabular form in Table 3.7.

Effect of pore water salt concentration on strength parameters of soil has been studied by keeping particle size composition as well as pore water concentration constant during the testing. Particle size composition of soil was kept at S_{150} = 50%, C_{75} = 25% and C_p = 25%. Water content of soil was kept at 10%. Four different molar concentrations of salt were used in pore water (1M, 2M, 3M & 4M). Since molecular weight of salt is 58.5 gms., 5.85 gm. of salt was dissolved in 100 ml of water to get 1M salt solution. Similarly 11.7 gm., 17.55 gm. and 23.4 gm of salt were dissolved in 100 ml of water to get 2M, 3M and 4M salt solution respectively. Results of direct shear testing as well as variation of strength parameters with pore water salt concentration are indicated in Tables 3.8 and 3.9 respectively. The graphical representation is shown in Figures 3.12, 3.13, 3.14 and 3.15.

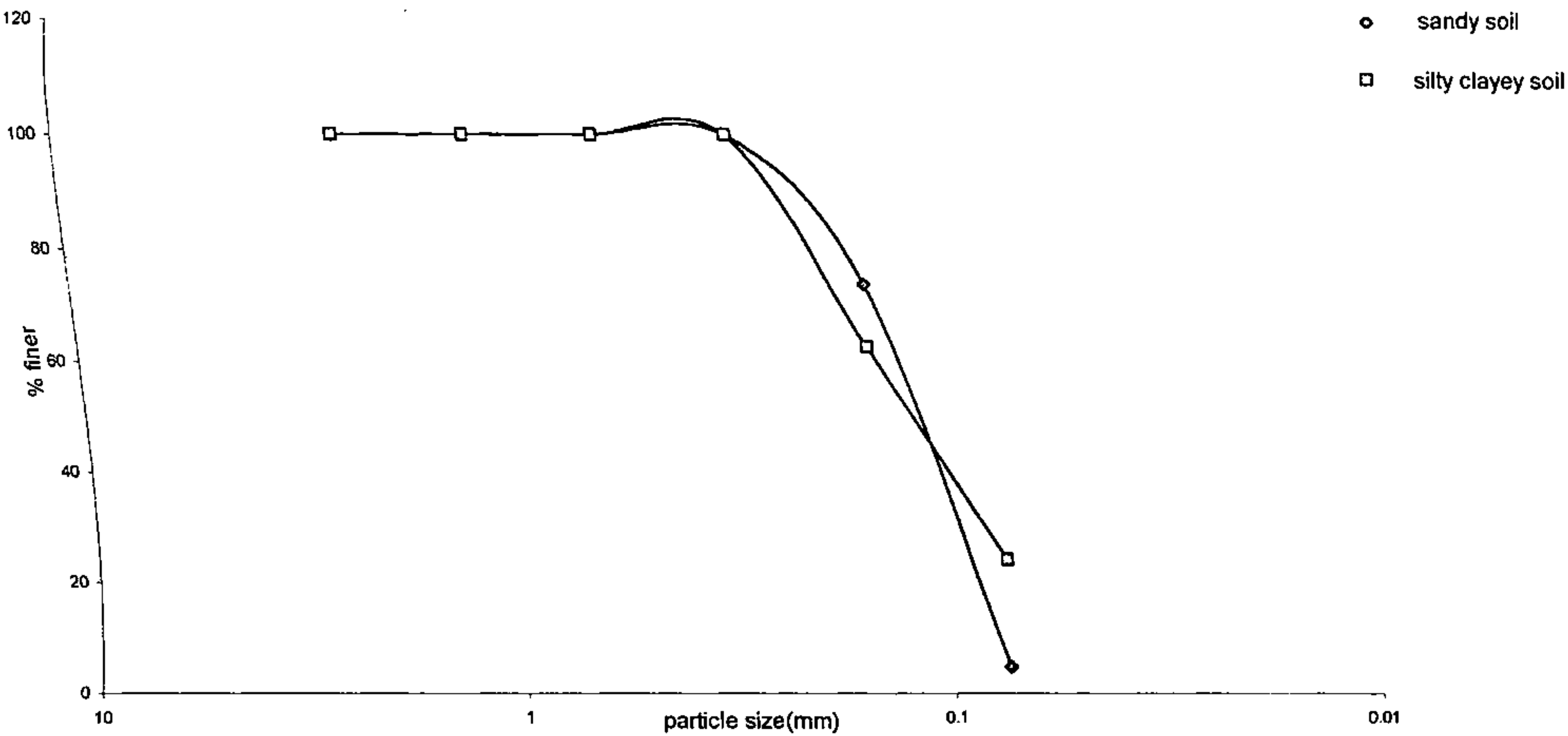

Figure 3.1 : Particle size distribution curve of experimental soil samples

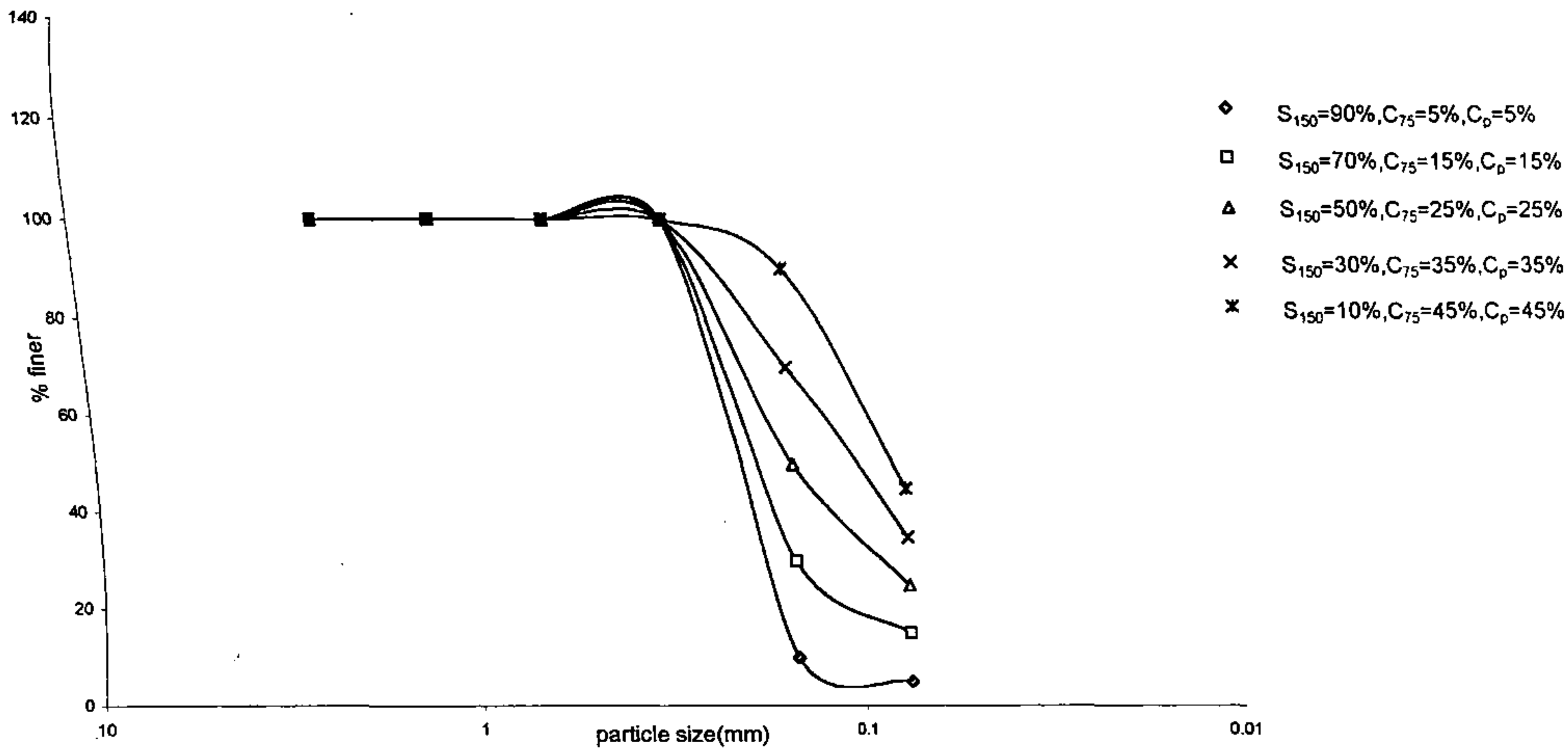

Figure 3.2 : Gradation curve of soil mix used in direct shear testing

TABLE 3.3 Important Properties of Soil Mix

Soil Composition (by weight)			Important Soil Properties				
Sand retained on 150μ sieve	Silty Clay retained on 75μ sieve	Silty Clay retained on pan	Liquid Limit (%)	Plastic Limit (%)	Minimum Dry Density (kg/m³)	Maximum Dry Density (kg/m³)	Specific Gravity
90%	5%	5%	24.5	14.3	1230.6	1572.3	2.37
70%	15%	15%	23.7	13.8	1269.8	1626.46	2.39
50%	25%	25%	24.4	14.1	1253.55	1621.16	2.42
30%	35%	35%	28	18.2	1217.78	1588.78	2.45
10%	45%	45%	29.1	19.4	1159.92	1520.42	2.48
0%	50%	50%	30.6	20.7	1198.93	1572.36	2.5

TABLE 3.4 Direct Shear Test Results on Local Soil Mix
(at 10% water content and varying particle size composition)

LEGEND
S_{150}=Sand retained on 150μ sieve
C_{75}=Silty Clay retained on 75μ sieve
C_p=Silty Clay retained on pan

Normal Stress (x 10^{-3}) MPa	Failure Shear Stress (x 10^{-3}) MPa				
	S_{150}=90% C_{75}=5% C_p=5%	S_{150}=70% C_{75}=15% C_p=15%	S_{150}=50% C_{75}=25% C_p=25%	S_{150}=30% C_{75}=35% C_p=35%	S_{150}=10% C_{75}=45% C_p=45%
9.8	9.8	12.74	13.96	15.92	16.9
19.6	16.9	16.66	15.92	17.39	22.05
29.4	24.5	20.58	18.13	18.86	27.44
39.2	31.6	24.5	20.33	20.58	33.07
49	38.4	28.66	22.78	22.05	38.4

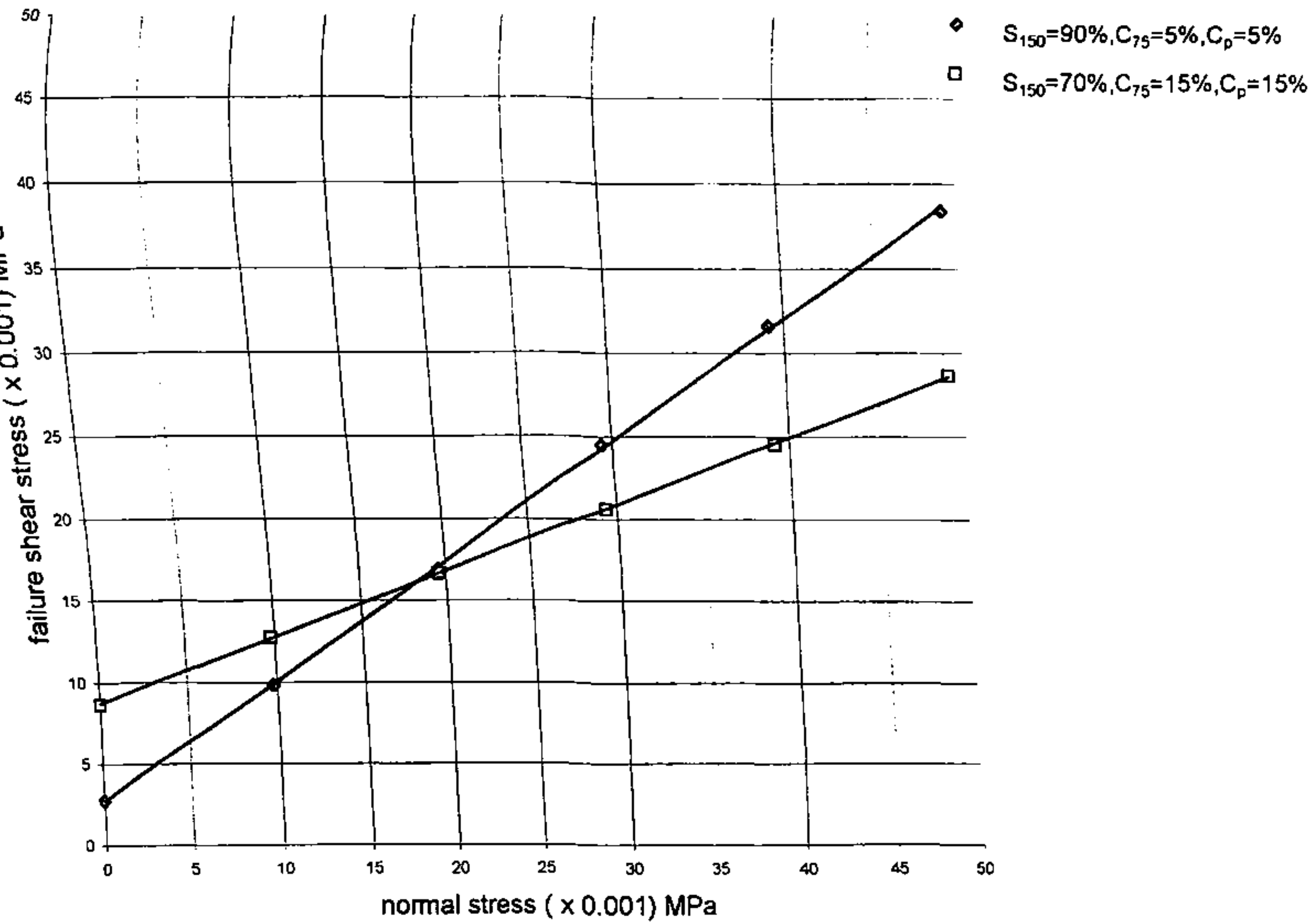

Figure 3.3 : Variation of failure shear stress as a function of normal stress for two soil samples containing 10% by weight of water

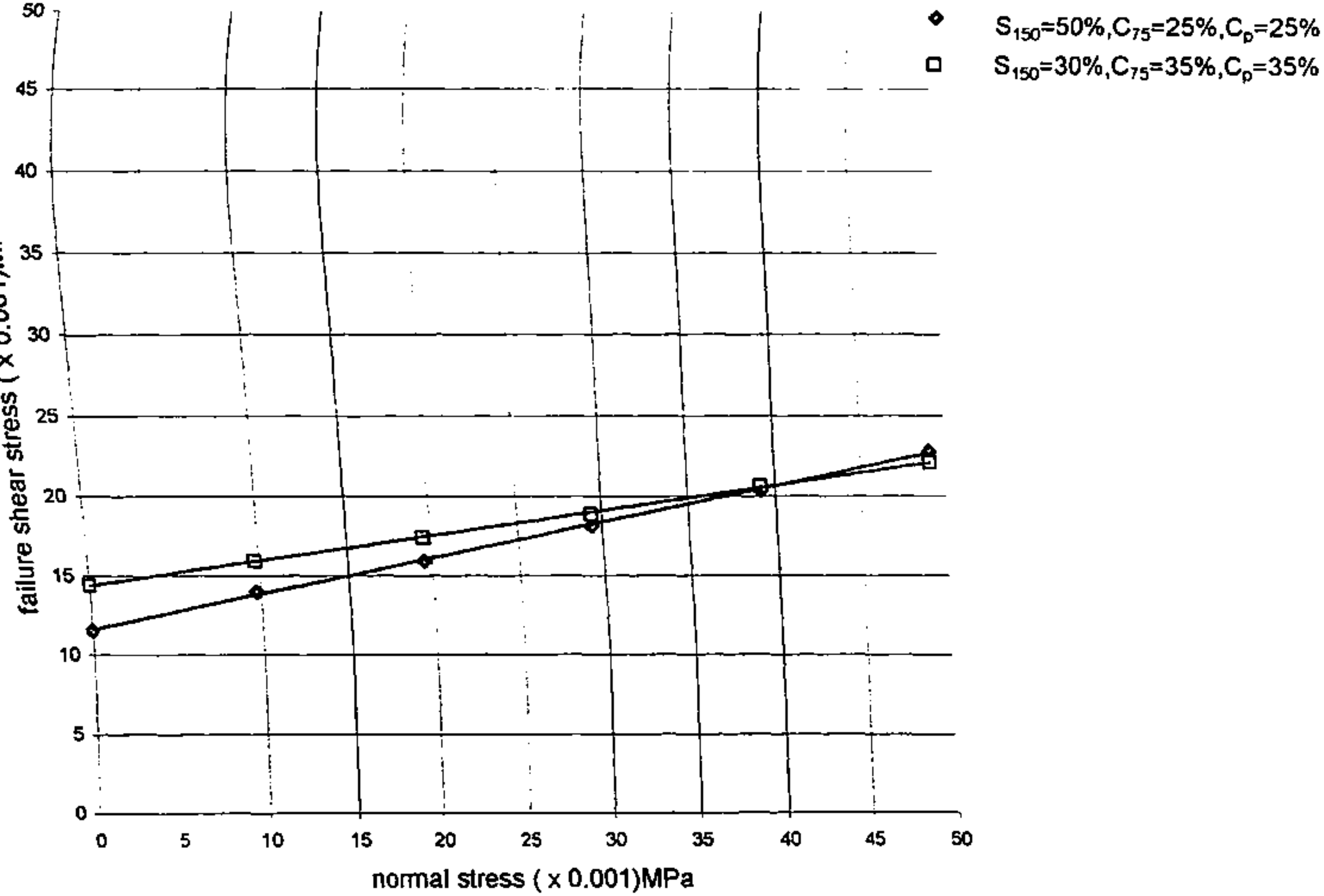

Figure 3.4 : Variation of failure shear stress as a function of normal stress for two soil samples containing 10% by weight of water

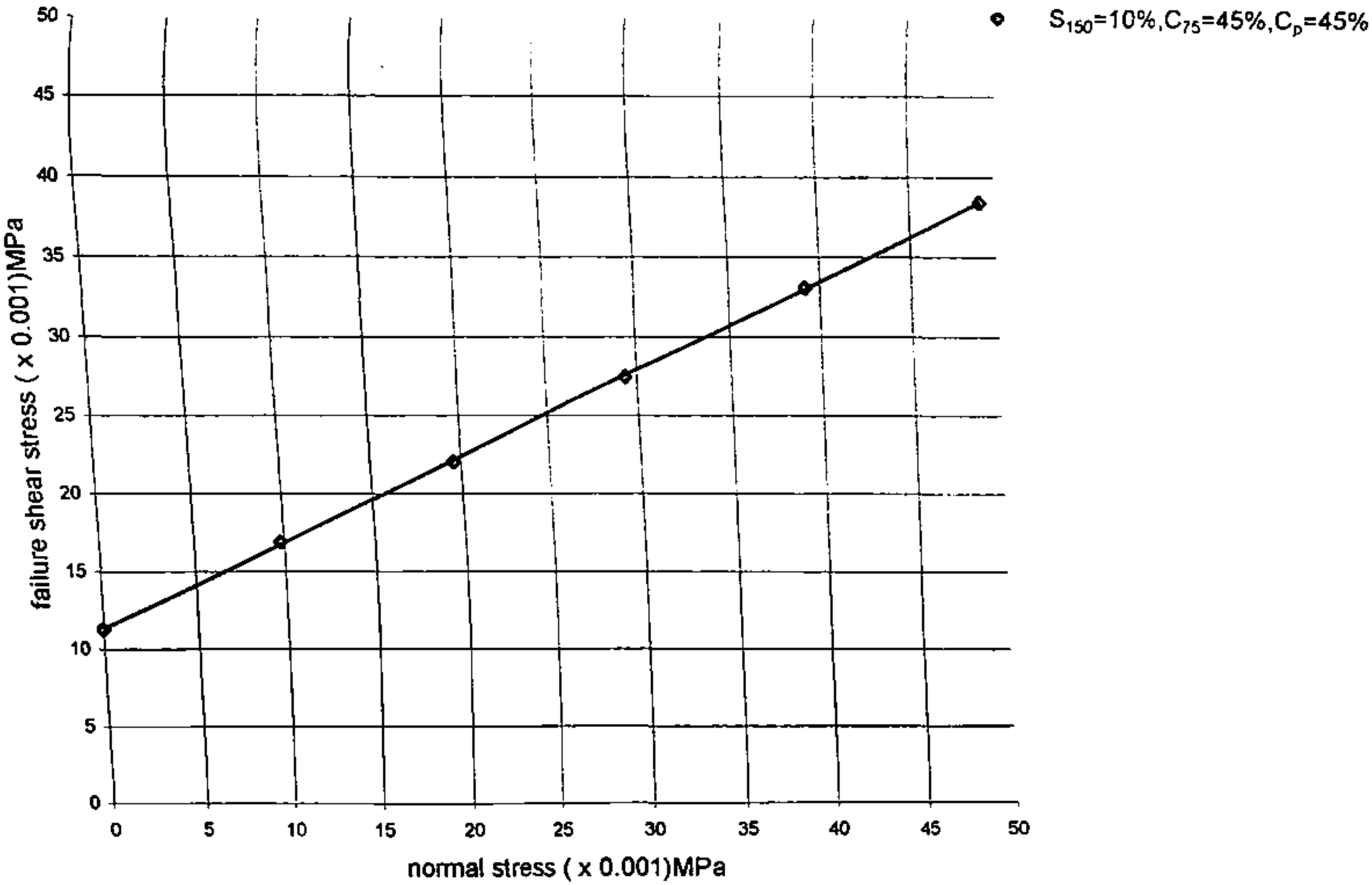

Figure 3.5 : Variation of failure shear stress as a function of normal stress for a soil sample containing 10% by weight of water

TABLE 3.5 Direct Shear Test Results on Local Soil Mix
(at 10% water content and varying particle size composition)

Soil Composition			Cohesion ($x 10^{-3}$) MPa	Angle of Internal Friction (degrees)
Sand retained on 150μ sieve	Silty Clay retained on 75μ sieve	Silty Clay retained on pan		
90%	5%	5%	2.74	35
70%	15%	15%	8.62	22.5
50%	25%	25%	11.56	13
30%	35%	35%	14.4	9
10%	45%	45%	11.27	29

TABLE 3.6 Direct Shear Test Results on Local Soil Mix
(effect of pore water content variation)

SOIL COMPOSITION
Sand retained on 150μ sieve = 50%
Silty Clay retained on 75μ sieve = 25%
Silty Clay retained on pan = 25%

Pore Water Salt Concentration = 0%

Normal Stress ($x 10^{-3}$) MPa	Failure Shear Stress ($x 10^{-3}$) MPa				
	W1	W2	W3	W4	W5
4.9	9.32	11.13	12.93	13.23	6.32
9.6	14.13	14.5	16.84	16.54	9.92
19.2	20.45	23.7	24.06	24.66	18.04
39.2	33.69	36.99	39.7	40.3	27.37
49	42.41	42.11	45.72	48.13	30.68

W1 = 0% Pore Water Content
W2 = 6% Pore Water Content
W3 = 10% Pore Water Content
W4 = 15% Pore Water Content
W5 = 20% Pore Water Content

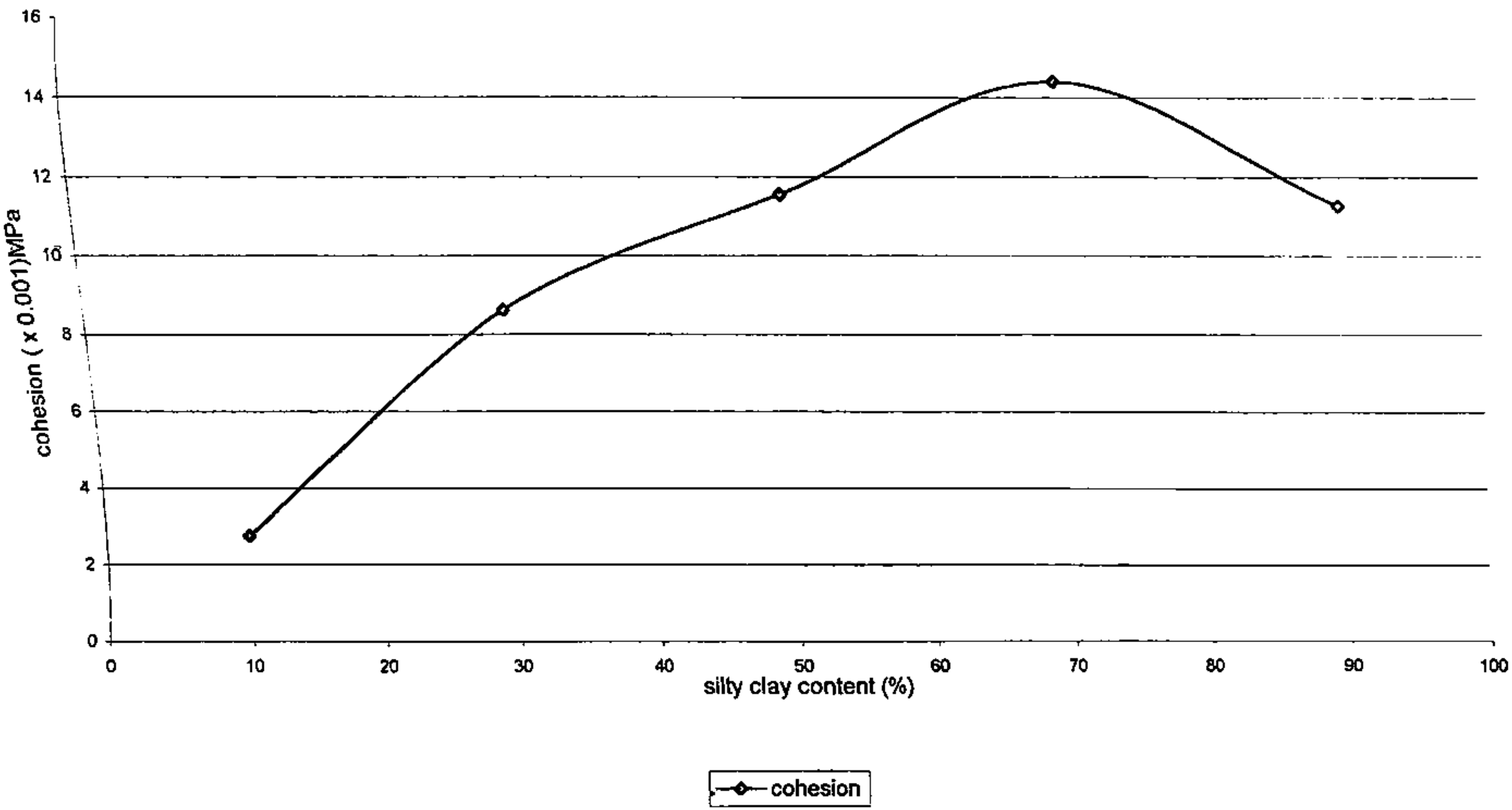

Figure 3.6 : Variation of cohesion with particle size composition (water content = 10%, salt concentration = 0M)

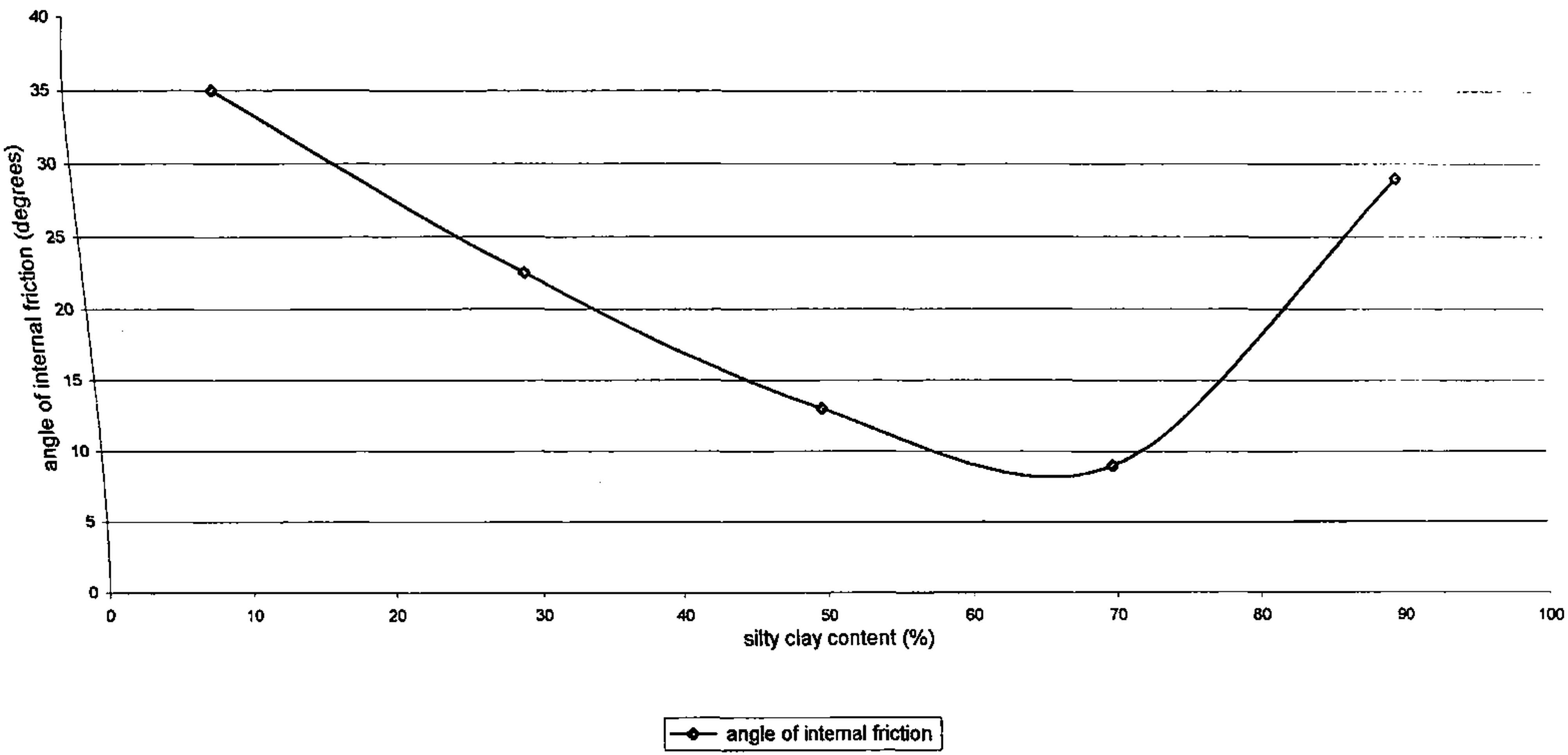

Figure 3.7 : Variation of friction angle with particle size composition (water content =10%, salt concentration = 0M)

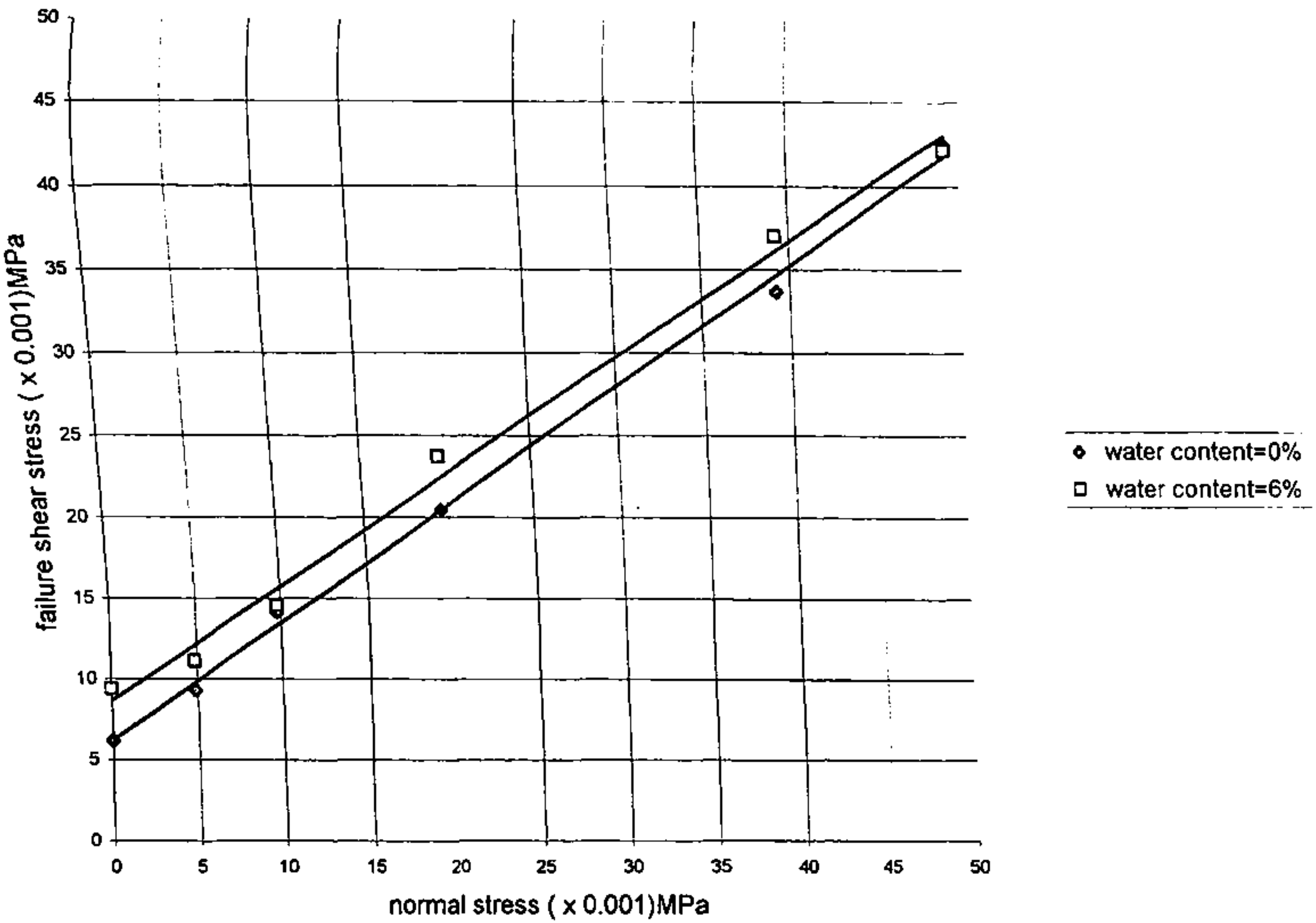

Figure 3.8 : Variation of failure shear stress as a function of normal stress for a soil mix at two different water contents (S_{150}=50%,C_{75}=25%,C_p=25%)

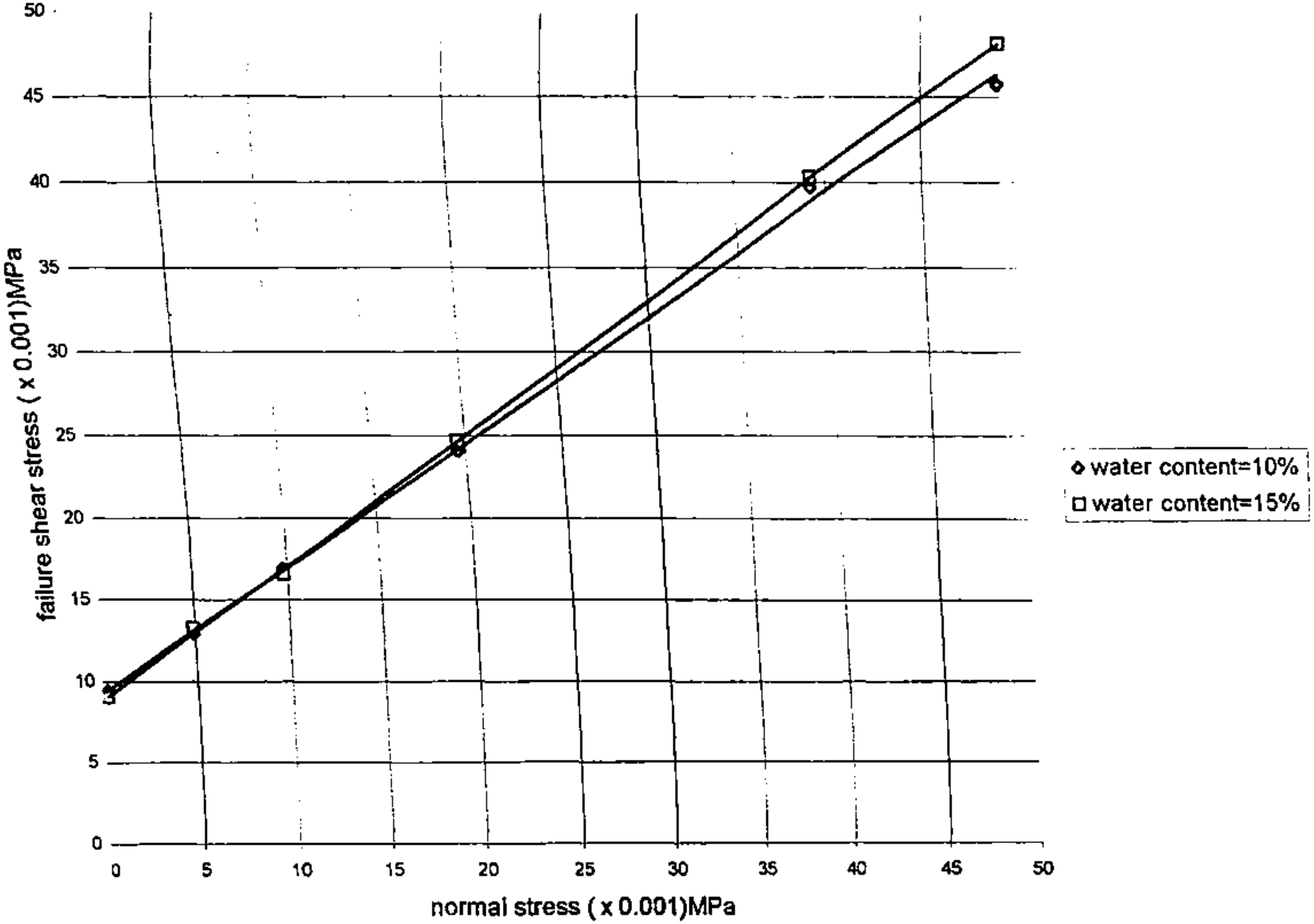

Figure 3.9 : Variation of failure shear stress as a function of normal stress for a soil mix at two different water contents (S_{150}=50%,C_{75}=25%,C_p=25%)

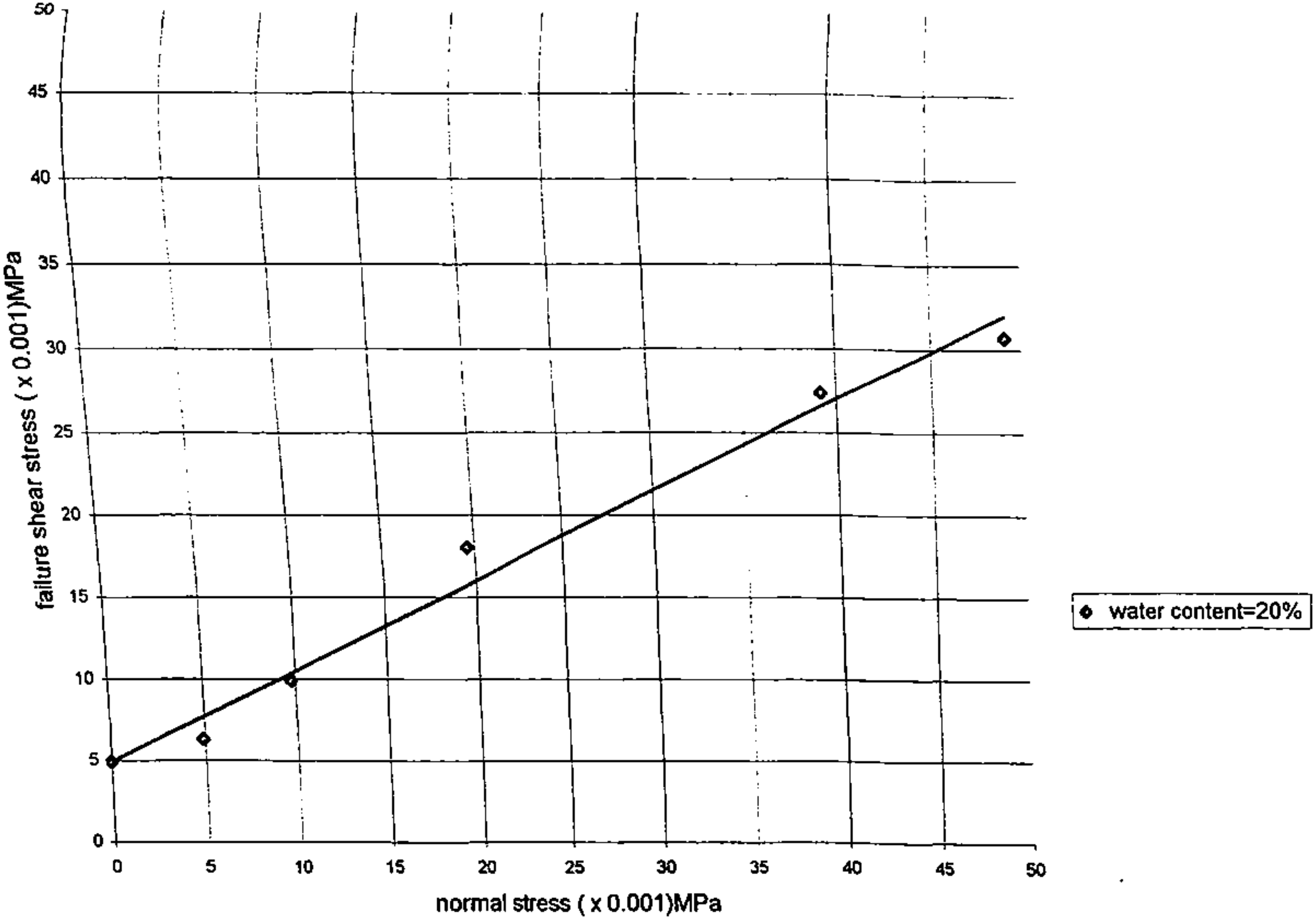

Figure 3.10 : Variation of failure shear stress as a function of normal stress for soil mix at 20% water content(S_{150}=50%,C_{75}=25%,C_p=25%)

TABLE 3.7 Direct Shear Test Results
(at constant particle size composition and varying water content)

Water Content (%)	Cohesion ($\times 10^{-3}$) MPa	Angle of Internal Friction (degrees)
0	6.2	35.92
6	9.407	34.47
10	9.4	36.98
15	9.045	38.55
20	4.93	28.92

TABLE 3.8 Direct Shear Test Results on Local Soil Mix
(effect of pore water salt concentration)

SOIL COMPOSITION
Sand retained on 150μ sieve=50%
Silty clay retained on 75μ sieve=25%
Silty clay retained on pan=25%

Water Content=10%

Normal Stress ($\times 10^{-3}$) MPa	Failure Shear Stress ($\times 10^{-3}$) MPa			
	S1	S2	S3	S4
4.9	13.23	13.83	14.44	13.23
9.6	17.14	17.74	17.44	17.74
19.2	24.06	24.06	24.36	23.16
39.2	40.3	40.3	37.9	37.3
49	46.62	49.03	46.62	44.5

S1=1M Salt Concentration
S2=2M Salt Concentration
S3=3M Salt Concentration
S4--4M Salt Concentration

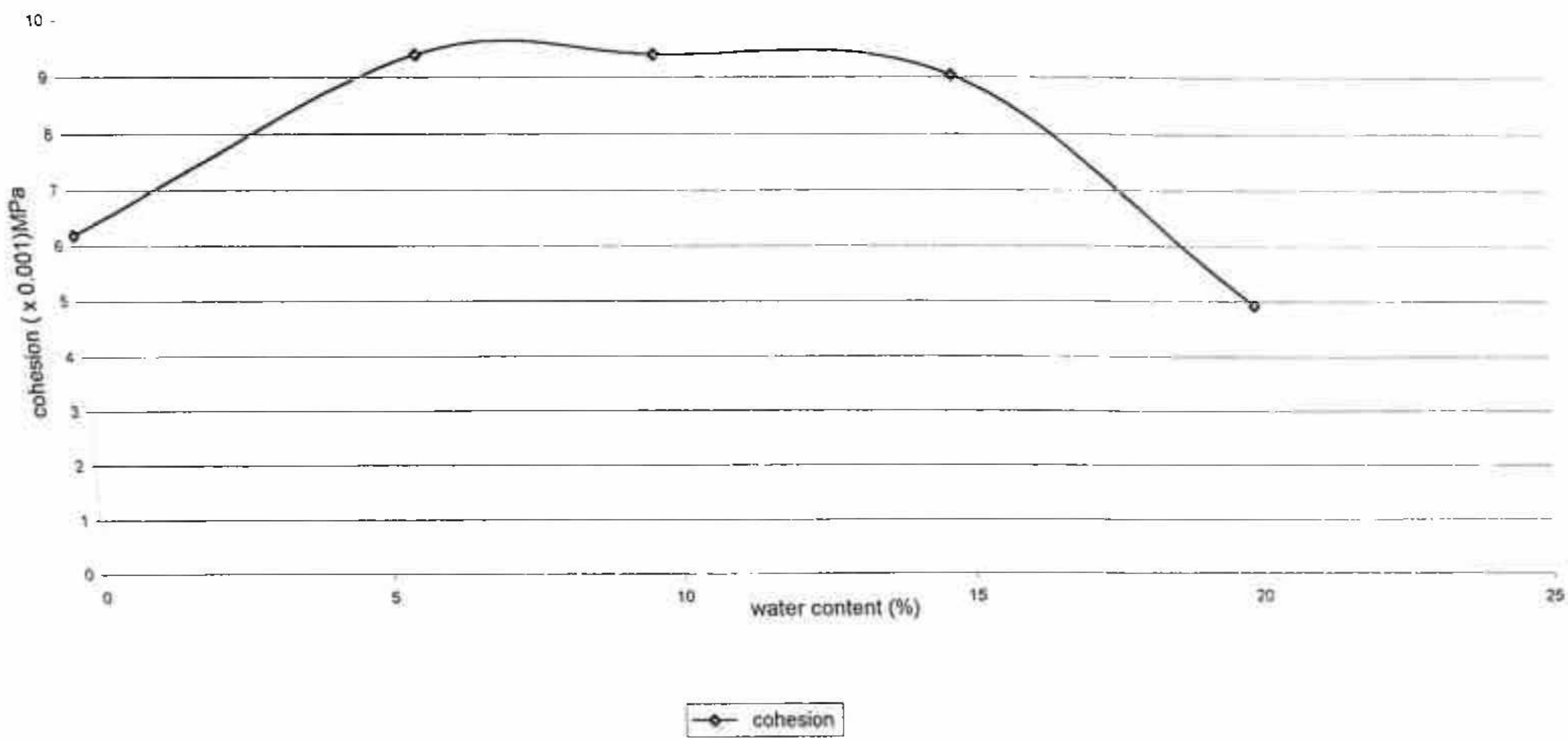

Figure 3.11 : Variation of cohesion with water content (S_{150}=50%,C_{75}=25%,C_p=25%,salt concentration =0M)

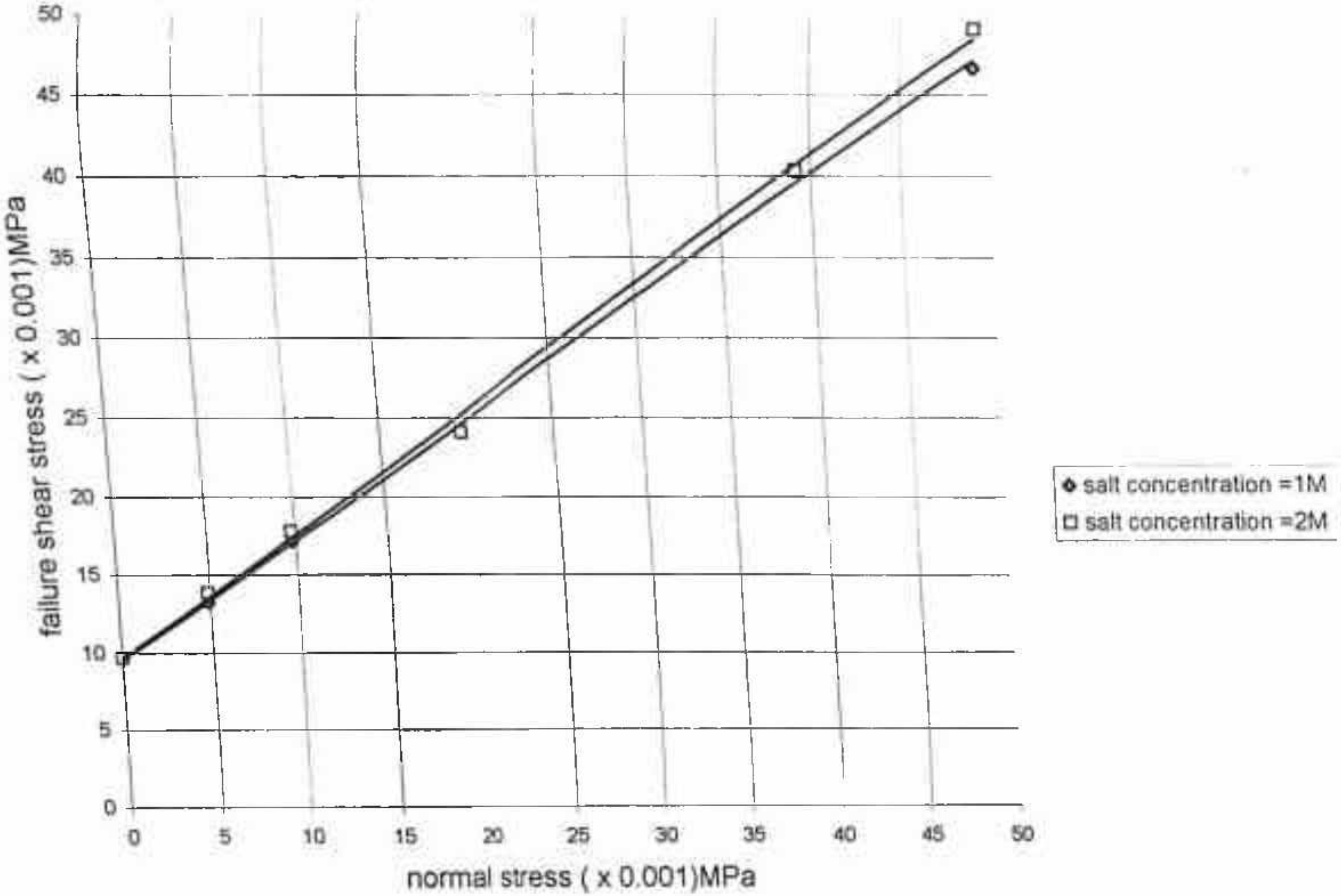

Figure 3.12 : Variation of failure shear stress as a function of normal stress for a soil mix at two different pore water salt concentration (S_{150}=50%,C_{75}=25%.C_p=25%, water content = 10%)

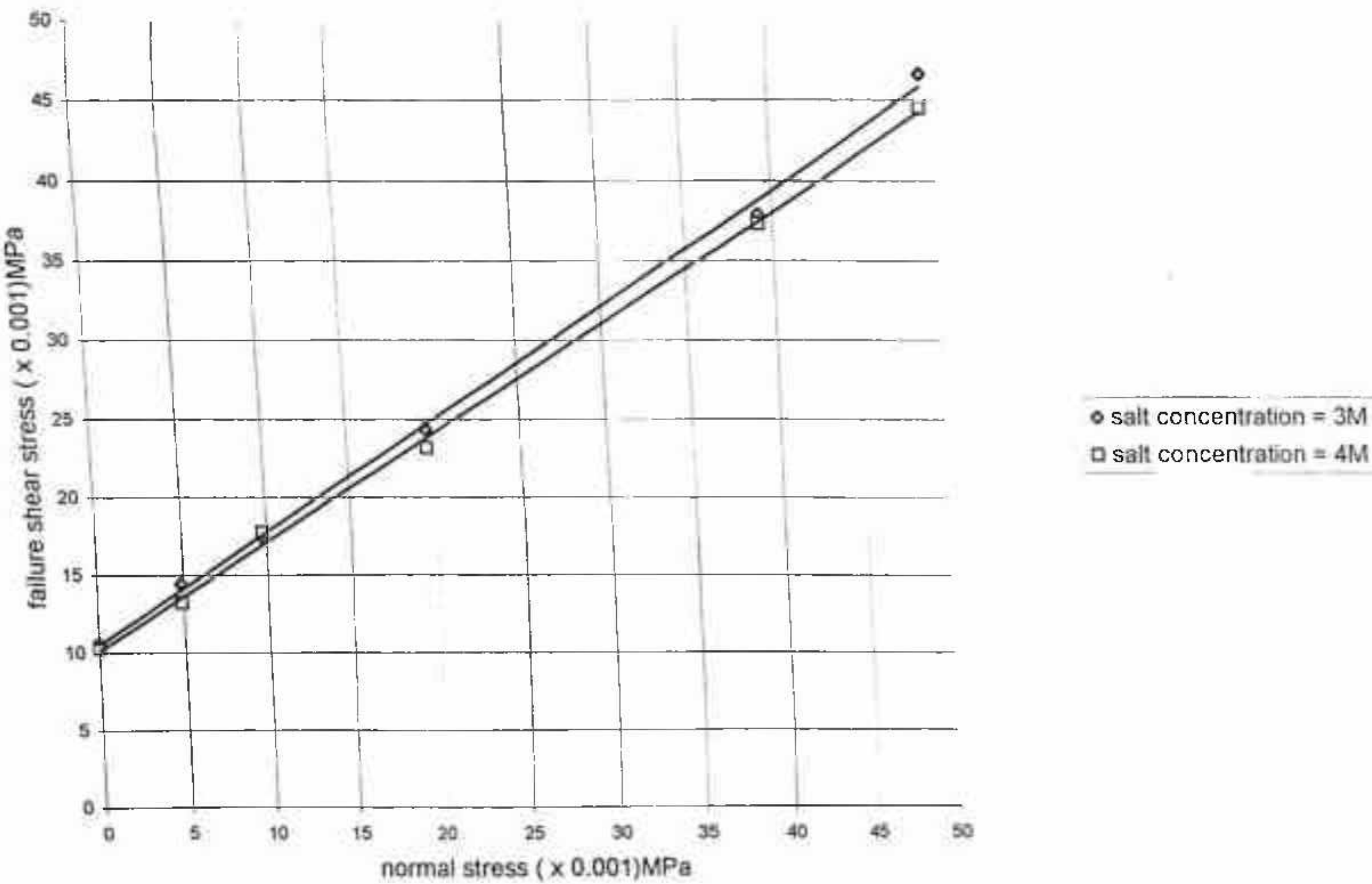

Figure 3.13 : Variation of failure shear stress as a function of normal stress for a soil mix at two different pore water salt concentration (S_{150}=50%,C_{75}=25%,C_p=25%,water content =10%)

TABLE 3.9 Direct Shear Test Results on Local Soil Mix
(at constant particle size composition and varying salt concentration)

Salt Concentration	Cohesion ($\times 10^{-3}$) MPa	Angle of Internal Friction (degrees)
1M	9.62	37.4
2M	9.69	38.36
3M	10.6	35.75
4M	10.23	34.81

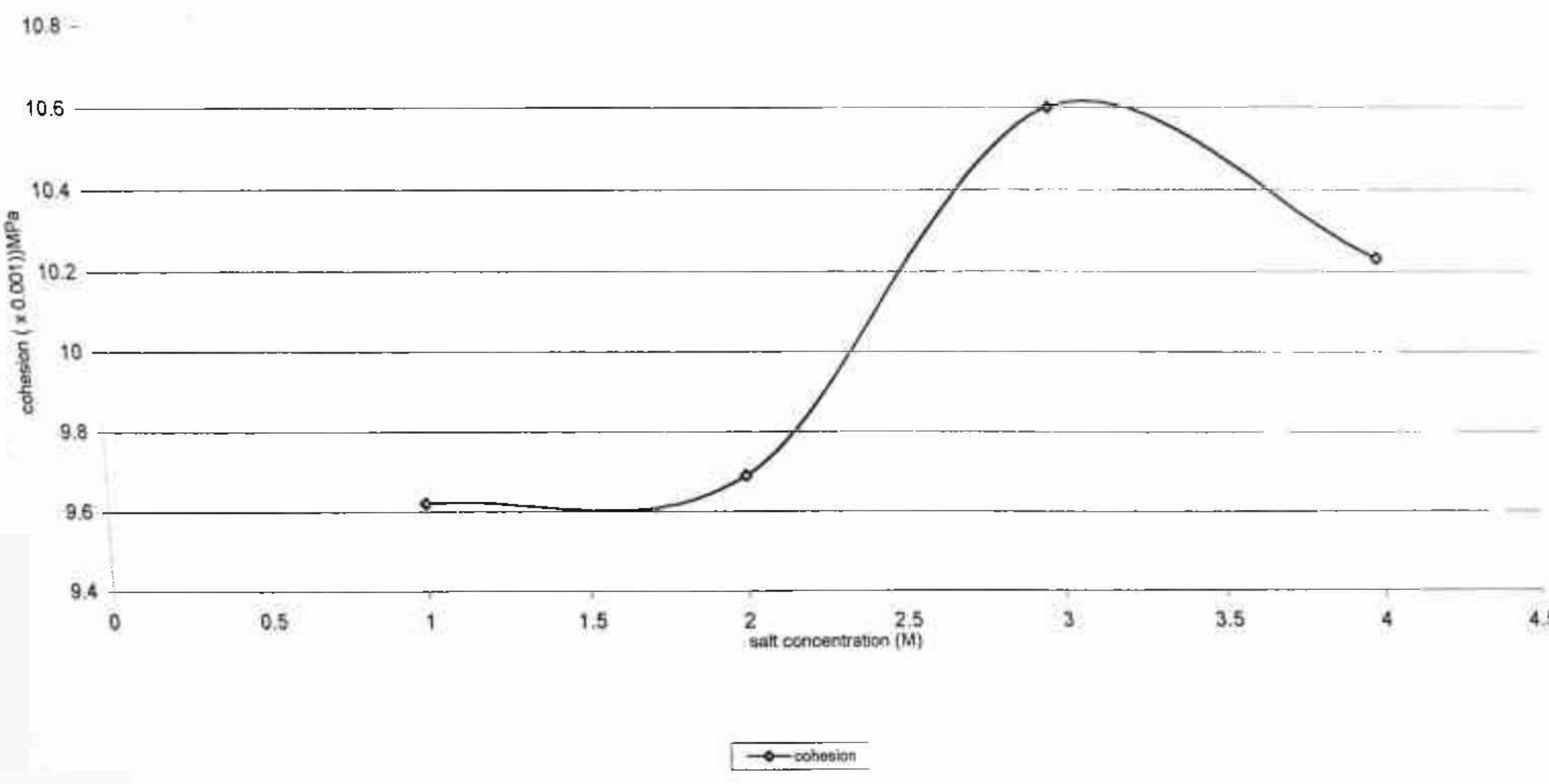

Figure 3.14 : Variation of cohesion with pore water salt concentration (S_{150}=50%,C_{75}=25%,C_p=25%, water content = 10%)

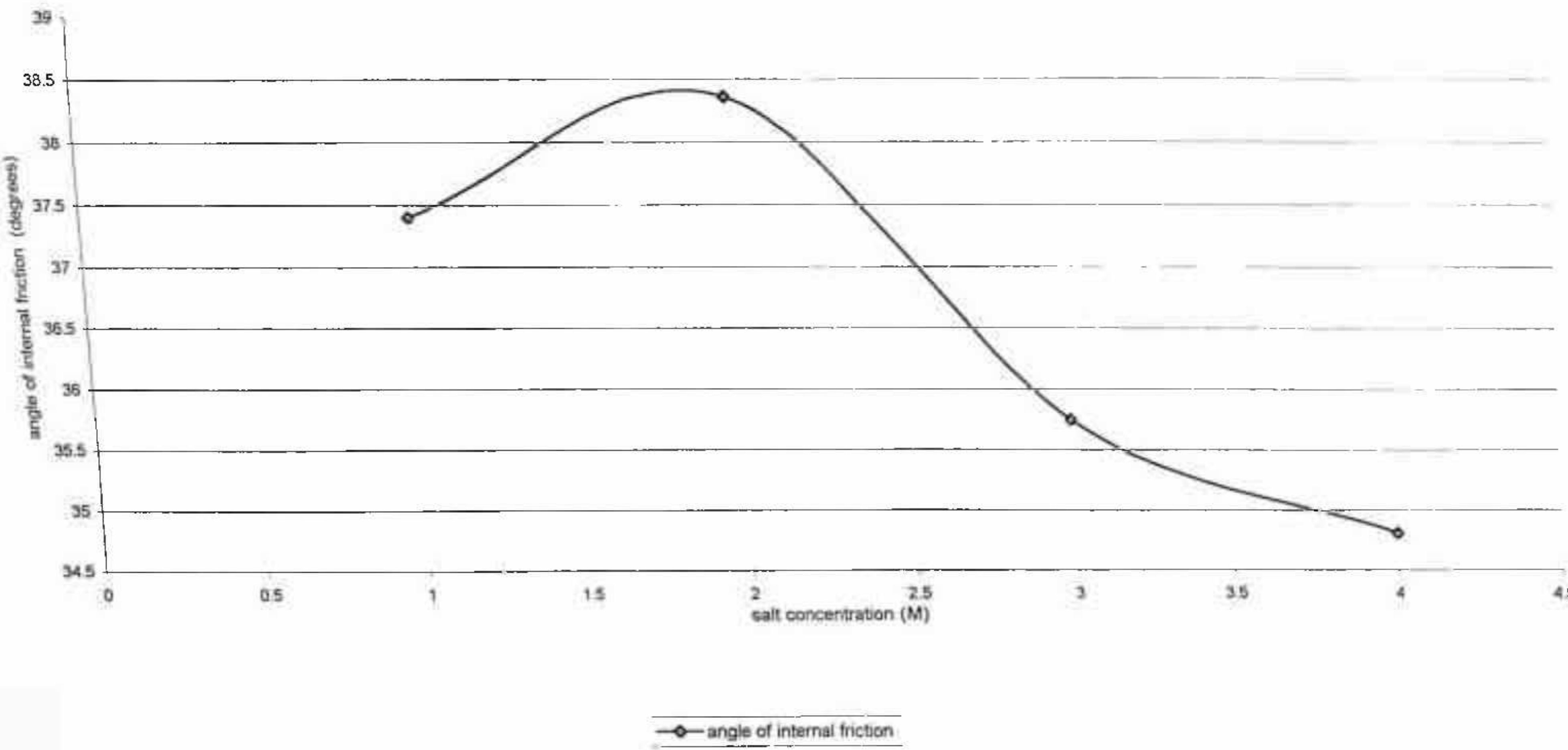

Figure 3.15 : Variation of friction angle with pore water salt concentration (S_{150}=50%,C_{75}=25%,C_p=25%, water content = 10%)

3.2 SHEAR STRENGTH BEHAVIOUR OF REINFORCED SOIL

TEST MATERIAL

Soil samples used in the present study were collected from locations close to the BITS campus. Sandy as well as silty clayey soil were used in the study. Method of collection of these soils has been described in detail on pages 46 and 47. Particle size composition of soil was kept constant in all the experiments. Soil sample consisted 50% by weight of sandy soil retained on 150μ sieve, 25% by weight of silty clayey soil retained on 75μ sieve and 25% by weight of silty clayey soil retained on pan. At this composition soil was found to have a liquid limit of 24.4% and plastic limit of 14.1%. This soil also had minimum dry density of 1253.55 kg/m^3, maximum dry density of 1621.16 kg/m^3 and specific gravity of 2.42.

Pore water content was also kept constant during testing. Tap water available in the laboratory was used. Water content was kept at 10% in the reinforced soil.

E-glass fibres were used as reinforcing agent in the present study. Fibres were taken from glass fibre woven mats. Fibre length was kept constant at 5 cm in all the experiments. Important properties of E-glass fibres are given in Table 2.1 on page 30.

TEST PROCEDURE

Reinforced soil was subjected to direct shear testing in present study. Fibre orientation was kept perpendicular to horizontal failure surface. Furthermore fibres were placed transverse to the sliding direction of direct shear testing. Weight fraction of fibres was the only parameter varied during testing.

In the first set of direct shear testing with soil reinforced with glass fibres, a layer of soil having composition and water content as mentioned earlier, was spread on a glass plate to get 6 cm by 6 cm plan area. Thickness of soil sample was kept at 2 cm. On top of this soil layer, 1 strand of glass fibres of 5 cm length was kept in the centre along the length of sample. Another layer of soil having 2 cm thickness was kept on top of it. One strand of glass fibres of 5 cm length was again kept on top of second soil layer in the centre. Fibre was kept along the same direction as the fibre in the previous layer. Finally another layer of 2 cm thick soil was kept on top of second fibre layer.

Thus the whole composite consisted of 3 layers of soil each 2 cm thick and having 6 cm by 6 cm plan area. In between two soil layers, there was a layer of glass fibre strand. Weight fraction of glass fibre in the composite was 0.02% by weight. Weight fraction of fibre in composite was obtained by taking ratio of total weight of fibre in composite to total weight of composite and expressed on percentage basis.

This composite was rotated 90^0 and placed inside the shear box. This made glass fibres vertical. Consequently it was perpendicular to the horizontal failure surface. Horizontal failure surface is predetermined failure surface in direct shear testing. Dimensions of the composite were selected based on the dimensions of the shear box (i.e. 6 cm X 6 cm X 6 cm). Same orientation of fibres was kept in all the experiments.

Direct shear testing was then conducted in usual manner. Failure shear load corresponding to four different normal loads were obtained. These loads when divided with inside plan area of shear box gave the corresponding normal and failure shear stress values Cohesion and angle of internal friction for the composite was then found out in the usual manner.

In further set of direct shear tests, fibre content of the composite was gradually increased. Method of preparing composite was same as before. However amount of glass fibres in between two soil layers were increased gradually. Thus all the composites tested were having three soil layers and two fibre layers. In these composites, fibre content gradually increased and the weight fraction of glass fibres in these composites was 0.02, 0.04, 0.06, 0.08, 0.1 and 0.12.

Composites having different weight fractions were placed in the shear box and direct shear testing was conducted in usual manner to obtain cohesion and angle of internal friction.

When fibre weight fraction was increased beyond 0.12, it was found that even after shearing to the maximum limit of the direct shear testing, horizontal load dial (proving ring) was not indicating failure even at the lowest normal load of 9.8×10^{-3} MPa. In other words, failure of the composite was not taking place even at the lowest normal load within the limits of direct shear testing. This observation is due to the fact that with more and more fibre addition, there is more and more increase in shear strength of the composite. When fibre content of composite increases beyond 0.12% by weight, it was not possible to measure strength of composite by direct shear testing.

One set of test was also conducted without the presence of glass fibres in the soil. Particle size composition of soil and pore water in soil was kept same as soil reinforced with fibres. Cohesion and angle of internal friction for this condition was also found using direct shear testing.

Results of direct shear testing on fibre reinforced soil matrix is shown in Table 3.10 and the variation of failure shear stress with normal stress is plotted in Figures 3.16, 3.17 and 3.18. Variation in cohesion and angle of internal friction with fibre content by weight in the composite are shown in Table 3.11. Variation of cohesion with fibre content and variation of angle of internal friction with fibre content is indicated in Figures 3.19 and 3.20 respectively.

TABLE 3.10 Direct shear test results of reinforced soil

SOIL COMPOSITION Sand (retained on 150μ sieve) = 50% Silty clay (retained on 75μ sieve) = 25% Silty clay (retained on pan) = 25% Water content = 10%							
	Failure shear stress ($\times 10^{-3}$) MPa						
Normal stress ($\times 10^{-3}$) MPa	F0	F1	F2	F3	F4	F5	F6
9.8	16.54	18.05	18.44	18.05	20.5	20.46	19.85
19.6	24.66	25.85	26.18	27.07	28.1	27.96	30.08
39.2	40.3	41.45	41.65	42.71	43.35	42.79	43.61
49	48.13	49.25	49.56	48.73	50.92	50.23	49.93

F0 = Glass fibre content, 0% by weight
F1 = Glass fibre content, 0.02% by weight
F2 = Glass fibre content, 0.04% by weight
F3 = Glass fibre content, 0.06% by weight
F4 = Glass fibre content, 0.08% by weight
F5 = Glass fibre content, 0.10% by weight
F6 = Glass fibre content, 0.12% by weight

TABLE 3.11 Variation of cohesion and angle of internal friction with fibre content (at constant particle size composition)

Fibre content (% by weight)	Cohesion ($\times 10^{-3}$) MPa	Angle of internal friction (degrees)
0	8.76	38.8
0.02	10.23	38.52
0.04	10.68	38.33
0.06	11.04	38.15
0.08	12.83	37.79
0.10	13.11	37.12
0.12	13.79	36.87

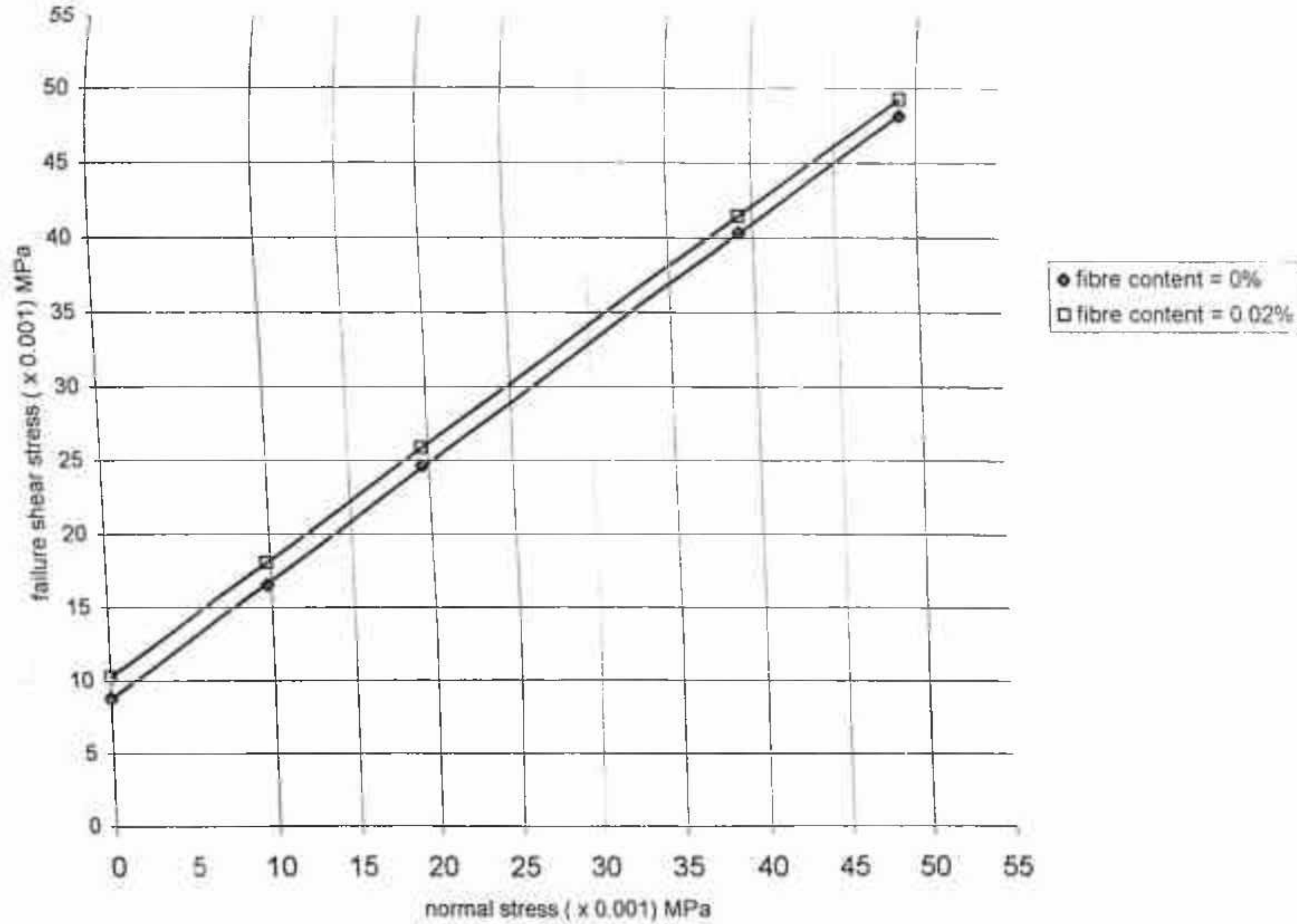

Figure 3. 16 : Variation of failure shear stress as a function of normal stress for soil sample at two different fibre content (S_{150}=50%,C_{75}=25%,C_p=25%)

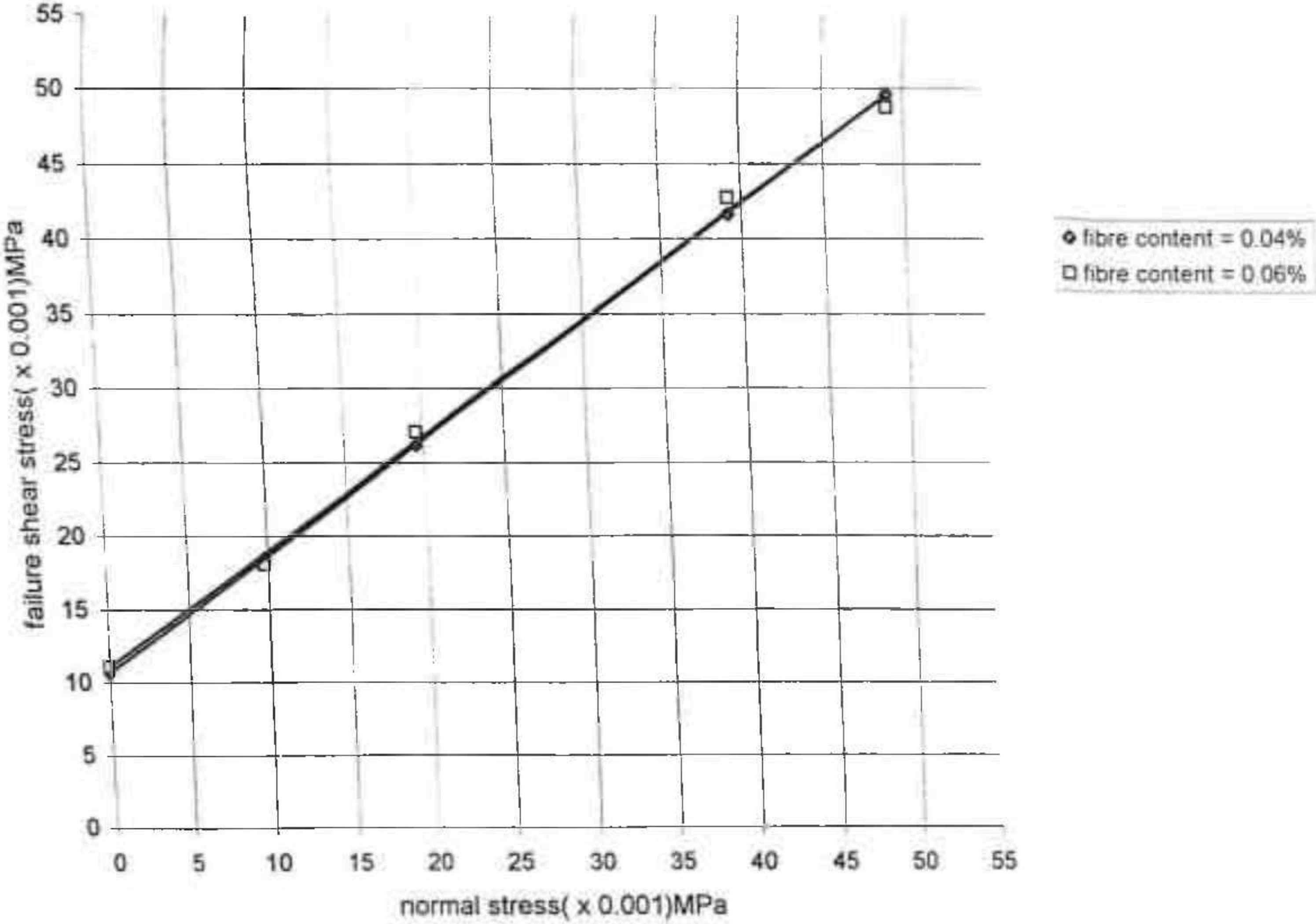

Figure 3.17 : Variation of failure shear stress as a function of normal stress for soil sample at two different fibre content (S_{150}=50%,C_{75}=25%,C_p=25%)

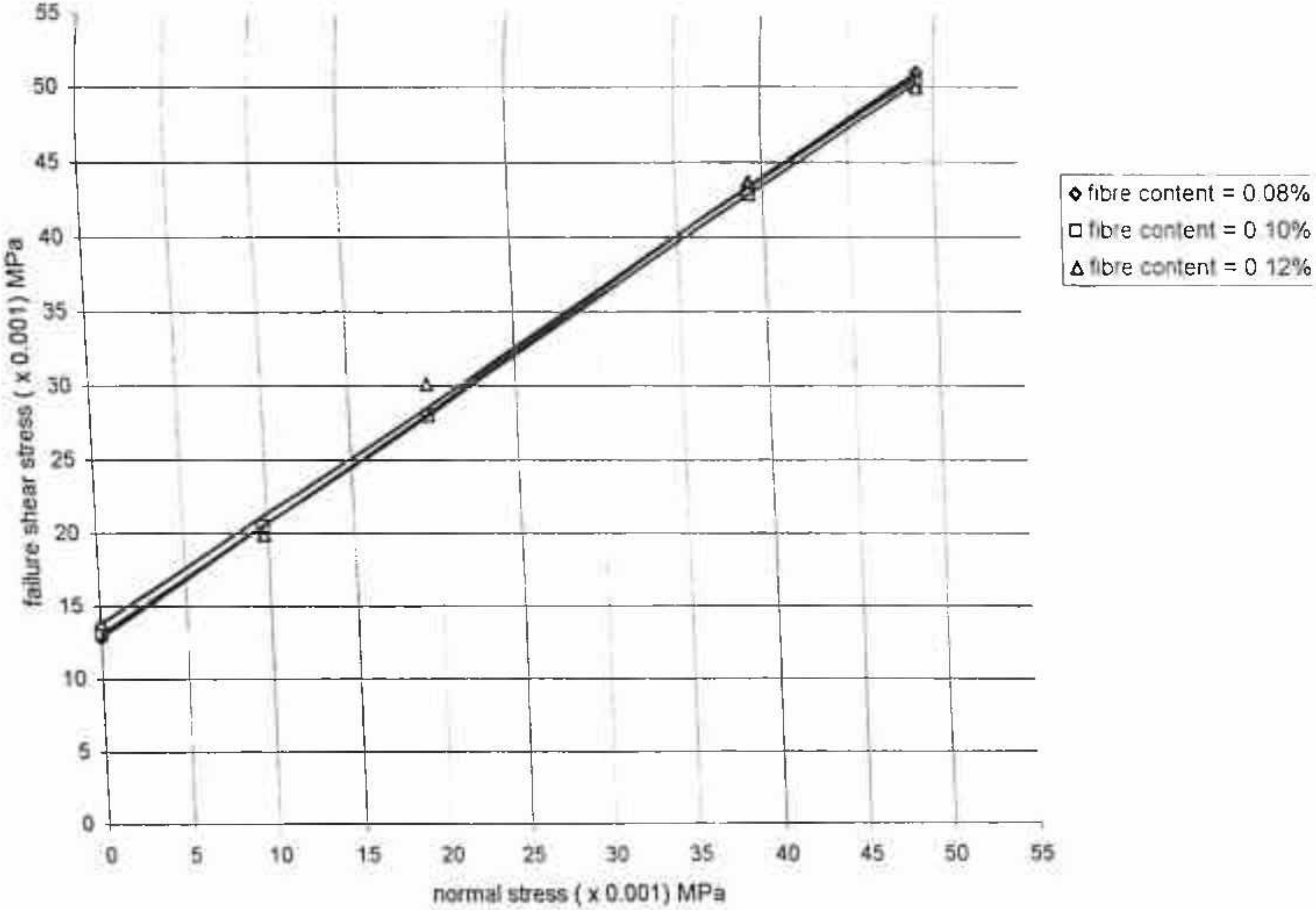

Figure 3.18 : Variation of failure shear stress as a function of normal stress for soil sample at three different fibre content (S_{150}=50%,C_{75}=25%,C_p=25%)

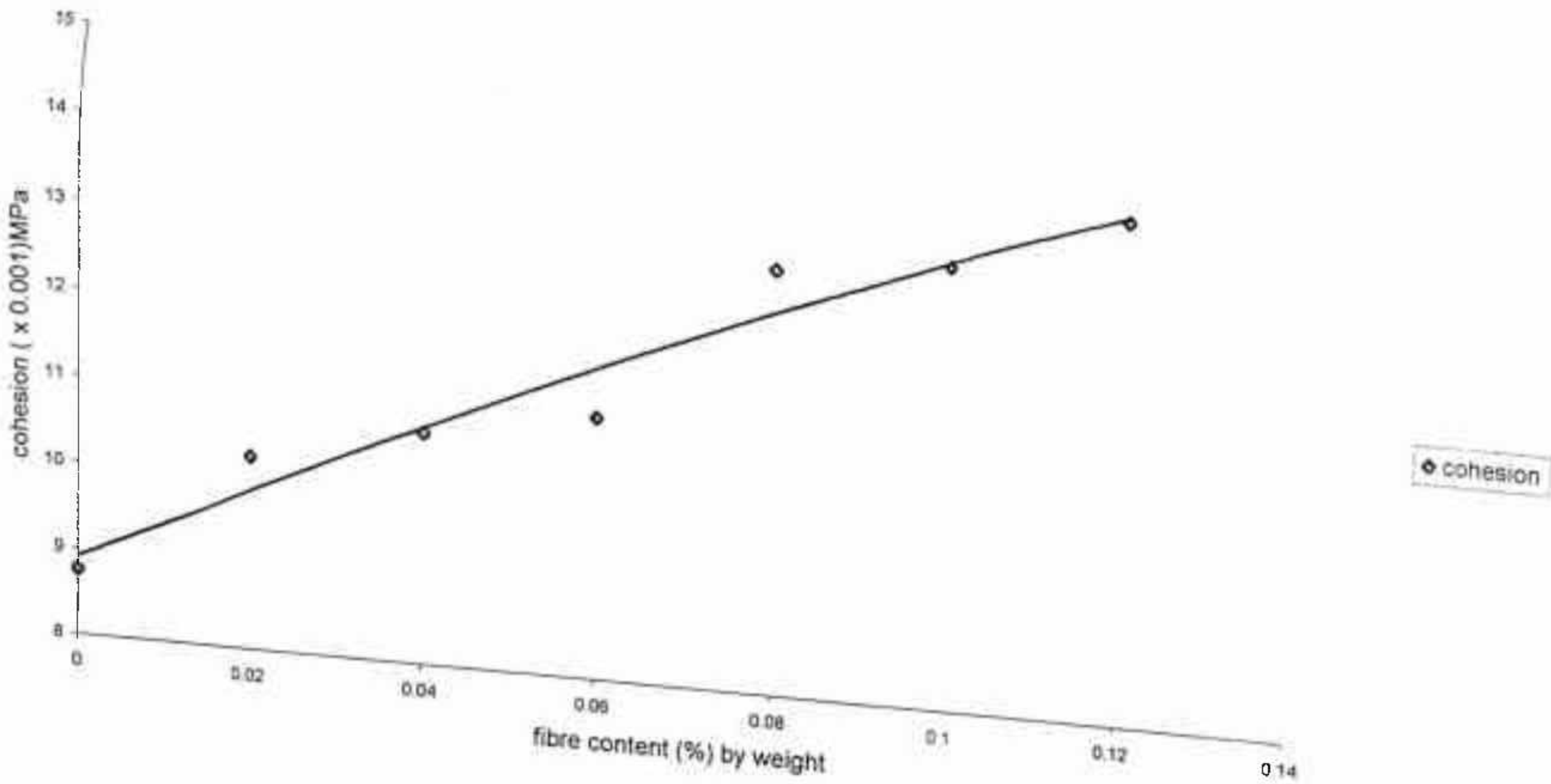

Figure 3.19 : Variation of cohesion with fibre content (S_{150}=50%,C_{75}=25%,C_p=25%)

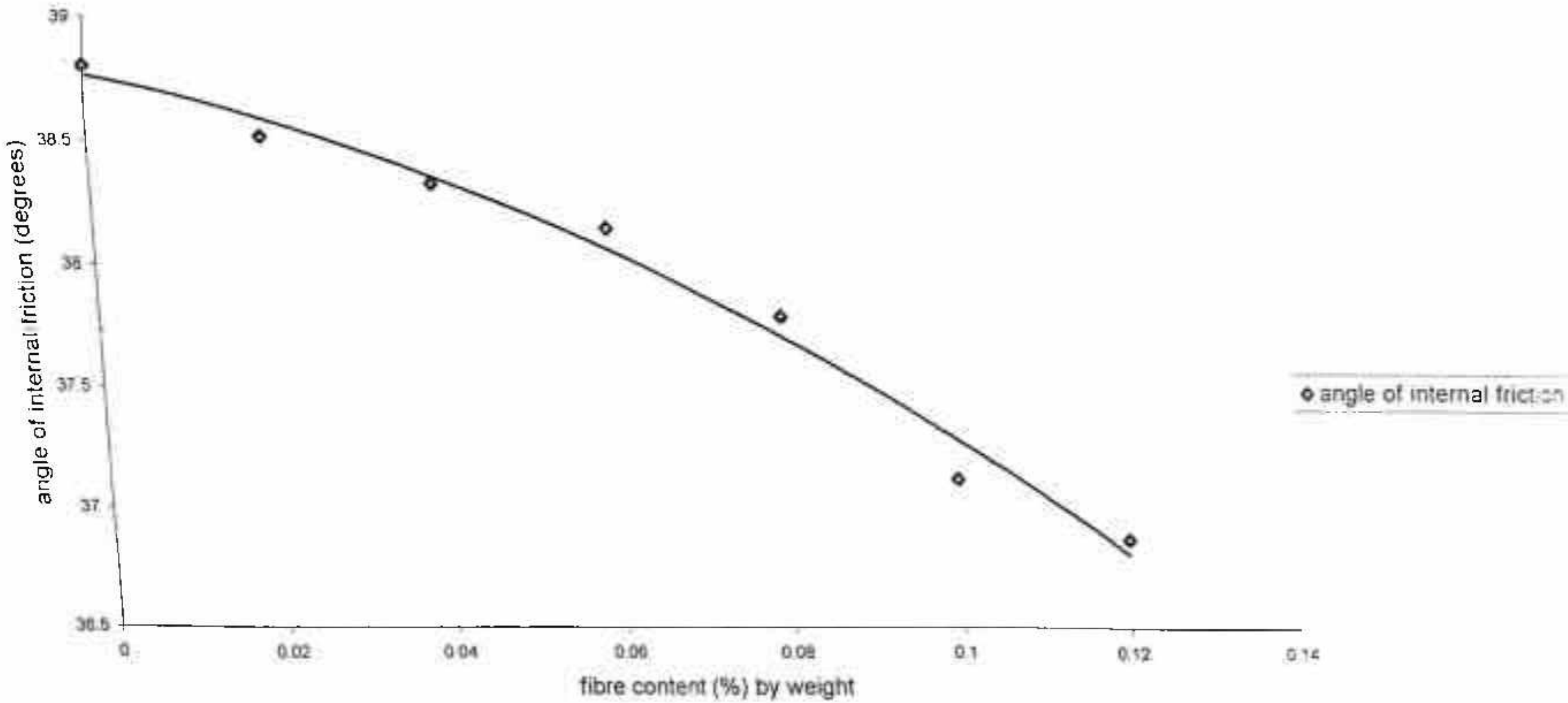

Figure 3.20 : Variation of angle of internal friction with fibre content (S_{150}=50%,C_{75}=25%,C_p=25%)

STRENGTH DETERIORATION OF GLASS FIBRE REINFORCED SOIL COMPOSITES

Strength deterioration of glass fibre reinforced soil composites with time was studied. Soil used in the present study, its method of collection and its important physical properties have already been described on page 72. This soil which was used in the present study was found to have a pH value of 8.48. Consequently soil was alkaline in nature. Pore water content was also kept constant at 10%.

E – glass fibres were used as the reinforcing agent. To study strength deterioration of glass fibre reinforced soil composites, direct shear testing was conducted. Testing was conducted using a sample having 0.12 weight fraction of glass fibres. Method of preparing fibre reinforced soil sample for direct shear testing has already been described on pages 73 and 74.

One set of direct shear testing was conducted just after fabricating the soil composites. Another set of direct shear testing was done on soil composites which were left in the shear box for a week. Direct shear testings were conducted on both types of samples in usual manner. The results of direct shear testing were then used to determine cohesion and angle of internal friction for both types of samples.

Typical results of direct shear testing on these two types of samples have been given on Table 3.12. Graphical representation of direct shear testing is given in Figures 3.21 and 3.22. Table 3.13 lists the values of shear strength parameters for these two types of samples.

TABLE 3.12 Direct shear test results to study strength deterioration of glass fibre reinforced soil composites having constant fibre weight fraction

	Failure Shear Stress (x 10^{-3}) MPa	
Normal Stress (x 10^{-3}) MPa	T1	T2
9.8	19.85	19.03
19.6	30.08	27.15
39.2	43.61	41.45
49	49.93	48.62

T1 = testing conducted at the time of composite formation
T2 = testing conducted one week after composite formation

TABLE 3.13 Variation of shear strength parameters due to strength deterioration of glass fibre reinforced soil composites
(fibre content = 0.12% by weight)

Sample specification	Cohesion (x 10^{-3})MPa	Angle of internal friction (degrees)
T1	13.79	36.87
T2	12.88	36.13

T1 = testing conducted at the time of composite formation
T2 = testing conducted one week after composite formation

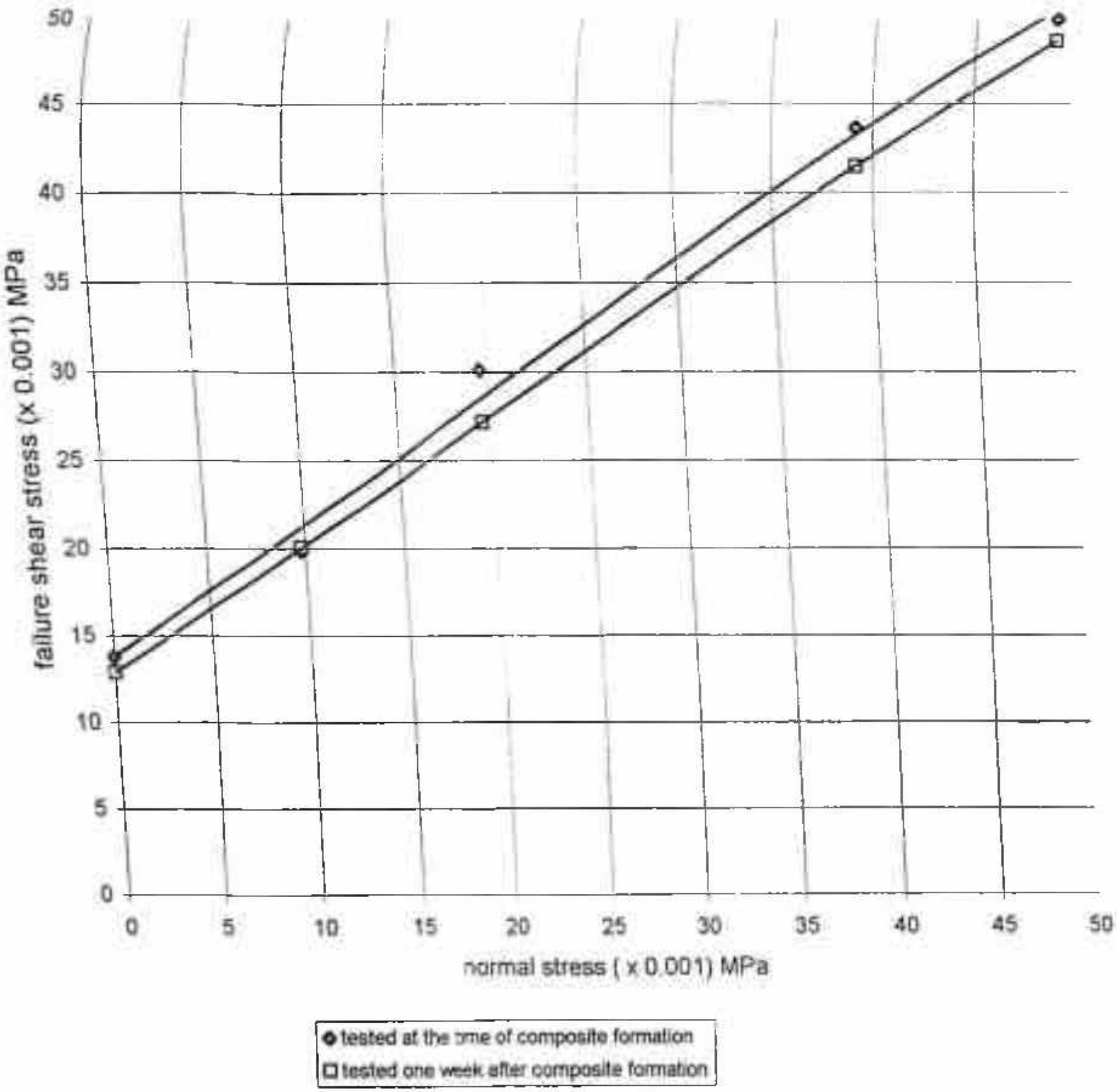

Figure 3.21 : Variation of failure shear stress as a function of normal stress to study effect of fibre deterioration in reinforced soil (S_{150}=50%,C[illegible]=25%,C_p=25%, fibre content = 0.12% by wt.)

3.3 ULTRASONIC TESTING OF SOILS

SOIL USED

Soil was collected locally close to BITS campus for the study related to ultrasonic determination of cohesion and angle of internal friction. Cohesion and angle of internal friction are important soil properties. They are used to determine shear strength of soils. Conventionally cohesion and angle of internal friction are determined by conducting direct shear test, unconfined compression or triaxial testing. These experiments require specific experimental setup and may be catagorised as destructive tests.

In the present study, an attempt has been made to estimate cohesion and angle of internal friction of locally available soils by knowing ultrasonic pulse velocity through the soil. Ultrasonic pulse velocity determination through soils is simple and can be catagorised as nondestructive testing technique.

Coarse as well as fine grained soil were used for this study. Location, method of collection and important properties of both the soils have already been described on pages 46 and 47. Particle size distribution of both the soils is given in Table 3.1 on page 47. The plot showing particle size distribution is given as Figure 3.1 on page 53 for both the soils.

Sieve analysis of both the soils was done. After doing sieve analysis, coarse grained soil retained on 150μ sieve, fine grained soil retained on 75μ sieve and fine grained soil retained on pan were used for further experiments. Coarse grained soil retained on 150μ sieve was classified as sandy. Similarly fine grained soil retained on 75μ sieve and on pan was classified as silty clay. This classification is based on dispersion test. Details of dispersion test have been described earlier on pages 47 and 48.

In addition to determination of cohesion and angle of internal friction, an attempt was made to correlate the safe bearing capacity of soils and ultrasonic pulse velocity through soil. Information about safe bearing capacity of soils is very useful in foundation design. Conventional method of obtaining bearing capacity of soils by using equations (2.4) and (2.5) have been described earlier on pages 43, 44 and 45. Whereas, parameters like effective surcharge, total unit weight of soil, depth of embedment of footing as well as width of footing as required in equations (2.4) and (2.5) can be obtained conveniently, information about cohesion and angle of internal friction require complicated experimental setup. Furthermore, if one may estimate the bearing capacity of soils directly by measuring ultrasonic pulse velocity through the soil under test, it will be a great step forward because determination of ultrasonic pulse velocity through soils is much simpler than determination of cohesion and angle of internal friction.

Soil samples used to estimate bearing capacity of soils by knowing ultrasonic pulse velocity through them was collected from locations close to BITS Pilani campus. All the locations were within a radius of 100 km from BITS Pilani campus. 40 soil samples from 40 different locations were collected. At all the locations, test pits were excavated upto a depth of 2 meters from ground surface. Soil samples were collected from that depth. Important soil properties like in-situ water content, in-situ void ratio and in-situ density for all the soil samples are given in Table 3.15.

PORE WATER USED

To estimate cohesion and angle of internal friction of soil by knowing ultrasonic pulse velocity through them, water content in the soil mix was taken as 10%. Same water content was used in all the experiments. These two parameters are required for calculation of safe bearing capacity of the soil as well as shear strength of the soil.

To estimate directly the safe bearing capacity of soil, by measuring ultrasonic pulse velocity through them, water content in each soil sample was varied. Water content equal to in-situ water content in each sample was taken. Tap water available in our soil mechanics laboratory was used in all the experiments.

TEST PROCEDURE

Ultrasonic testing was done by using ultrasonic materials tester (Model : Emelco type UCT3). This ultrasonic materials tester is a low ultrasonic frequency (150 kHz) tester for civil engineering applications. Coarse grained samples like soils can conveniently be tested with this ultrasonic materials tester. Transmission time of the ultrasonic wave was measured through a given soil sample of known thickness. Testing was done using through transmission technique.

This measurement is used to find out the ultrasonic velocity of longitudinal wave through the soil sample. Ultrasonic velocity is obtained using equation (1.2) given earlier on page 13.

Transmitting and receiving transducers having diameter of 36 mm each were placed on the opposite faces of the soil sample so that their axes remain colinear. Grease was used as coupling agent between transducer face and soil sample. Ultrasonic wave passes through the soil sample from transmitting to receiving transducer. Transmission time of ultrasonic pulse was measured using this ultrasonic materials tester.

To estimate cohesion and angle of internal friction of soils by knowing ultrasonic pulse velocity through them, sandy as well as silty clayey soils were used. Sandy soil retained on 150μ sieve, silty clayey soil retained on 75μ sieve and silty clayey soil retained on pan were used for calibration tests. Five different set of experiments were conducted by changing the weight fraction of sand and silty clay in each set. Fraction of sand and silty clay used in each of the five different set of experiments is same as that given earlier on page 51.

In each set of experiments, required amount of sandy and silty clayey soils were taken. 10% water was added to the soil mix. This soil water mixture was statically compacted in a wooden frame. Density of soil compact in the wooden frame was 1.45 g cm^{-3}. Inner plan area of wooden frame was 60 mm X 60 mm. Furthermore the thickness of wooden frame was 17 mm. The values of ultrasonic pulse velocity through the soil samples, were determined using ultrasonic materials tester.

Variation of ultrasonic transmission velocity with particle size composition of soil is indicated in Table 3.14. This variation plot is shown in Figure 3.22. This plot further indicates the variation of cohesion with varying particle size composition for same soil samples. Variation of ultrasonic pulse velocity as well as angle of internal friction with particle size composition of soil is similarly plotted in Figure 3.23. Figures 3.22 and 3.23 can be used to estimate cohesion and angle of internal friction of soil samples by knowing ultrasonic pulse velocity through the samples. Further details regarding use of ultrasonic technique for measurement of cohesion and angle of internal friction will be discussed in the next chapter.

TABLE 3.14 Variation of ultrasonic pulse velocity with particle size composition of soil (water content = 10%)

Soil composition (percentage by weight)			Sample thickness (centimeters)	Transmission time (micro-seconds)	Ultrasonic pulse velocity (meters per second)
Sand (retained on 150μ sieve)	Silty clay (retained on 75μ sieve)	Silty clay (retained on pan)			
90	5	5	1.7	50.0	340.0
70	15	15	1.7	45.2	376.1
50	25	25	1.7	37.9	448.5
30	35	35	1.7	33.3	510.5
10	45	45	1.7	37.0	459.4

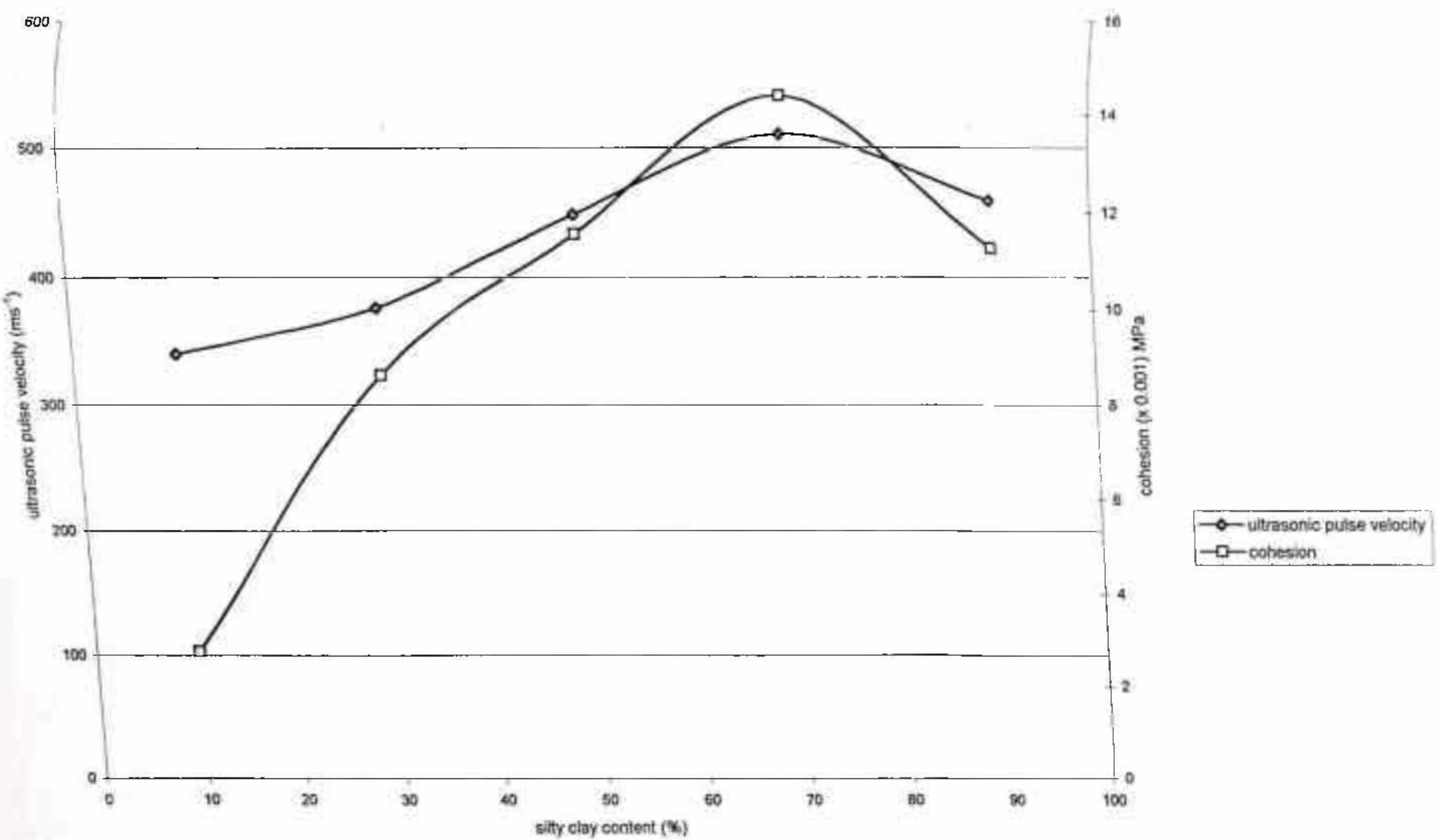

Figure 3.22 : Cohesion and ultrasonic pulse velocity variation with soil particle size composition (at 10% water content)

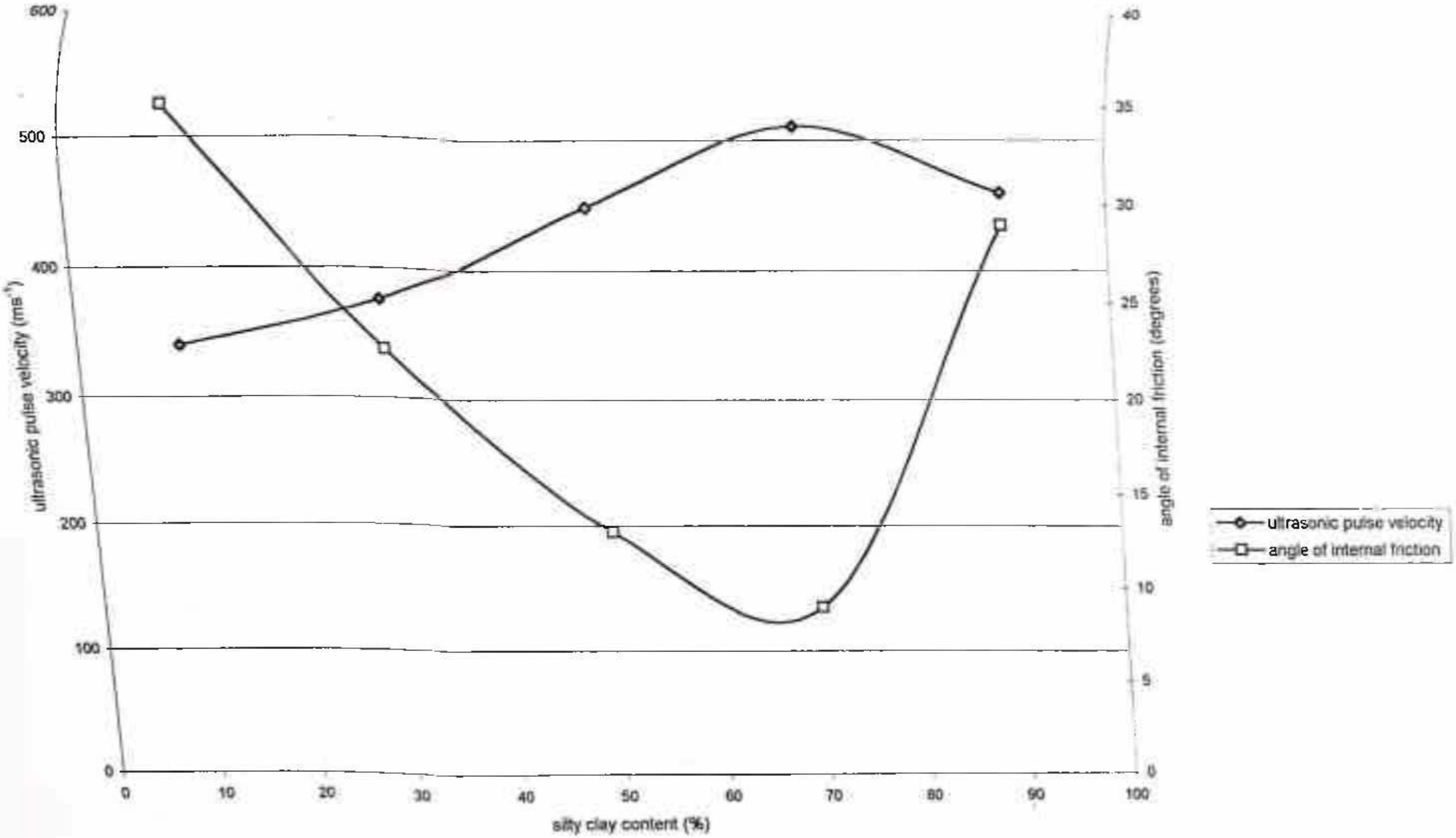

Figure 3.23 : Variation of angle of internal friction and ultrasonic pulse velocity with soil particle size composition (at 10% water content)

For estimation of safe bearing capacity of soils by measuring ultrasonic pulse velocity through the soils, soil samples from 40 different locations were taken. Safe bearing capacity of these soil samples from conventional methods can be obtained by using equation (2.5) given on page 43. This method has been discussed in detail on page 44. Safe bearing capacity values thus obtained for all the soil samples have been given in Table 3.15.

These soil samples were mixed with required amount of water to get in-situ water content in the soil mix. This soil mix was compacted in the wooden frame. Amount of soil mix compacted in the wooden frame was such that the density of soil compact was 1.45 g cm^{-3}. Details of preparing soil samples in the wooden frame has already been described on page 88. Ultrasonic pulse velocity through all these soil samples were measured using ultrasonic materials tester. These measured velocity values for all the soil samples are tabulated in Table 3.15.

Data points indicating variation of ultrasonic pulse velocity through soil samples with respect to safe bearing capacity value of the soil samples have been plotted in Figure 3.24. Curve of best fit through these data points is plotted in Figure 3.24. There exists a correlation between safe bearing capacity of soil samples and ultrasonic pulse velocity (correlation coefficient = 0.94). However there is scatter of data points with respect to best fit curve.

Consequently there appears to be a need to combine certain soil properties with ultrasonic pulse velocity measurements to get a new or apparent ultrasonic pulse velocity and then correlate the apparent ultrasonic velocity with safe bearing capacity. These soil properties preferably should have significant influence on ultrasonic pulse propagation through soils as well as on safe bearing capacity values of these soils. Such soil properties are its density, void ratio and water content.

Data points indicating variation of in-situ density, in-situ void ratio and in-situ water content of soils with respect to safe bearing capacity values of soils have been plotted in Figures 3.25, 3.26 and 3.27 respectively. Curves of best fit through these data points in all the three figures have also been drawn.

In-situ density, void ratio and water content of a particular soil sample has been linearly combined with ultrasonic pulse velocity through the soil sample to get apparent ultrasonic velocity, $\mathbf{V}_{app}$. Using this method apparent ultrasonic velocity value for all the samples have been determined. Details of the method have been given in next chapter.

Apparent ultrasonic velocity obtained for all the soil samples have been listed in Table 3.15. Data points indicating variation of apparent ultrasonic velocity with respect to safe bearing capacity value of the soil samples have been plotted in Figure 3.28. Curve of best fit through these data points have also been plotted in the same figure. Data points indicating variation of apparent ultrasonic velocity with respect to safe bearing capacity value of the soil samples are indicating improved correlation (correlation coefficient = 0.946). There is little scatter of the data points with respect to best fit curve.

TABLE 3.15 : Results of ultrasonic testing of soils

SBC	T	Time	V	ρ	e	w	V_{app}
52.0	1.7	56.0	303.4	140.3	87.0	60.0	8677.39
64.9	1.7	56.5	300.8	141.1	86.6	59.8	9174.78
64.9	1.7	55.3	307.1	141.7	86.4	57.5	9459.77
65.6	1.7	54.0	314.5	141.3	86.2	57.3	9502.69
70.0	1.7	55.9	304.3	141.6	85.1	56.3	9609.20
70.0	1.7	53.9	315.6	141.7	86.2	56.4	9854.33
73.7	1.7	54.8	310.2	142.1	86.0	54.7	10096.69
77.2	1.7	53.7	316.6	142.3	85.2	58.7	10117.66
76.7	1.7	53.9	315.3	142.2	85.6	54.6	10200.56
72.8	1.7	53.0	320.5	142.2	85.0	55.4	10337.59
79.3	1.7	52.3	325.3	143.0	86.0	54.9	10402.93
79.8	1.7	53.3	319.2	143.3	85.0	55.1	10488.08
77.7	1.7	50.7	335.3	142.4	85.0	55.1	10811.51
80.3	1.7	49.7	342.1	143.8	85.2	53.5	10913.09
80.2	1.7	50.9	333.7	144.7	84.0	55.2	11253.14
84.6	1.7	48.7	348.8	144.2	84.7	53.6	11268.66
84.1	1.7	45.9	340.3	144.8	84.3	54.2	11321.54
82.5	1.7	49.4	344.4	144.5	84.6	53.2	11481.73
84.3	1.7	49.2	345.5	145.3	84.6	53.1	11727.09
88.0	1.7	46.3	367.2	144.3	84.9	53.5	11737.68
87.0	1.7	49.1	346.2	145.6	84.6	52.2	11778.30
88.8	1.7	46.5	365.3	144.4	85.0	50.7	11847.07
85.7	1.7	47.4	358.3	145.8	84.6	54.2	11893.92
88.0	1.7	46.5	365.8	145.4	84.8	52.0	11973.13
91.6	1.7	47.2	360.1	145.6	83.0	48.3	12124.04
94.7	1.7	43.8	388.2	144.6	83.0	51.5	12137.62
92.6	1.7	44.6	380.7	145.2	83.6	48.2	12266.40
96.7	1.7	43.3	392.5	145.1	83.8	49.8	12351.79
96.8	1.7	44.9	378.3	145.7	83.2	49.7	12622.72
93.4	1.7	44.2	384.3	145.0	82.6	49.9	12652.84
92.3	1.7	43.6	389.8	144.8	83.6	48.1	12698.77
97.6	1.7	42.6	398.5	144.5	82.4	47.4	12702.55
98.2	1.7	42.5	399.7	145.7	82.6	49.5	12859.83
96.2	1.7	41.2	412.1	147.0	83.8	50.8	12970.52
99.2	1.7	42.0	404.2	147.1	83.0	49.6	13293.75
98.5	1.7	41.6	408.7	147.0	82.0	49.6	13730.82
99.5	1.7	40.9	416.1	146.9	82.0	47.6	13763.39
107.8	1.7	40.8	416.2	146.9	81.0	45.2	14233.61
111.2	1.7	40.4	420.3	147.4	78.0	41.4	15932.22
120.1	1.7	40.0	425.3	149.0	73.0	42.3	19050.03

SBC=Safe bearing capacity (x 10^{-3})MPa, T=Sample thickness (cm), Time=Pulse transmit time (μsec), ρ=In-situ soil density (x 10^{-2}g/cc), e=In-situ void ratio (x 10^{-2}), V=Pulse velocity (ms^{-1}), w=In-situ water content (x 10^{-1}%), V_{app}=Apparent ultrasonic velocity.

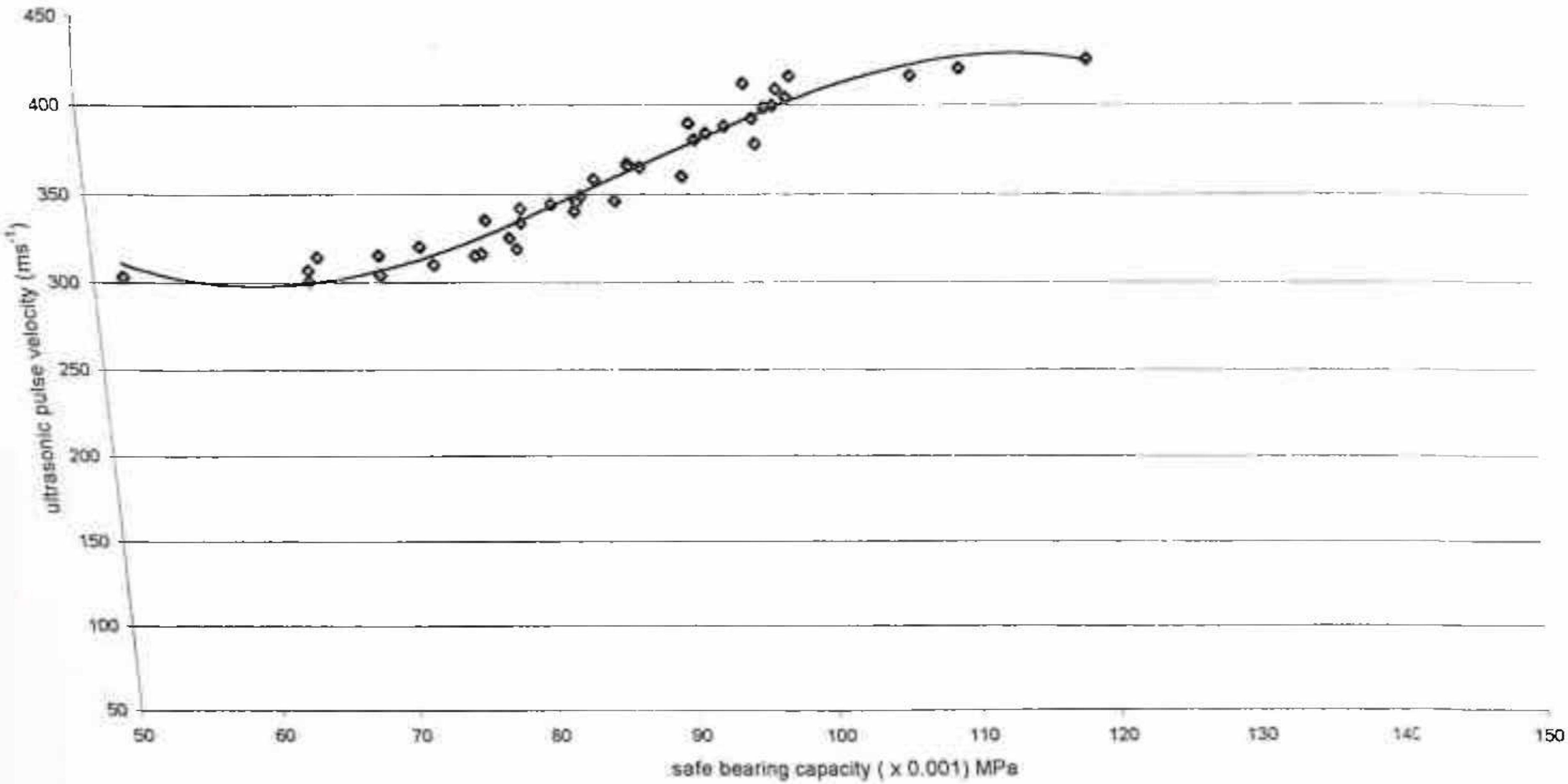

Figure 3.24 : Variation of ultrasonic pulse velocity with soil safe bearing capacity

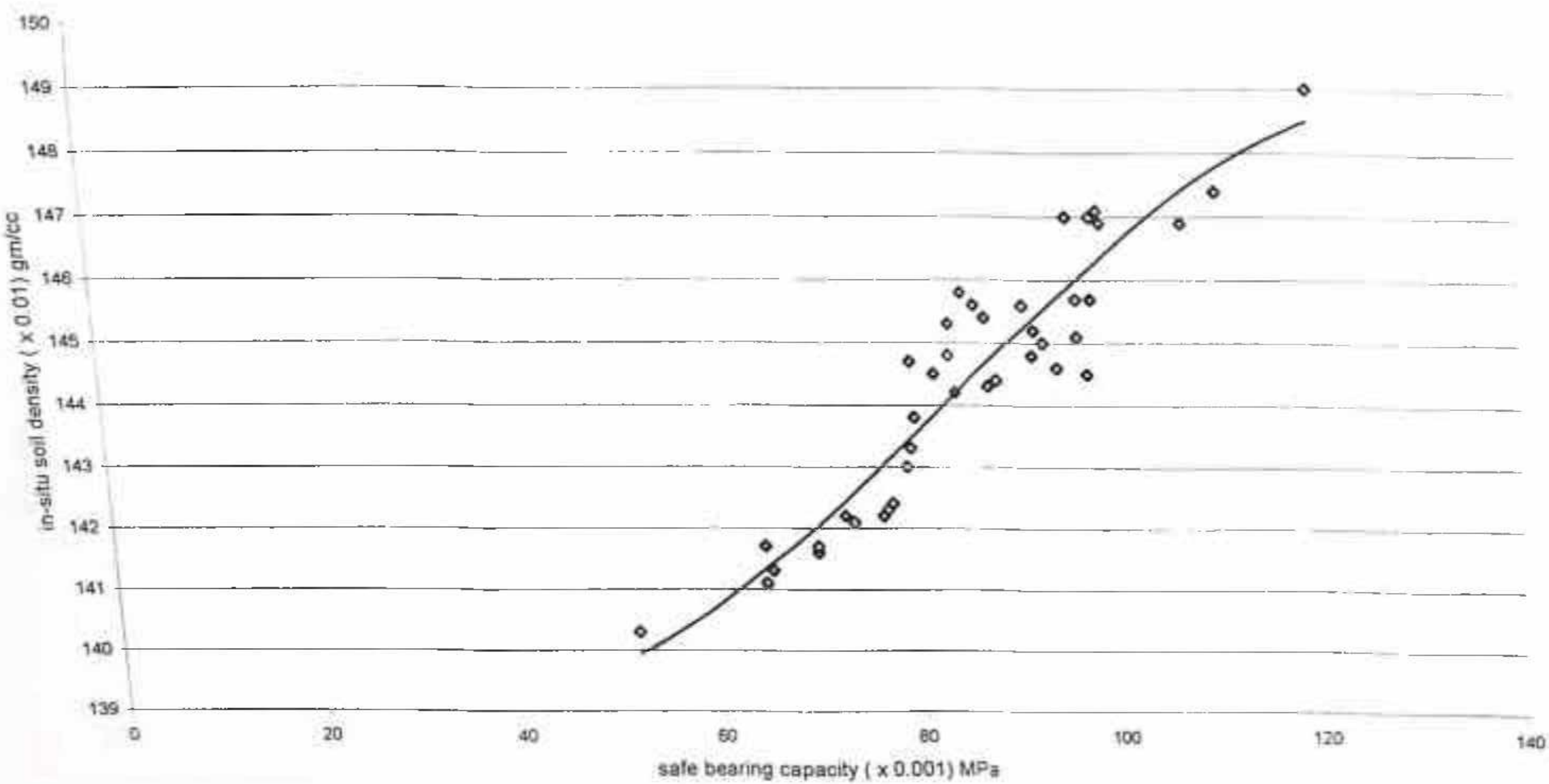

Figure 3.25 : Variation of in-situ soil density with soil safe bearing capacity

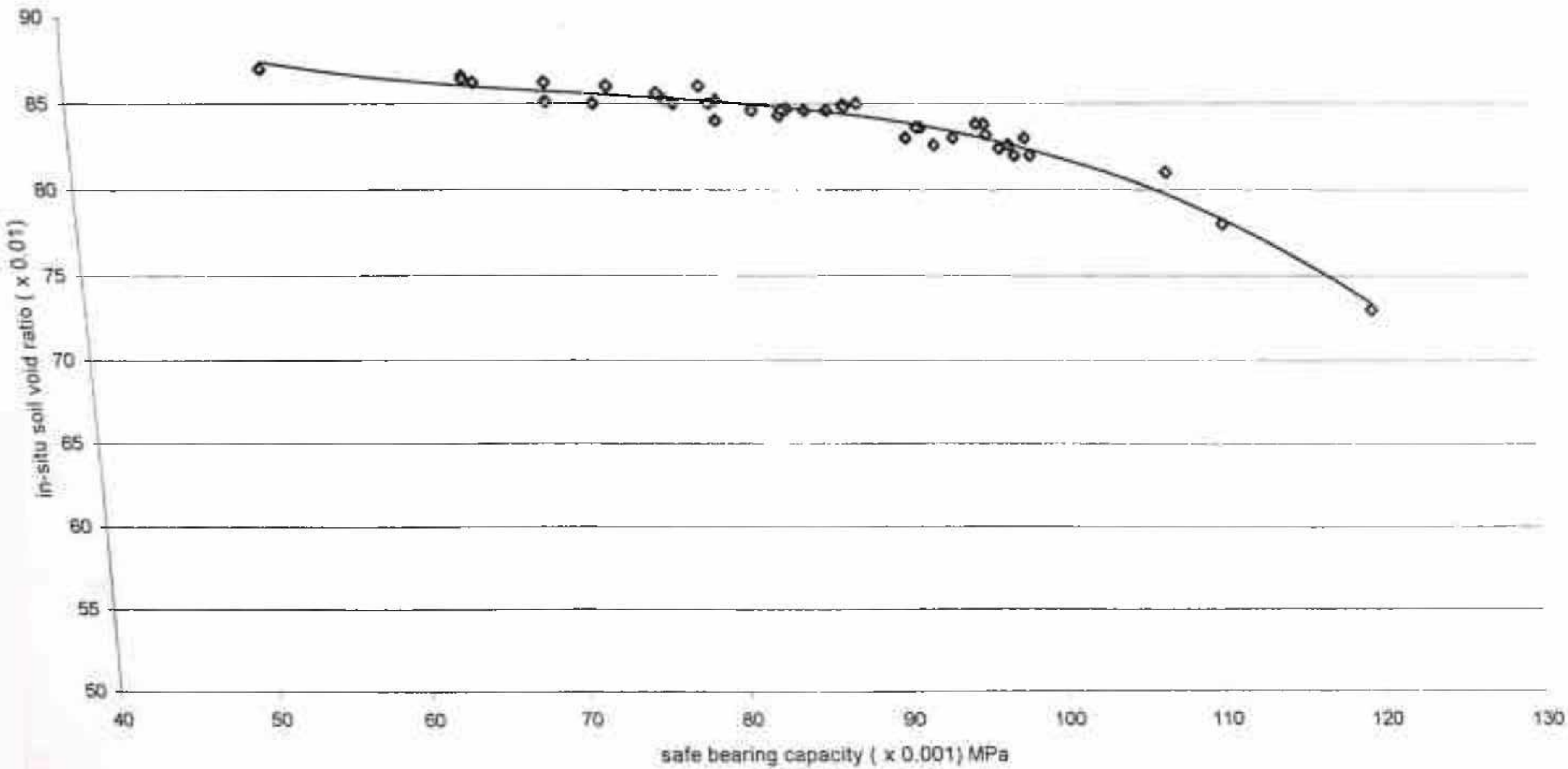

Figure 3.26 : Variation of in-situ soil void ratio with soil safe bearing capacity

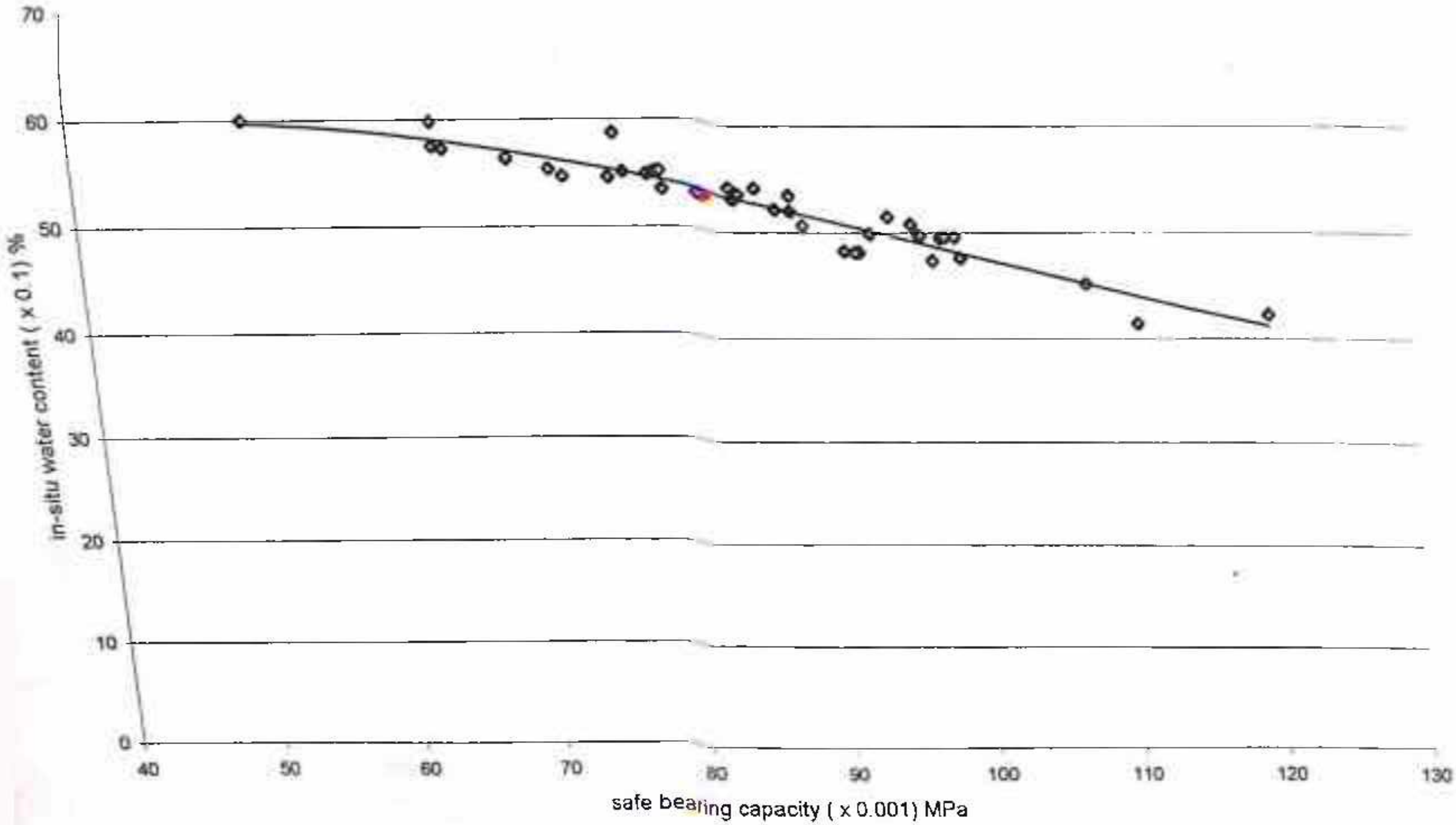

Figure 3.27 : Variation of in-situ water content with soil safe bearing capacity

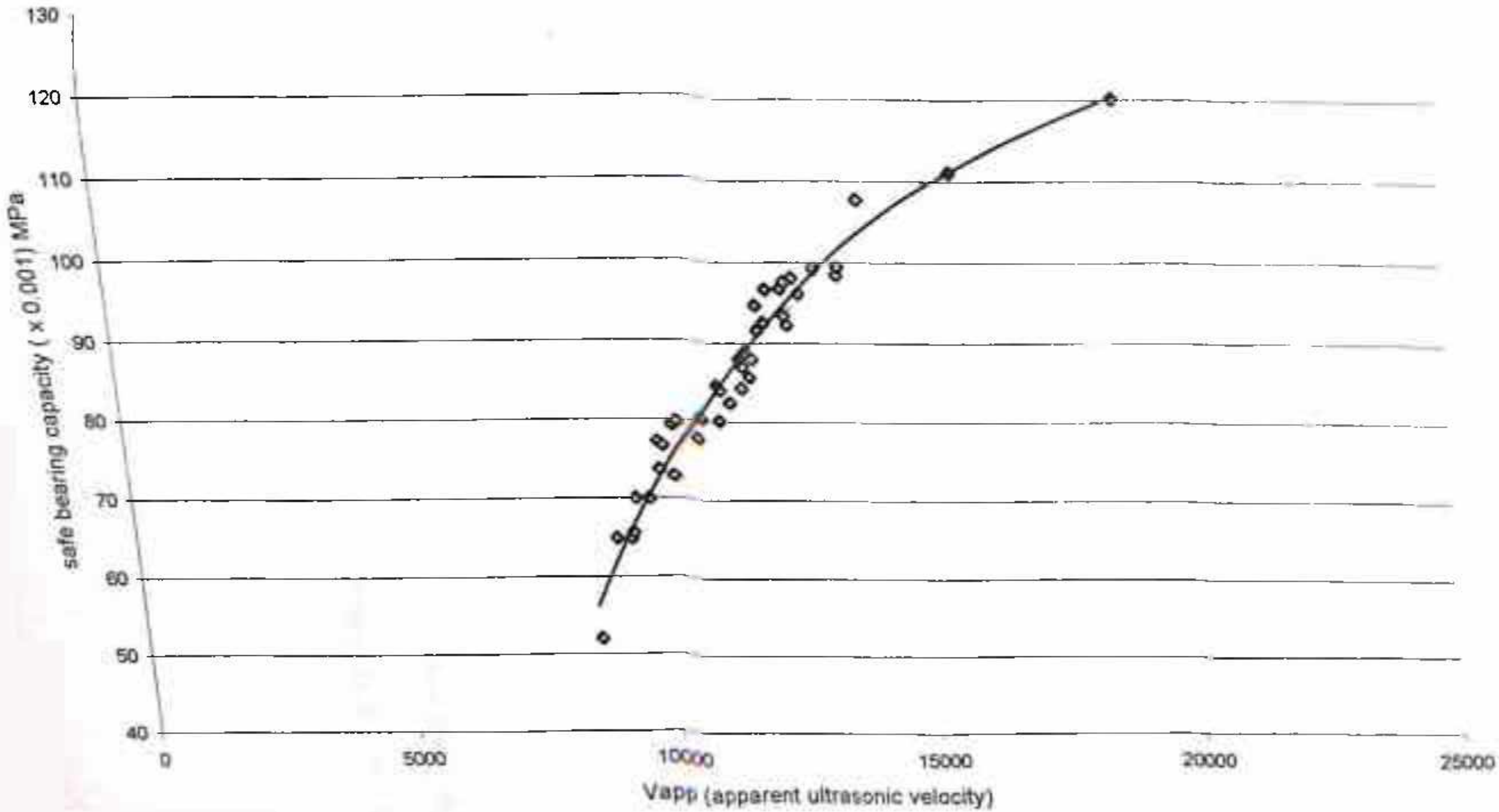

Figure 3.28 : Correlation between apparent ultrasonic velocity and soil safe bearing capacity

4. RESULTS AND DISCUSSION

4.1 SHEAR STRENGTH BEHAVIOR OF SOILS

Particle size composition, pore water content and salt concentration of pore water are important factors which affect cohesion and angle of internal friction and thus govern soil shear strength. Effect of these factors on the aforementioned shear strength parameters was studied in detail. Direct shear testing was conducted to find out the shear strength parameters for varied particle size composition, pore water content and salt concentration.

EFFECT OF PARTICLE SIZE COMPOSITION

In the first set of experiments, particle size composition of the soil was changed. Pore water content was kept constant at 10%. Similarly salt concentration of pore water was kept constant at 0 M. It was observed that:

(a) As weight fraction of sandy soil (retained on 150μ sieve) was reduced from 90% to 30%, cohesion of the soil mix increased from 2.74 x 10^{-3} MPa to 14.4 x 10^{-3} MPa. However angle of internal friction of soil mix decreased from 35^0 to 9^0.

(b) When weight fraction of sandy soil was further reduced from 30% to 10%, cohesion of the soil mix decreased from 14.4×10^{-3} MPa to 11.27×10^{-3} MPa and the angle of internal friction of soil mix increased from 9^0 to 29^0.

Cohesion decides the cohesive strength of soils and angle of internal friction governs the frictional strength of soils.

Short range forces of interaction in the case of coarse grained soil and long range forces of interaction in the case of fine grained soil are the dominant forces of interaction. Short range interactions result in frictional strength and long range interactions result in cohesive strength (Olson, 1962). When coarse grained content (sand retained on 150μ sieve) of soil mix was decreased from 90% to 30%, cohesive strength was found to increase and frictional strength was found to decrease.

Further decrease in coarse grained content (i.e. from 30% to 10%) results in higher percentage of fine grained soil. For such a soil mix, cohesive strength decreases and frictional strength increases. This could be attributed to the fact that as fine grained content increases, it leads to aggregation of fine grained particles resulting in floc formation in the presence of water. Ramanasastry and Rao (1987) have stated that flocculation is one of the main reason for the alteration of soil properties. These flocs act more like coarse grained particle in the soil mix. Consequently coarse grained behavior becomes more significant (i.e. frictional strength increases and cohesive strength decreases).

Hence one may conclude that at coarse grained content of 30% by weight, local soil mix has maximum cohesion. Furthermore angle of internal friction is minimum at this composition. Soil of BITS campus having high cohesive strength and low frictional strength has variety of usage and the same has been discussed in detail in the next chapter.

EFFECT OF PORE WATER CONTENT

In the second set of experiments, pore water content of the local soil mix was changed. Particle size composition of soil mix was kept constant (S_{150} = 50%, C_{75} = 25% and C_p = 25%). Similarly salt content of pore water was also kept constant at 0 M. Pore water content was varied from 0% to 20%. It was observed that:

(a) As water content was increased from 0% to 6%, cohesion of the soil mix increased from 6.2×10^{-3} MPa to 9.407×10^{-3} MPa.

(b) Further increase in water content from 6% to 20% however decreased cohesion of soil mix from 9.407×10^{-3} MPa to 4.93×10^{-3} MPa.

Jennings and Burland (1962) have stated that changes in the water content of a soil can cause changes in soil behavior. Pore water has very little effect on short range forces of interaction. Long range forces however are greatly affected due to amount of pore water present. This happens because in the presence of pore fluid, there is a formation of a membrane like layer around clay particles. This layer causes repulsion between clay particles. On the other hand, Bolt (1956) has mentioned that presence of pore fluid favors hydrogen bonding between clay particles and causes clay particles to stick together. Hence it appears that as water content is increased from 0% to 6%, hydrogen bonding formation is favored. Effect of hydrogen bonding is dominant over the effect of membrane like layer formation for water content in the range of 0% to 6%. Due to hydrogen bonding, sticking tendency of clay particles increases resulting in higher cohesion. Further increase in water content from 6% to 20% results in membrane like layer formation resulting in mutual repulsion between clay particles. Consequently cohesion of the soil decreases. Maximum cohesion was observed at 6% water content for the soil tested.

Angle of internal friction was found to decrease from 35.92^0 to 34.47^0, when water content of soil tested was increased from 0% to 6%. When water content of soil tested was further increased from 6% to 15%, angle of internal friction increased from 34.47^0 to 38.55^0. If the water content is increased even beyond 15%, i.e. from 15% to 20% it results in decreased angle of internal friction of soil tested (i.e. from 38.55^0 to 28.92^0).

Attempts have been made in past to explain the effect of pore water onto frictional behavior of soils containing coarse as well as fine grained particles. However, unanimity in understanding has not been achieved as to what is the exact effect of pore water on frictional behavior of a soil containing coarse as well as fine grained particles. It changes from soil to soil. It also depends on variety of interactions between soil particles (Nagaraj et al, 1991c). In the present study also no definite pattern in variation of angle of internal friction of soil tested was observed when water content was increased from 0% to 20%.

EFFECT OF PORE WATER SALT CONCENTRATION

In the third set of experiments, pore water salt concentration was changed. Particle size composition of soil tested was kept constant (S_{150} = 50%, C_{75} = 25%, C_p = 25%). Similarly water content of soil tested was also kept constant at 10%. Pore water salt concentration was varied from 1M to 4 M. It was observed that:

(a) As pore water salt concentration was increased from 1M to 3 M, cohesion of the soil tested increased from 9.62×10^{-3} MPa to 10.6×10^{-3} MPa. Further increase in pore water salt concentration from 3M to 4 M, however resulted in decreased cohesion of soil tested (i.e. from 10.6×10^{-3} MPa to 10.23×10^{-3} MPa).

(b) As pore water salt concentration was increased from 1M to 2 M, angle of internal friction of soil tested increased from 37.4^0 to 38.36^0. Further increase in pore water salt concentration from 2M to 4 M, however resulted in decreased angle of internal friction of soil tested (i.e. from 38.36^0 to 34.81^0).

When salt concentration of the pore water increases, it causes decrease in thickness of membrane like layer. Decrease in thickness of membrane like layer enhances the effect of long range attractive forces between clay particles. Consequently sticking tendency of the clay particles increases, resulting in increased cohesion. Further increase in pore water salt concentration, however brings negatively charged clay particles too close to each other. Under these conditions clay particles tend to repel each other. Consequently repulsive effect between clay particles becomes dominant resulting in decreased cohesion (Mitchell and Arulanandan, 1968). In the present study, cohesion of soil tested was found to show similar pattern of variation when salt concentration of pore water was increased from 1M to 4 M. Maximum value of cohesion was observed at a pore water salt concentration of 3 M.

As far as effect of pore-water salt concentration on angle of friction is concerned, initial increase in pore water salt concentration brings soil particles closer resulting in increased physical contact between particles. This in turn results in increased frictional strength. Further increase in pore water salt concentration reduces spacing between soil particles even more and at this stage Coulombic repulsion between soil particles becomes more significant forcing soil particles to move apart and the physical contact between the particles decreases. This causes decrease in frictional strength (Seed and Chan, 1959). In the present study, angle of internal friction of soil tested was found to show similar pattern of variation when pore water salt concentration was varied from 1M to 4 M with a maximum value at 2 M salt concentration.

4.2 REINFORCED SOIL CHARACTERISTICS

Effect of increasing weight fraction of glass fibres on cohesion and angle of internal friction of glass fibre reinforced soil composite was studied in detail. Shear strength parameters of cohesion and angle of internal friction were determined by conducting direct shear testing. Gray and Ohashi (1983) have stated that fibre type, fibre length, fibre orientation, fibre diameter and weight fraction of fibres in the composite, are the significant factors which do affect cohesion and angle of internal friction of the composite.

In the present study, fibre type, fibre length, fibre orientation and fibre diameter were kept constant in the glass fibre reinforced soil composite. Weight fraction of glass fibres in the glass fibre reinforced soil composite was the only factor which was varied to observe changes in cohesion and angle of internal friction of the glass fibre reinforced soil composite due to changes in weight fraction of glass fibres. The following observations were made:

(a) As the weight fraction of glass fibres in the glass fibre reinforced soil composite was increased from 0 to 0.12, cohesion of the composite increased from 8.76×10^{-3} MPa to 13.79×10^{-3} MPa.

(b) As the weight fraction of glass fibres in the glass fibre reinforced soil composite was increased from 0 to 0.12, angle of internal friction of the composite decreased from 38.8^0 to 36.87^0.

When the weight fraction of glass fibres in the glass fibre reinforced soil composite was increased beyond 0.12, it was found that even after shearing to the maximum limit of direct shear testing, failure of the composite did not take place even at the lowest possible normal load.

Cohesion arises due to sticking tendency between individual soil particles. Hydrogen bonding and van der Waals bonds are the significant forces causing sticking of soil particles.

Decrease in the thickness of membrane like layer around soil particles also cause soil particles to stick together (Skempton and Northey, 1952).

The physical performance of the fibres in the soil samples and consequently performance of the fibre reinforced soil composite is critically dependent on the physico-chemical micro-structure of the soil samples. Physico-chemical micro-structure of the fibres in the interfacial region, where the fibres and soil make contact, also play significant role in altering physical performance of the fibre reinforced soil composite. Interface between the fibres and soil provides physical as well as chemical bonding between the fibres and soil (Maher and Ho, 1994).

Interface between glass fibres and soil particles are quite surface active. Due to physico-chemical interaction between glass fibres and soil particles at the interface, adhesion develops between the two. Consequently physical bonding between the glass fibres and soil particles increases. Adhesion between glass fibres and soil particles increases with increase in weight fraction of glass fibres in the composite. Adhesion between glass fibres and soil particles also increases with increase in fibre surface roughness and increase in coarse grained fraction of the soil sample (Andrawes et al, 1980).

With increasing weight fraction of glass-fibres in the composite, adhesion between soil particles and glass fibres increases resulting in better physical bonding between the two phases. Apart from better physical bonding, increased adhesion between soil particles and glass fibres results in decrease in thickness of membrane like layer around soil particles. Combined effect of decrease in thickness of membrane like layer around soil particles and van der Waals attractive forces between soil particles due to better adhesion, enhances sticking tendency between soil particles. This results in increase in cohesive strength of the glass fibre reinforced soil composite. In the present study, cohesive strength of glass fibre reinforced soil composite was found to increase with increase in weight fraction of glass fibres.

When glass fibres are present in the soil matrix, the contact area between soil particles gets reduced. Furthermore, if diameter of glass fibres is smaller compared to particle size of soil particles of the soil matrix, glass fibres tend to slip under external load. This causes actual physical contact between soil particles to decrease. Consequently frictional resistance of the glass fibre reinforced soil composite is less (Michalowski and Zhao, 1996). In the present study, glass fibre diameter was smaller than the particle size of the soil tested and therefore this Michalowski and Zhao phenomenon was applicable. In soils without glass fibre reinforcements, frictional resistance is either due to physical contact between the soil particles or due to attractive electrical forces of interaction between soil particles (Matsui et al, 1980).

Due to reduction in contact area between soil particles, there is reduction in attractive electrical forces of interaction between fine grained soil particles as well. This reduction in attractive electrical forces also reduces frictional resistance of the glass fibre reinforced soil composite. Consequently overall effect of increasing weight fraction of glass fibres in the glass fibre reinforced soil composite is decrease in frictional strength. In the present study, angle of internal friction of glass fibre reinforced soil composite decreased with increasing weight fraction of glass fibres.

For studying the strength behavior of fibre reinforced soil, orientation of fibres was kept perpendicular to the failure surface. Strength behavior of fibre reinforced soil with such orientation can most conveniently be studied by conducting direct shear testing. Orientation of fibres perpendicular to the failure surface is the most preferred orientation if fibre reinforced soil is to be subjected to shear stresses. Fibre reinforced soils with orientation of fibres perpendicular to failure surface has variety of practical applications and the same has been discussed in the next chapter. Results of present experimental study on fibre reinforced soil of BITS campus will thus be of use for those varied practical applications.

Strength behavior of fibre reinforced soil is also studied by having two dimensional planer orientation of fibres. Strength behavior of fibre reinforced soil with such orientation of fibres can most conveniently be studied by conducting triaxial test. Two dimensional planer orientation of fibres is the most preferred orientation if fibre reinforced soil is subjected to compressive stresses. Fibre reinforced soil is subjected to such stresses if it is used in pavement design and at the foundation base. Due to certain experimental set-up constraints, triaxial test on fibre reinforced soil samples could not be carried out. Studies related to triaxial test on fibre reinforced soil samples are available in reference such as Ranjan et al (1996) and Kaniraj and Havanagi (2001) etc.

Several investigations have been made on fibre reinforced soils. Brown and Sheu, (1975), Wu et al, (1988a), Wu et al, (1988b), and Wu and Watson (1998) have reported that plant roots act as reinforcement in soil and increase the shear strength of soil. Gray and Ohashi (1983), and Shewbridge and Sitar (1989) have reported the results of laboratory tests on oriented fibre reinforced soils. These studies show that fibres cause significant modification and improvement in engineering properties of soil including its shear strength. Gray and Ohashi (1983) have reported increase in cohesion and nearly constant value of angle of internal friction of fibre reinforced soil with increase in fibre content with perpendicular orientation of fibres in sandy soils. However information on the effect of reinforcing soil (containing sand as well as silty clay) with E-glass fibres with perpendicular orientation of fibres on cohesion and angle of internal friction of fibre reinforced soil composite have not been reported in these studies.

An experimental study was undertaken to study strength deterioration of glass fibre reinforced soil composite with time. For this purpose, four different normal loads were applied on glass fibre reinforced soil composite and corresponding shear strength were determined using direct shear testing apparatus. It was observed that shear strength at a particular normal stress decreased for composite tested one week after fabrication with respect to composite tested soon after fabrication. Similar trends were observed for all the applied normal loads. It was also observed that:

(a) Cohesive strength of the glass fibre reinforced soil composite when tested at the time of composite fabrication was 13.79×10^{-3} MPa. However this strength decreased to 12.88×10^{-3} MPa when composite was tested after a week from the date of fabrication.

(b) Angle of internal friction of the glass fibre reinforced soil composite when tested at the time of composite fabrication was 36.87^0. However the value of angle of internal friction decreased to 36.13^0 when composite was tested after a week of fabrication.

In the literature, it has been stated that properties of E-glass fibres deteriorate with ingression of water. Furthermore it is stated that E-glass fibres degrade faster if soil is alkaline (Derucher et al, 1998). In the present study also E-glass fibre reinforced soil composite indicated loss of cohesive strength and angle of internal friction with time. Reduction in shear strength at a particular normal load with time was observed. The deterioration of properties of composite is due to deterioration in properties of E-glass fibres with time since soil used was alkaline (pH = 8.48) and 10% water was present in the composite. Consequently strength deterioration of E-glass fibre reinforced soil with time puts limitations onto the practical use of such reinforced soils.

4.3 ULTRASONIC TESTING OF SOILS

Propagation of ultrasonic pulses through soil samples depends on soil microstructure. Particle size composition, particle angularity, soil void ratio and soil density are some of the important factors affecting ultrasonic pulse propagation through soils (Desai et al, 1995, Bachrach et al, 2000). In the present study, effect of varying particle size composition of soil onto ultrasonic pulse propagation through the soil has been studied.

EFFECT OF SOIL PARTICLE SIZE COMPOSITION

In the first set of experiments, particle size composition of the soil was changed while determining ultrasonic pulse velocity through soils. Pore water content was kept constant at 10%. It was observed that:

(a) As weight fraction of sandy soil (retained on 150μ sieve) was reduced from 90% to 30%, ultrasonic pulse velocity through soil increased from 340.0 ms^{-1} to 510.5 ms^{-1}.

(b) When weight fraction of sandy soil was further reduced from 30% to 10%, ultrasonic pulse velocity through soil decreased from 510.5 ms^{-1} to 459.4 ms^{-1}.

When an ultrasonic pulse propagates through soil, attenuation of the ultrasonic pulse takes place along the travel path. Scattering of ultrasonic pulse at the microscopic interface of soil particles is an important mechanism for attenuation of ultrasonic pulse (Molyneux and Schmitt, 2000). When particle size of soil increases, scattering of ultrasonic pulses also increases. Consequently an increase in soil particle size results in increased attenuation. An increase in attenuation of ultrasonic pulses through soil, in turn leads to higher transmission time of ultrasonic pulses through soil. Consequently ultrasonic pulse velocity decreases.

In the present study, sand content of soil was reduced from 90% to 30%, i.e. fine-grained content of soil increased from 10% to 70%. This resulted in the decrease of ultrasonic attenuation leading to higher ultrasonic pulse velocity. The observed increase in ultrasonic velocity was from 340 ms^{-1} to 510.5 ms^{-1} corresponding to the said decrease in sand content of soil from 90% to 30%.

If one goes for further reduction in sand content of soil from 30% to 10% i.e. increase in fine grained content from 70% to 90%, it leads to aggregation of fine grained soil particles in the presence of water resulting in floc formation. The flocculated fine grained soil particles behave like coarse grained particles (Kaya and Fang, 2000). Consequently attenuation also increases evident from Figures 3.22 and 3.23. Exact silty clay content of soil can be determined by conducting sieve analysis. Consequently by knowing both the ultrasonic pulse velocity through soil and silty clay content of soil, cohesion of soil can be obtained from Figure 3.22. Similarly by knowing ultrasonic pulse velocity through soil and silty clay content of soil, angle of internal friction of soil can be obtained from Figure 3.23.

Thus depending upon the magnitude of ultrasonic pulse velocity through soil, pulse velocity alone or along with silty clay content of soil can be used to estimate its cohesion and angle of internal friction value. Conventional methods of cohesion and angle of internal friction determination require complicated experimental setup. Calculations required for the determination of cohesion and angle of internal friction are also lengthy. However ultrasonic pulse velocity through soil and silty clay content of soil can be determined more conveniently. Hence estimation of cohesion and angle of internal friction of soil by knowing ultrasonic pulse velocity through it will be a great step forward. Information about cohesion and angle of internal friction of soil is frequently needed when soil is used for different Civil engineering applications.

Figures 3.22 and 3.23 have been prepared by collecting soils from two different locations close to BITS campus. At one location soil was predominantly sandy. Similarly at the second location soil was predominantly silty clay. However at other locations of BITS campus, soil typically will contain sand as well as silty clay fraction with silty clay content between 10% and 90%. Consequently Figures 3.22 and 3.23 can be used to estimate cohesion and angle of internal friction of soil by knowing ultrasonic pulse velocity through the soil.

ESTIMATION OF SAFE BEARING CAPACITY

In the present experimental study, an attempt has been made to estimate safe bearing capacity of soils by knowing ultrasonic pulse velocity through the soils. Suggested method requires information about ultrasonic pulse velocity through soils and certain soil properties to estimate safe bearing capacity. All these parameters can be obtained by conducting simple experiments. This method is simpler compared to conventional method of safe bearing capacity determination which requires information about too many parameters and calculations involved are lengthy.

Soil parameters such as density, water content, void ratio etc. affect the safe bearing capacity of the soil (Ingra and Baecher, 1983 and Zadroga, 1991). The same properties affect pulse velocity too (Robertson et al 1995 and Koerner et al 1976). Consequently ultrasonic transmission velocity is expected to show some kind of correlation with safe bearing capacity of soil. The variation of safe bearing capacity with ultrasonic pulse velocity is shown in Figure 3.24 on page 95. Data point for each soil tested has been plotted. Best fit curve through these experimental data points have been drawn. This curve can be treated as calibration curve and may be used for measuring safe bearing capacity of soil after measuring the pulse velocity through the soil.

As far as scatter of data points with respect to best fit curve in Fig. 3.24 is concerned, it can be minimized by linearly combining certain soil properties with ultrasonic pulse velocity. We may term it as *apparant ultrasonic pulse velocity* $\mathbf{V_{app}}$. Parameter $\mathbf{V_{app}}$ when correlated with safe bearing capacity, a better correlation is obtained.

Apparent ultrasonic velocity V_{app} in the present study was taken in the following mathematical form:

$$V_{app} = aV + b\rho + ce + dw \tag{4.1}$$

Where,

V = Ultrasonic pulse velocity (ms^{-1}).

ρ = In-situ soil density (x 10^{-2} gm/cm^3).

e = In-situ void ratio of soil (x 10^{-2}).

w = In-situ water content of soil (x 10^{-1}%).

a, b, c and d are constants.

Following simple method was used to determine the unknown constants of equation (4.1). Data points indicating variation of in-situ density, in-situ void ratio and in-situ water content of soils with respect to safe bearing capacity of soils were plotted in Figures 3.25, 3.26 and 3.27 on pages 96, 97 and 98 respectively. Dependence of safe bearing capacity of soils on aforementioned soil properties is evident from these figures. Curve of best fit through these data points have also been drawn in the Figures 3.25, 3.26 and 3.27. To obtain these curves, soil properties (i.e. in-situ density, in-situ void ratio and in-situ water content) have been taken as cubic functions of safe bearing capacity. A cubic function has four constants. Same number of constants are required in equation (4.1). Consequently cubic functions were used in the present study to get best fit curve in Figures 3.25, 3.26 and 3.27. Since simple method has been used to obtain apparent ultrasonic velocity V_{app}, as a linear combination of ultrasonic pulse velocity and soil properties, multi-variable functional relation between safe bearing capacity and soil properties were not pursued.

Following is the mathematical form of aforementioned cubic equations. These equations were obtained using Microsoft Excel (Person, 1998).

$$V = -0.002(SBC)^3 + 0.5244(SBC)^2 - 41.46(SBC) + 1336.3 \tag{4.2}$$

$$\rho = -0.00002(SBC)^3 + 0.0056(SBC)^2 - 0.319(SBC) + 144.46 \tag{4.3}$$

$$e = -0.00009(SBC)^3 + 0.0191(SBC)^2 - 1.4357(SBC) + 122.77 \tag{4.4}$$

$$w = 0.0006(SBC)^3 - 0.1446(SBC)^2 + 10.555(SBC) - 189.59 \tag{4.5}$$

Parameters **V**, **ρ**, **e** and **w** have already been described in connection with equation (4.1). **SBC** indicates safe bearing capacity of soil in equations (4.2), (4.3), (4.4) and (4.5). Based on several iterations, the following cubic functional relation between safe bearing capacity (**SBC**) and $\mathbf{V_{app}}$ was tried:

$$V_{app} = -0.0011(SBC)^3 + 0.3014(SBC)^2 - 24.544(SBC) + 900 \tag{4.6}$$

Also, from equations (4.1), (4.2), (4.3), (4.4) and (4.5):

$$
\begin{aligned}
V_{app} &= aV + b\rho + ce + dw \\
&= a[-0.002(SBC)^3 + 0.5244(SBC)^2 - 41.46(SBC) + 1336.3] + \\
&\quad b[-0.00002(SBC)^3 + 0.0056(SBC)^2 - 0.3196(SBC) + 144.46] + \\
&\quad c[-0.00009(SBC)^3 + 0.0191(SBC)^2 - 1.4357(SBC) + 122.77] + \\
&\quad d[0.0006(SBC)^3 - 0.1446(SBC)^2 + 10.555(SBC) - 189.59]
\end{aligned} \tag{4.7}
$$

Collecting coefficients of terms $(SBC)^3$, $(SBC)^2$, (SBC) and constant term of equation (4.7), equation (4.7) can be rewritten as:

$$
\begin{aligned}
V_{app} = &[(-0.002)a - (0.00002)b - (0.00009)c + (0.0006)d](SBC)^3 + \\
&[(0.5244)a + (0.0056)b + (0.0191)c - (0.1446)d](SBC)^2 + \\
&[(-41.46)a - (0.319)b - (1.4357)c + (10.555)d](SBC) + \\
&[(1336.3)a + (144.46)b + (122.77)c - (189.59)d]
\end{aligned} \tag{4.8}
$$

Equating coefficients of $(SBC)^3$, $(SBC)^2$, (SBC) and constant term of equations (4.6) and (4.8), following relations are obtained:

$$(-0.002)a - (0.00002)b - (0.00009)c + (0.0006)d = -0.0011 \tag{4.9}$$

$$(0.5244)a + (0.0056)b + (0.0191)c - (0.1446)d = 0.3014 \tag{4.10}$$

$$(-41.46)a - (0.319)b - (1.4357)c + (10.555)d = -24.544 \tag{4.11}$$

$$(1336.3)a + (144.46)b + (122.77)c - (189.59)d = 900 \tag{4.12}$$

Equations (4.9) to (4.12) are a system of four linear equations with four unknowns. Solving these equations for **a**, **b**, **c** and **d**, we get: $a = 0.7285$, $b = 372.4$, $c = -503.6$ and $d = 0.364$.

Solution of equations (4.9) to (4.12) was obtained using Matlab (Pratap, 1999). Also as equations (4.9) to (4.12) are four independent linear equations, values of **a, b, c,** and **d** are unique and stable.

Aforementioned constants **a, b, c,** and **d** can be put in equation (4.1) to get the following:

$$V_{app} = 0.7285V + 372.4\rho - 503.6e + 0.364w \tag{4.13}$$

Equation (4.13) has been used to calculate apparent ultrasonic velocity ($\mathbf{V_{app}}$) for all the 40 soil samples studied. These values are given in Table 3.15 on page 94. Data points indicating variation of safe bearing capacity with $\mathbf{V_{app}}$ have also been plotted in Figure 3.28 on page 99. Curve of best fit through these data points have also been shown in Figure 3.28.

By comparing Figure 3.24 on page 95 and Figure 3.28 on page 99, it can be concluded that there is considerable reduction in scattering of data points with respect to best fit curve. Consequently best fit curve of Figure 3.28 provides better estimation of safe bearing capacity. $\mathbf{V_{app}}$ being an improvement on **V**, it was bound to provide a better fit with less scatter, as is evident from Fig. 3.28.

5. CONCLUSIONS

Soil is used for variety of Civil engineering applications. Soil and foundation engineers are interested in knowing about important engineering properties of soils. Information about these soil properties helps in selecting appropriate type of soil for a particular engineering application. In the present experimental study, shear strength behavior of soil has been studied in detail. Results reported in the present study are based on experimental observations on soil samples collected from locations close to BITS Pilani campus.

Origin of shear strength in soil has been explained at the particle level. Different types of interactions between soil particles resulting in soil shear strength have been discussed. Related literature review for interactions between soil particles has been provided.

Based on review of literature, it may be concluded that shear strength in soils has two components, viz. the cohesive component and the frictional component. These are called shear strength parameters of the soil. Different types of interparticle interactions between soil particles are responsible for these shear strength parameters. Features such as composition of soil, pore water content, in-situ salt concentration in the soil are some of the important factors which affect shear strength of soil. Literature study was also done to explain shear strength behavior of saturated uncemented, saturated cemented and partially saturated soils.

Thus the suggested experimental method in which we use V_{app} instead of V, involves determination of ultrasonic velocity, in-situ density, void ratio and water content. These four parameters can easily be obtained through simple experimental set-ups to get the value of V_{app}. Using V_{app}, value of safe bearing capacity can directly be obtained from Figure 3.28.

The results of present study in the form of Figure 3.28, may be used for obtaining safe bearing capacity of other soil samples. Soil samples from other regions will have different ultrasonic velocity, density, void ratio and water content resulting into different values of V_{app}. For these V_{app} values, safe bearing capacity can easily be obtained from the calibration curve. Consequently, ultrasonic technique can effectively be used for estimation of safe bearing capacity of soils.

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Based on review of literature, it may be concluded that shear strength in soils has two components, viz. the cohesive component and the frictional component. These are called shear strength parameters of the soil. Different types of interparticle interactions between soil particles are responsible for these shear strength parameters. Features such as composition of soil, pore water content, in-situ salt concentration in the soil are some of the important factors which affect shear strength of soil. Literature study was also done to explain shear strength behavior of saturated uncemented, saturated cemented and partially saturated soils.

In the present study, effect of soil particle size composition, pore water content and salt concentration of pore water on to cohesive and frictional strength of soil has been studied. Results of the experimental study have been explained in terms of interparticle interactions between soil particles.

When particle size composition of the soil was changed, it was found that the soil having 30% (by weight) of coarse grained particles and 70% (by weight) of fine grained particles had the maximum cohesion. Furthermore angle of internal friction of soil was minimum at this composition. Consequently a soil mixture having 70% fine grained particles and 30% coarse grained particles should be used for construction of low cost housing, earthen embankments, earthen dams, earthen slopes, foundation base, earthen pavements, as well as for subgrade material in the highway construction. As is well known, in all aforementioned civil engineering applications, high cohesive strength of soil is required. Furthermore since frictional strength of soil was found to be minimum at this particle size composition, it will help in prevention of crack formation in soil when it is used for these applications.

When pore water content of soil was changed, it was found that at 6% water content, soil had maximum cohesion. Consequently 6% pore water content should be provided in soil. Angle of internal friction of soil at this water content was also not very high. Consequently possibility of crack formation in soil will be small at this level of pore water content.

When pore water salt concentration of soil was changed, it was found that at 3M pore water salt concentration, soil had maximum cohesion. Consequently required amount of salt should be added in the pore water to get this concentration. This will help in further enhancing cohesive strength of soil, which is desirable. Angle of internal friction of soil was found to be maximum at 2M concentration. At 3M concentration, angle of internal friction of soil was less. This is also desirable

Based on experimental studies on soil samples collected from locations close to BITS Pilani campus, one may conclude that the soil having 70% fine grained particles, 30% coarse grained particles and 6% pore water content should be used for different civil engineering applications. Interestingly most of the soil samples of the present study had fine grained particles in the range of 65 to 75%. Furthermore, in-situ water content of most of the soil samples collected from this area is in between 4 to 5%. These in-situ values are very close to optimum particle size composition and pore water content established by the present experimental study. Hence it can be concluded that soil in its natural state may be used for different applications. If 3M salt concentration in pore water is provided, it will further enhance soil property for its varied applications. If required, soil can be compacted at the aforementioned composition and then used for different civil engineering applications.

Experimental studies were also conducted on fibre reinforced soils. Effect of reinforcement of the soil using glass fibres onto cohesive strength of soil and on angle of internal friction value were studied. Local soil was used as experimental matrix material and E-glass fibres were used as reinforcing agents.

In the present experimental study, orientation of E-glass fibres was kept perpendicular to the failure surface within the soil mass. With increasing weight fraction of E-glass fibres in the composite from 0.02 to 0.12, cohesion of the reinforced soil was found to increase. However, angle of internal friction of the composite was found to decrease with increasing weight fraction of fibres.

Since E-glass fibres were found to increase cohesive strength of soil, they can be used in applications such as earthwalls, embankments, dams, slopes, foundation beds etc. because a high cohesive strength is desired for all these applications. In all these applications, a prominent failure surface is present within soil and the same can be located using standard techniques. E-glass fibres can be oriented perpendicular to this failure surface to inhibit the crack propagation within the soil.

Increasing weight fraction of E-glass fibres in the soil resulted in decrease of angle of internal friction. This is also a desirable factor because this helps in minimum crack formation in the reinforced soil. However with increasing time duration, cohesion and angle of internal friction of fibre reinforced soil were found to decrease. Decrease in shear strength of fibre reinforced soil, at a particular normal load, with time was also observed. Aforementioned behavior of fibre reinforced soil has been explained in literature wherein it is stated that properties of E-glass fibres deteriorate in alkaline soil under the presence of moisture. Similar conditions were present in the present study too and one may conclude that E-glass fibres will have limited application as far as reinforcement of moist alkaline soil is concerned.

Use of non-destructive testing (NDT) techniques is finding increasing applications for assessing the quality of materials including quality of soils. These testing techniques are very useful because they provide the desired information about the properties of the material without impairing their future usefulness. Ultrasonic testing is one such non-destructive testing (NDT) technique and is used for testing materials of civil engineering importance (e.g. concrete, wood, brick etc.).

In the present study, ultrasonic testing was performed on soil samples. Through transmission technique was used to find the ultrasonic pulse velocity through soil samples and the effect of particle size composition onto ultrasonic velocity was studied. This was correlated to the effect of particle size on cohesive strength and angle of internal friction. A calibration curve was obtained for estimating cohesion of soil sample by just measuring the ultrasonic velocity through soil. Calibration curve was also obtained for estimating angle of internal friction by knowing the ultrasonic velocity through the soil.

Cohesion and angle of internal friction are important soil properties. These properties of soil are frequently needed whenever soil is to be used for different civil engineering applications. Conventional method of cohesion and angle of internal friction determination require complicated experimental set-up, whereas measuring ultrasonic pulse velocity through soil samples requires only a simple set-up and therefore the latter appears to be a better option.

The calibration curves were obtained for soil samples having wide range of particle size composition (10% silty clay to 90% silty clay). Soil in the natural state in general will have particle size composition in the aforementioned range. Consequently calibration curves of present study can be used to estimate cohesion and angle of internal friction of soils in its natural state.

In the present study, calibration curve was also obtained for estimating safe bearing capacity of soils by measuring the ultrasonic velocity. For plotting the calibration curve, experimental analysis of soil samples was undertaken in usual manner. Safe bearing capacity is an important soil property and the conventional methods of safe bearing capacity determination are rather complicated. Ultrasonic pulse velocity measurement, howevere, is quite simple and therefore a welcome alternative for assessing safe bearing capacity of soils.

Calibration curve for estimating safe bearing capacity of soils from ultrasonic pulse velocity however was associated with scatter of data points with respect to the best fit curve. To minimize this scatter, a new parameter V_{app} (apparent ultrasonic velocity) was introduced. This parameter was taken as a linear combination of ultrasonic pulse velocity, its in-situ density, its water content and the void ratio. A new calibration curve was plotted for estimating safe bearing capacity of soil by knowing V_{app}, i.e. apparent ultrasonic pulse velocity. This calibration curve was found to have much reduced scatter of data points with respect to the best fit curve. Consequently, the new calibration curve would provide a better estimate of safe bearing capacity. All the parameters nedded for the determination of apparent ultrasonic velocity can be determined by performing simple experiments.

Finally, it may be concluded that by measuring ultrasonic pulse velocity through soils, one can effectively estimate its cohesive strength, angle of internal friction and its safe bearing capacity nondestructively.

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7. PUBLICATIONS OUT OF PRESENT WORK

JOURNAL PAPERS :

1. *Paper entitled "Soil Bearing Capacity Determination Using Ultrasonics" got published in the "Journal of Pure and Applied Ultrasonics" (Year 2000, Vol. 22, No. 3, pp 70-77)*

2. Paper entitled "Pore Fluid Effect on Shear strength of Pilani Soil" has been accepted for publication in the "Journal of Institution of Engineers (India) Civil Engineering Division".

CONFERENCE PAPERS :

1. Paper entitled "Use of Ultrasonics to Estimate Soil Bearing Capacity" got accepted for poster paper presentation at the International Conference and Exhibition of Ultrasonics (ICEU-99) held at New Delhi (2nd Dec. – 4th Dec. 1999).

2. Paper entitled "Correlation Between Strength Parameters and Ultrasonic Velocity of Soils" got accepted for oral presentation at the National Seminar 'NDT Voyager in 2000s' (NDE-99) held at Vadodara (17th Dec. – 19th Dec. 1999).

3. Paper entitled "Interrelation Between Ultrasonic Pulse Velocity, Density and Bearing Capacity of Pilani Soil" got accepted for oral presentation at the 11th National Symposium on Ultrasonics to be held at Anantapur (10th Jan. – 11th Jan. 2002).