Evaluation of mechanical behavior in relation to index properties of different soils in Pakistan

By

Muhammad Sabtain Raza Khan

NUST- 2018-MS-Geotech-00000276278

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Geotechnical Engineering

NUST Institute of Civil Engineering (NICE)

School of Civil and Environmental Engineering (SCEE)

National University of Sciences and Technology (NUST)

H-12 Sector, Islamabad, Pakistan

2021

By

Muhammad Sabtain Raza Khan

NUST-2018-MS-Geotech-00000276278

A Thesis submitted in partial fulfillment of
the requirements for the degree of

Master of Science

In

Geotechnical Engineering

Department of Geotechnical Engineering

NUST Institute of Civil Engineering (NICE)

School of Civil and Environmental Engineering (SCEE)

National University of Sciences and Technology (NUST)

H-12 Sector, Islamabad, Pakistan

2021

PLAGIARISM DECLARATION

i. I know the meaning of plagiarism and declare that all the work in the document, save for that which is properly acknowledged, is my own. This thesis/dissertation has been submitted to the Turnitin module (or equivalent similarity and originality checking software) and I confirm that my supervisor has seen my report and any concerns revealed by such have been resolved with my supervisor.

ii. I have used the NUST Synopsis and Thesis Manual as Author-date-referencing-guide based on the APA convention for citation and referencing. Each significant contribution and quotation in this dissertation from other work /research has been attributed and has been cited and referenced, accordingly.

iii. This dissertation is my own work.

iv. I have not allowed and will not allow anyone to copy my work with the intention of passing it as his or her own.

Signature: ____________________________ Date:

Student Name: **Muhammad Sabtain Raza Khan**

THESIS ACCEPTANCE CERTIFICATE

It is certified that final copy of MS thesis written by **Mr. MUHAMMAD SABTAIN RAZA KHAN (Registration No. NUST-2018-MS-Geotech-00000276278)** of **NUST INSTITUTE OF CIVILENGINEERING(NICE)**, has been vetted by undersigned, founded complete in all respects as per NUST Statutes / Regulations, free of plagiarism, errors, and mistakes and is accepted as partial fulfillment for the award of MS degree. It is further certified that necessary amendments as pointed out by GEC members of the scholar have also been incorporated in the said thesis.

Signature: ______________________________

Name of Supervisor: **Dr. Tariq Mahmood Bajwa**

Date: ______________________________

Signature **(HOD)**: _________________________

Date: ______________________________

Signature **(Dean/Principal)**: __________________

Date: ______________________________

This is to certify that the

Thesis titled

Evaluation of mechanical behavior in relation to index properties of different soils in Pakistan

Submitted by

Muhammad Sabtain Raza Khan

has been accepted towards the partial fulfillment

of

the requirements

for

Master of Science in Geotechnical Engineering

____

Dr. Tariq Mahmood Bajwa

(Thesis Supervisor)

Assistant Professor

NUST Institute of Civil Engineering (NICE)

DEDICATED

TO

HAZRAT SYEDA FATIMAH-AL-ZAHRA (A.S)

&

HAZRAT IMAM MUHAMMAD IBN AL- HASAN AL- MEHDI (A.S)

ACKNOWLEDGEMENT

.

All thanks and praises to ALMIGHTY ALLAH who gave the patience and perseverance and enabled me to complete my master's degree. Countless salutations upon Holy Prophet (P.B.U.H), the source of knowledge and guidance for mankind in every walk of life.

I am in debt of gratitude to my research supervisor **Dr. Tariq Mahmood Bajwa** for his guidance and encouragement at every step of this journey. He has been a ray of hope and a being seek council for every problem life has to offer at this point in time as he is a the real meaning of the teacher, a spiritual father. His motivation, appreciation and time made this research possible. I am grateful to my guidance committee members and teachers for their heartfelt supervision. I am thankful to my friends for being the best community around. Their love, respect, cheerfulness and intellect were my support and lead. I would like to thank Muhammad Waleed, Muhammad Umair for being there when I was in need of strong spirits. I am obliged for their contribution in this research as well as other happenings.

Last but not least; I would like to pay earnest gratitude to my family, my parents and my siblings for their absolute support, encouragements, prayers and fortitude. Thank you mom, dad, Sehrish Afzal, Due Zainab Raza and Hammad Raza for being the best the world has to offer. You all are my life and will be the most beautiful thing ever.

Table of Contents

List of Figures

List of Tables

ABSTRACT

The variations in mechanical behavior of a soil over time play a key role in the design of all civil infrastructure projects, including highways, runways, canal linings, landslides, earthen dams, retaining walls and foundations, and for a particular structure, it is pre-requisite to precisely simulate mechanical behavior of used soils with respect to project specific conditions. It is well known that a particular soil behaves in a different fashion than others, under similar loading conditions due to differences in their index properties, such as specific gravity, particle size distribution, consistency limits and dry unit weight. Though research studies are available in literature in this regard, reporting the mechanical behavior in relation to index properties of different soils, but the scope of these studies is quite limited as they are mainly focused on few soil types or only consider few parameters of practical interest, and furthermore, no research of this type has been conducted in the context of Pakistan so far, specifically. The objective of this study is to investigate the mechanical behavior in relation to index properties for several different soils, got from various zones of Pakistan, and regarding this, several triaxial, California Bearing Ratio (CBR), Oedometer and index properties tests were performed to estimate the compressibility parameters, strength parameters and other geotechnical properties to develop some useful empirical relationships for these soils. The test data shows that the shear strength, compressibility and CBR change with %age changes in clay contents. The highly plastic soil (Nandipur soil) provides maximum dry density at lower peak, and at higher water contents than other soils. The sandy soils, such as ML, SC and SP need less water to reach at maximum dry density. The maximum dry density decreases and optimum moisture content increases with an increase in plasticity index, providing strong relationships. The compression and swelling indices show strong relationships with Atterberg limits, %age clay contents, and optimum moisture contents. The

triaxial tests show that cohesion and angle of internal friction provide strong inverse relationship with %age clay contents, and furthermore, CBR shows strong inverse relationship with specific gravity, Atterberg limit, optimum moisture content, and cohesion and strong direct relationships with maximum dry density and angle of internal friction.

CHAPTER 1

1. INTRODUCTION

1.1. General

The mechanical behavior of a soil plays a crucial role in the design of highways, runways, canal linings, landslides, earthen dams, retaining walls and foundations. For a particular structure, the mechanical behavior of a soil needs to be defined with respect to site specific conditions. It is quite ascertained that a soil behaves in a different fashion at one location in comparison to other due to variations in their index properties. The mechanical behavior of different soils varies at different locations due to variations in geological factors such as soil composition, mineralogy, weathering, and climate change etc. It is imperative that a geotechnical engineer always relies upon soils with greater strength to design an infrastructure, economically.

The index properties including specific gravity, particle size distribution, consistency limits, and dry density play an important role in defining the mechanical behavior of a soil. Various studies are available in the literatures which correlate shear strength and California bearing ratio (CBR) with index properties of different soils, following analytical and experimental approaches. (Yılmaz, 2000) carried out regression analysis to correlate undrained shear strength with liquidity index of different clayey soils, got from various locations of Turkey and reported a mathematical expression to estimate the undrained shear strength from the liquidity index. (Obasi and Anyaegbunam, 2005) performed experimental work and regression analysis on different tropical clays to establish relationships between undrained shear strength and plasticity index and reported that a good correlation was found between shear strength and plasticity index. (Dolinar, 2010) studied the normalized undrained shear strength in relation to plasticity index, considering the variations in mineralogical properties of fine-grained soils, however, the study was lack in providing a uniform criterion to determine the normalized undrained shear strength from plasticity index of these soils. (Rashmi and Desai, 2010) correlated CBR with Atterberg limits and optimum moisture content of different clayey soils, got from various zones of Gujrat, India and reported that CBR decreased with an increase in plasticity index. (Shirur and Hiremath, 2014) evaluated CBR in relation to Atterberg limits and maximum dry density of different clayey soils using single and multiple linear regression analysis and concluded that CBR was inversely proportional to plasticity index, and it varied linearly with maximum dry density.

Furthermore, the study reported that the Atterberg limits showed insignificant influence on CBR value. (Talukdar, 2014) correlated soaked CBR value with MDD, OMC, LL, PL, and PI of soil samples collected from different locations of flood prone state Assam, India, and concluded that CBR decreased with an increase in PI and optimum moisture content of soil. (Rakaraddi, Gomarsi, 2015) examined the soaked CBR in connection to liquid limit, plastic limit, plasticity index, optimum moisture content, maximum dry density, and percentage fineness of different soils, collected from different areas of Bagalkot district, India. The test data showed that liquid limit was greatly linked to soaked CBR, considering the assessment factor R^2. (Rehman et al., 2017) examined 25 different samples to correlate CBR with consistency limits using different empirical relationships and reported that CBR showed an inverse relationship with %age finer, optimum moisture content and consistency limits, and furthermore, the relationship between CBR and maximum dry density was linear. (Khalkhali and Mirghasemi, 2009) examined the behavior of coarse-grained soils with numerical and experimental data sets performing direct shear tests and concluded that the angle of internal friction and the sample’s dilation was increased with increase in the particle size, while a reduction in apparent cohesion was noticed. (Dafalla, 2013) performed direct shear tests to examine the influences of changes in clay and moisture content on clayey sand mixture and concluded that cohesion increased with %ages increase in clay size particles, and the angle of internal friction and cohesion reduced with an increase in moisture content. Similarly, (Alias et al., 2014) studied the effects of particle size on shear strength of granular materials performing direct shear tests and reported that larger size particles produced higher effective angle of internal friction, which ultimately resulted in higher shear strength.

So, studies are available in the literature reporting the mechanical behavior in relation to index properties of different soils, but these studies were mainly focused on California bearing ratio (CBR) and undrained shear strength. So, in this study, the authors follow a comprehensive experimental and analytical approach to examine the mechanical behavior in connection to index properties of several different soils. The soil specimens with different basic geotechnical properties were collected from various zones of Pakistan. Several triaxial compression, California bearing ratio (CBR) and oedometer tests were performed to investigate the mechanical behavior of these soils, and the index properties are determined performing Atterberg limits, grain size distribution, specific gravity, and standard Proctor tests.

The test data showed that the shear strength, compressibility and CBR changed with %age changes in clay contents, and the influences of change were quite higher towards lower %age of clay contents than higher %ages. It was further noticed that the mechanical behavior is a true function of index properties in most cases. However, in few cases, the empirical relationships showed fair relationships for mechanical behavior in relation to index properties.

1.2. Justification of the research work

The research study is very useful in the context of Pakistan as it investigates the mechanical behavior in relation to index properties for several different soils and reports some interesting relationships, useful for engineers and scientists to estimate the required parameters on the basis of pre-defined criteria in this study. Furthermore, no such type of study has been executed in the region so far, even on a smaller scale.

1.3. Research objectives

The objective of this research is as:

- To investigate the mechanical behavior in relation to index properties of different soils in Pakistan.

1.4. Thesis outlines

Here is the breakdown of the thesis, highlighting the summary for each chapter.

Chapter 1 reports the general overview highlighting problem statement, study justification and objectives of the work.

Chapter 2 reports reviews to proceed this work on the basis of concepts in already published work.

Chapter 3 presents materials and methods used in the research work to attain the set objectives.

Chapter 4 formulates the test results and reports some useful discussions on the basis of study test data.

Chapter 5 reports the conclusions and few key recommendations, drawn from the study.

CHAPTER 2

2. LITERATURE REVIEW

2.1. General

This chapter summarizes the concepts and methodology in context of the previous research which are useful to proceed this research work.

2.2. Clay particle and clay mineralogy

Soil particles interaction depends on both spacing and orientation between the soil particles, and on the basis of these parameters, clay generally presents two types of structures, i-) flocculated structure (Figure 2.1), and ii-) dispersed structure (Figure 2.2). In dispersed structure, the net particle forces are repulsive and in flocculated structure, the net particle forces are attractive.

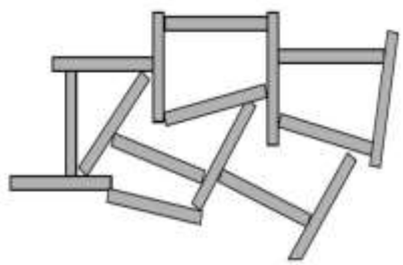

Figure 2.1: Flocculated clay structure

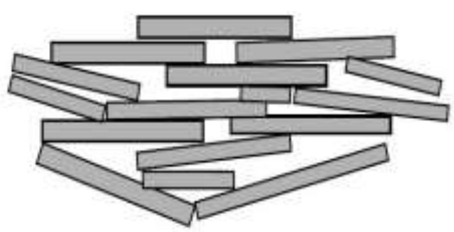

Figure 2.2: Dispersed clay structure

(Scott and Ronald, 1963) stated that soil mineralogy plays a great role on its engineering behavior. The major problem which is greatly associated with the clayey soils is its swelling potential, which is relatively greatly depends upon the soil mineralogy. (Velde, 1992) reported that the clay particles having size less than 2 μm show more tendencies to adsorb water due to net negative charge of colloids. (Soga and Mitchell, 1993) showed that the smaller particle size, negative electrical charge, higher plasticity and highly weathering resistance are the properties which are associated with clayey soils. The study further added that most clayey minerals are in platy shape, tabular shape or/and needle shape. Figure 2.3 showed particle size range criteria of the soil.

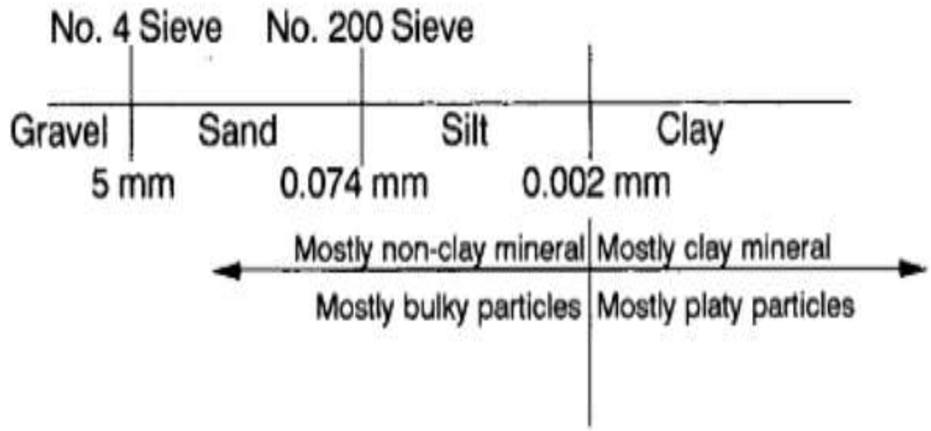

Figure 2.3: Particle size range criteria of the soil

2.3. Composition of clayey soil

Mostly clay minerals composition structural elements are silica tetrahedral and alumina octahedral sheet. These are formed depending on crystalline arrangement and combination of these sheets. In silica tetrahedral composition single silicon atom is surrounded with four oxygen atoms while in alumina octahedral six oxygen or hydroxyl is surrounded with aluminum while in some cases magnesium or iron was also present. (Grim, 1953) classified octahedral into two types depending on the valency of cation such as di-octahedral and tri- octahedral. Figure 2.4 represent silicon tetrahedral and aluminum octahedral form of layers.

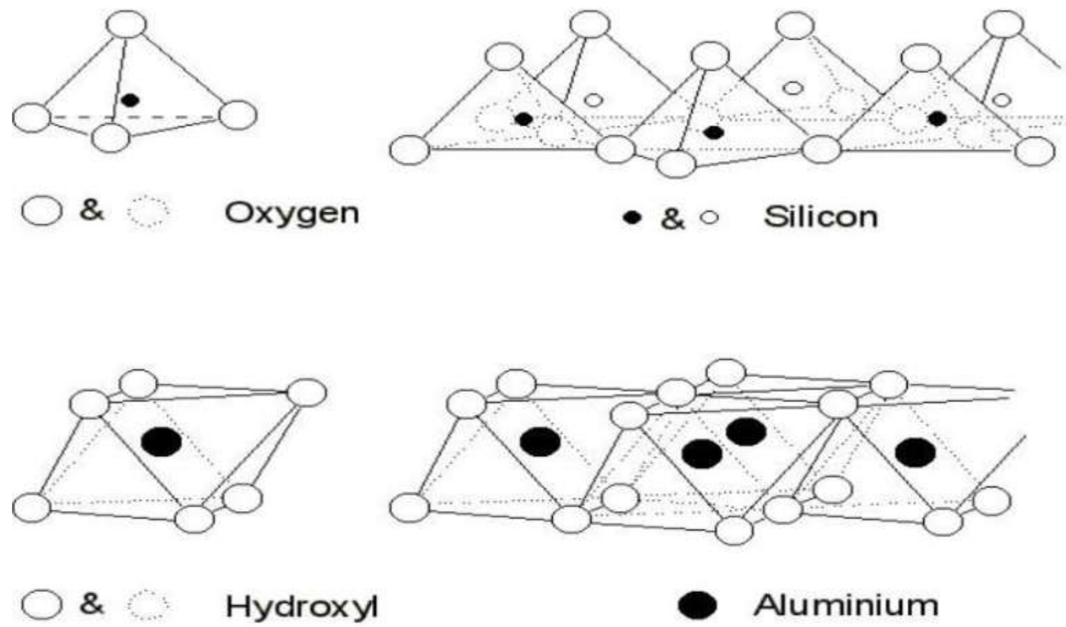

Figure 2.4: Silicon tetrahedral and aluminum octahedral structural arrangements

2.4. Types of clay minerals

Clay minerals are classified into different group on the bases of physical and chemical properties. These properties depend on crystalline arrangement and their combination. Brief introduction of these minerals are discussed below.

2.4.1. Kaolinite

Kaolinite is most abundant mineral that present in nature. Kaolinite is formed due to the chemical weathering of the aluminum silicate minerals like feldspar having chemical composition is $2SiO_2Al_2O_32H_2O$. This type of minerals structure comprising of two sheets one silica tetrahedral and the other alumina octahedral. These sheets are interconnected with strong hydrogen bonding force therefore they have less water absorption capacity therefore these have little tendency to change in volume change can be observed during wetting and drying. Kaolinite has low swelling potential then other minerals due to low specific area. Kaolinite crystals are formed by repeating 0.72 nm thickness of mineral layers. It is not uncommon to have kaolinites crystals 70 to 100 layers thick (Holtz and Kovacs, 1981). The range of cation exchange capacity is 3 to 15meq/100 gm due to little substitution within the minerals sheets. Kaolinite is widely used in pottery, paint filler and paper coating. Figure 2.5 showed kaolinite structure.

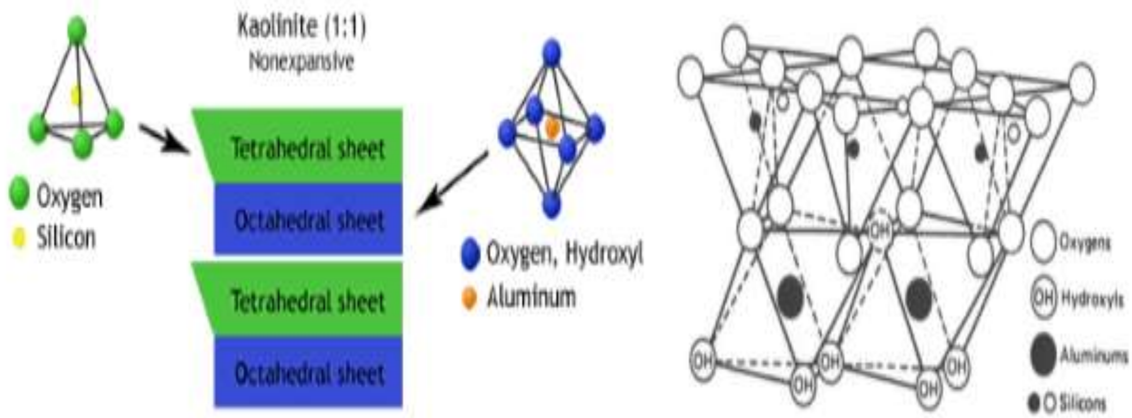

Figure 2.5: Kaolinite structure (Soga and Mitchell, 1993)

2.4.2. Illite

Illite mineral was discovered by Professor Grim from University of Illinois therefore its name came from Illinois. The mineral structure comprises of two silica tetrahedral sheets and one aluminum octahedral sheet. The basic structure of illite mineral resembles to montmorillonite, however the major difference is that the layers are bonded by potassium. The potassium bond is relatively stronger than Van der Waals forces of montmorillonite and relatively weaker than hydrogen bond of kaolinite (Grim, 1953). Illite have less swelling potential then montmorillonite due to non-exchangeable K^+ ions, and its range for cation exchange capacity (CEC) is 10 to 40 meq/100gm. Figure 2.6 shows the structural arrangement of illite mineral

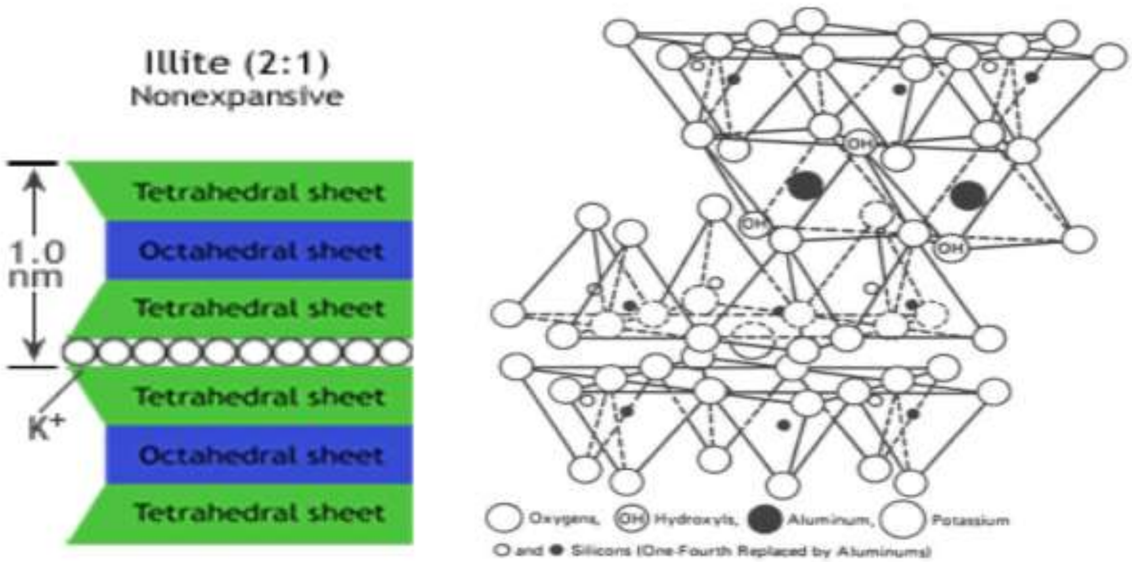

Figure 2.6: Illite structure (Mitchell and Soga, 1993)

2.4.3. Montmorillonite/smectite

Montmorillonite is the most common mineral of smectite group that mostly derived from volcanic ash (Grim, 1953). This type of mineral structural comprising of one central octahedral sheet with two silica tetrahedral sheets. The silica sheets are interconnected by Van der Waals forces and have a deficiency of net negative charge exist on octahedral sheet. Therefore, the exchangeable ions and water can enter between the sheets and break the layer. Water can penetrate into the mineral due to weak Van der Waals forces; water can penetrate into the sheets and causes the layer separation, so montmorillonite swelling behavior can be characterized. Due to large specific area these types of minerals have more water absorption capacity. The range of cation exchange capacity of these mineral varies 80 to 150 meq/100gm.These minerals are widely used oil industries, clay liners, seepage prevention, lubrication, and prevention of groundwater contamination (Thair and Sarapaa , 2008). Figure 2.7 shows the structural arrangement of smectite.

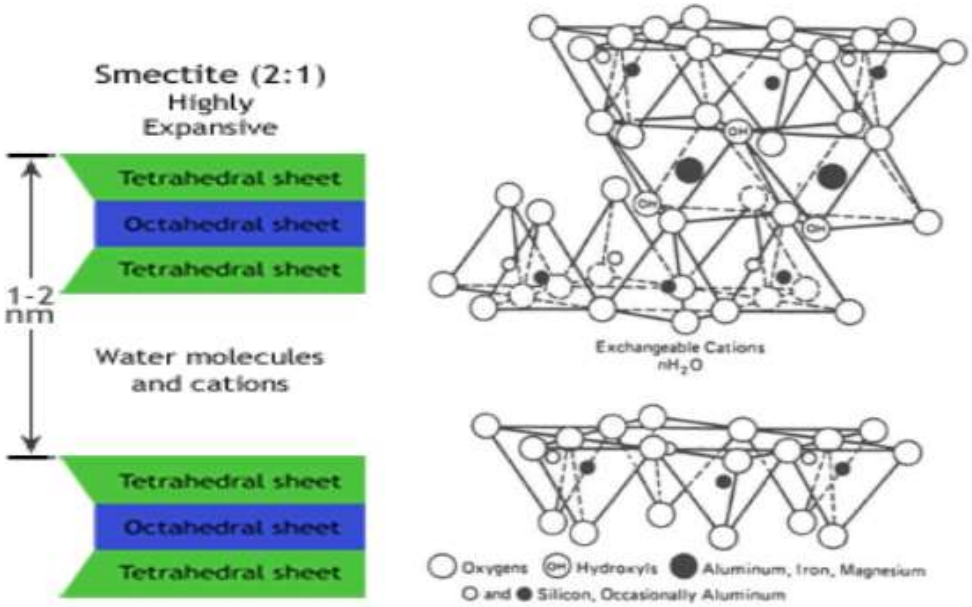

Figure 2.7: Structural arrangement of smectite (Mitchell and Soga, 1993)

2.5. Cation exchange capacity

Cation exchange capacity of clay mineral is the amount of exchangeable cations that can be replaced by the cations of higher replacing power than the absorbed cation. It is useful parameter for the determination of the clay mineral properties of the soil. Higher cation exchange capacity means that the mineral has higher water absorption capacity. CEC is represented in term of milli equivalents per 100 grams of soil, and it depends on the type of clay minerals, crystalline structure, and surface area of the clay minerals.

Table 2.1: Cation exchange capacity of clay mineral (Mitchell and Soga, 1993)

Mineral	**CEC (meq/100g)**
Montmorillonite	8-150
Illite	10-40
Kaolinite	3-15

2.6. Diffuse double layer

Clay has negative charge on its surface therefore it can be balanced by positively charged cation. These cations are spread in two layers called as diffuse double layer. Stern layer is formed due to those ions those have strong attractive force and nearest to the colloids while Gouy layer are formed due to those ions those have weak bonding forces due to more spacing between colloids. The thickness of diffused double layer is measured by the composition and concentration of cation. Concentration of cation is maximum when it is close to the colloids and decreases with increasing distance while concentration of anion is minimum when it is close to the colloid and increases by increasing distance. When concentration of cations and anions become equal it is called boundary of the diffused double layer. Thickness of the diffused double layer plays a major role of swelling and shrinkage behavior of the clayey soil. (Mitchell and Soga, 1993) stated that, thickness of diffused double layer and swelling behavior in clay minerals increases at lower concentration of cation. Thickness of the diffused double layer is also increases by increasing temperature. The swelling in clayey minerals depend upon their chemical structure, which is a function of water content (Carter and Bentley, 1991). Figure 2.8 showed DDL mechanism of the soil.

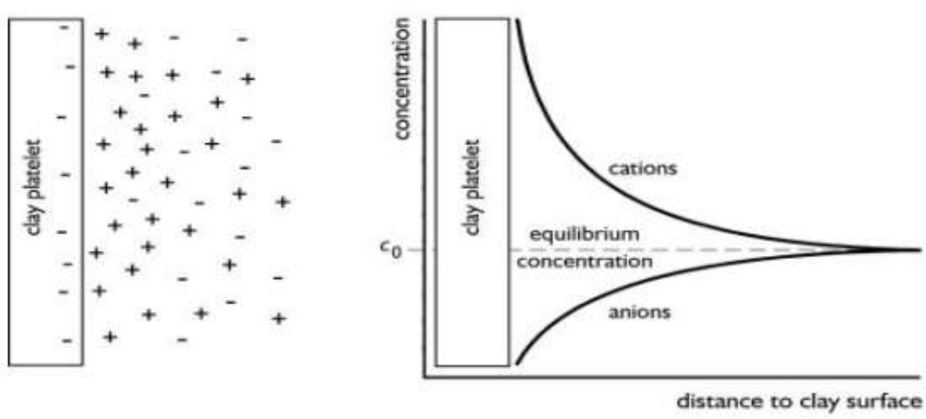

Figure 2.8: Scheme of diffuse double layer theory (Mitchell and Soga, 1993)

2.7. Activity

The changes in soil swelling behavior depend on plasticity of the soil and percentage of clay fraction (Skempton, 1953). Furthermore, according to (Janbu, 1970)activity is also associated with the types of clay minerals. Table 2.2 shows activity of different clay minerals.

Table 2.2: Activity of clay minerals (Janbu, 1970)

Mineral	Activity
Quartz	0
Calcite	0.18
Mica	0.23
Kaolinite	0.33-0.40
Illite	0.9
Montmorillonite, Ca	1.50
Montmorillonite, Na	7.20

(Mitchell and Soga, 1993) showed that there is strong relationships exist between salt concentration and activity. Figure 2.9 showed relationship between soil activity and salt concentration.

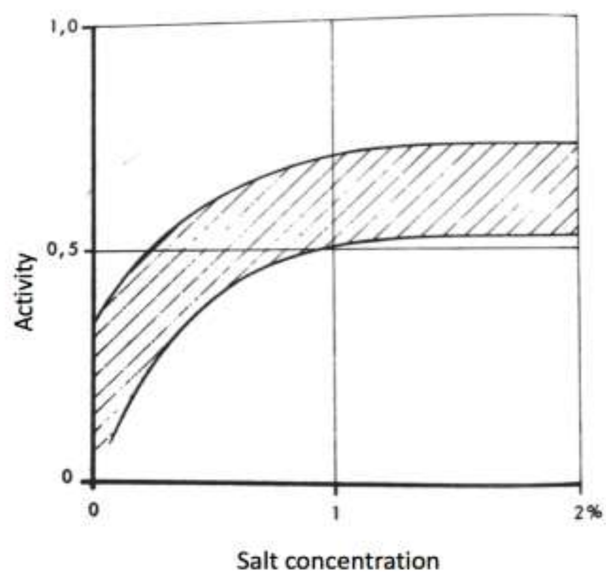

Figure 2.9: Relationship between activity and salt concentration (Janbu, 1970)

2.8. Coefficient of uniformity

Coefficient of uniformity (Cu) is used to determine the particle size range which is the ratio of D_{60} to D_{10} size of particles.

$$C_U = \frac{D_{60}}{D_{10}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eq (2.1)}$$

In Equation 2.1, D_{60}= Particle size corresponding to 60 % finer, D_{10} = Particle size corresponding to 10 % finer. The coefficient of uniformity is different for different particle size such as for; well graded gravel, Cu >4; well graded sand, Cu >6; and for poorly graded sand Cu<6. Soil with Cu =1 provides grains of equal sizes.

2.9. Coefficient of curvature

The coefficient of curvature (Cc) as in Equation 2.2 is used to estimate gradation of particles

$$C_c = \frac{(D_{30})^2}{(D_{60})(D_{10})} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eq (2.2)}$$

In Equation 2.2, D_{60} = particle size corresponding to 60% finer, D_{30} = Particle size corresponding to 30% finer, D_{10} = Particle size corresponding to 10% finer. The coefficient of curvature for well graded soil falls between 1 and 3.

2.10. Soil characterization

2.10.1. Grain size distribution

The grain size distribution played an important role to define the engineering properties of various soils (Tyler and Wheatcraft, 1992). Soils with different grain size distribution can be used for different purposes. Well graded soil performs better as a filler material than uniformly graded soils in controlling the drainage applications. Hydraulic conductivity in certain situations of practical applications can be controlled with certain ranges of the grain size distribution. Sieve analysis and hydrometer tests are performed to estimate the grain size distribution of all soils. Figure 2.10 shows the grain size distribution for various soil types (Atkinson, 1993). Table 2.4 reports the soil nature on the basis of coefficient of uniformity (Janbu, 1970). The coefficient of uniformity is directly estimated with grain size distribution profile. The lower value of coefficient of uniformity means that the soil specimen is loosely packed.

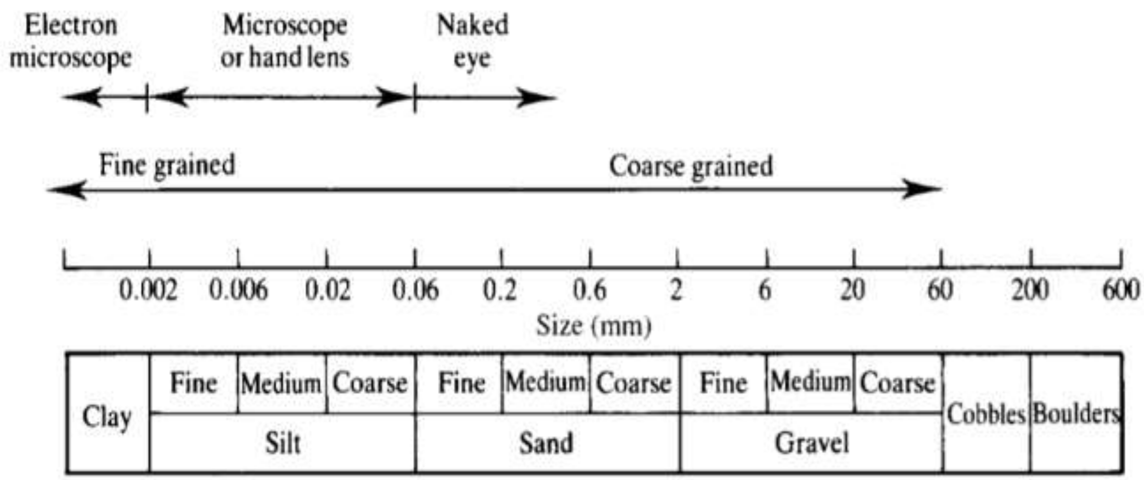

Figure 2.10: Classification of soil particles on the basis of grain sizes (Atkinson, 1993)

Table 2.3: Soil description according to coefficient of uniformity (Janbu, 1970)

C_u	Description
< 5	Uniformly graded
5-15	Medium graded
> 15	Well-graded

2.10.2. Consistency limits

Consistency limits characterize the soil behavior for various ranges of water contents. These limits are useful for identification of mechanical behavior of soil such as shear strength, compressibility and swelling potential of cohesive soil. (Smith et al., 1985) did experimental evaluation of different soil types for Atterberg limits, and concluded that non cohesive soils showed more volume changes at lower plastic limit while cohesive soil showed more volume changes at higher plastic limit.

(Schmitz et al., 2004) derived correlation between clay contents and consistency limits, performing experimental work on soil samples, collected from the area of Belgian basin and concluded that the clay mineralogy enabled to change the geotechnical properties of the soil. (Zolfaghari et al., 2015) studied rich limestone soil collected from different regions of Iran and concluded that the Atterberg limit increased with an increase in the %age of organic matter and cation exchange capacity.

2.10.3. Specific gravity

Specific gravity of soil is an important parameter, used for the soil characterization and to measure the engineering properties of the soil. (Oyediran and Durojaiye, 2011) reported that specific gravity is an important property to guess the swelling potential of a soil, initially. (Oyediran and Durojaiye, 2011) worked on residual clay of Nigeria and concluded that the specific gravity of clay varied from 2.69 to 2.72. Soil having high degree of laterization means higher specific gravity. (Tuncer and Lohnes, 1977) worked on laterite soil collected from the area of Hawaii and concluded that soil strength increased with an increase in the specific gravity, and it was greatly dependent upon the degree of weathering. Table 2.4 showed specific gravity criteria of different soils.

Table 2.4: Typical Values of Specific gravity (Bowles, 2012)

Type of soil	Specific gravity
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.70-2.80
Soil with mica or iron	2.75-3.00
Organic soil	1.00-2.60

2.10.4. Moisture density relationships

Soil compaction is a mechanical process to enhance the soil stability. The mechanical behavior of a soil is greatly associated with the compaction curve, which is unique for both wet side and dry side of optimum for a particular and as well as for different soils. Several studies are available in literature correlating the moisture density relationships with other parameters of the soil.

(Ikeagwuani et al., 2018) performed unconsolidated undrained triaxial tests to correlate the maximum dry densities and optimum water content with cohesion on soil samples, collected from different zones of Nigeria and resulted that highly plastic clayey showed strong correlation between cohesion and dry density, while the low plastic clay was comparatively unable to provide good relationships. (Ali et al., 2019) correlated the index properties with the compaction curve, while working with 27 different soils and reported that a poor relation exists between compaction characteristics and index properties of the soil. However, the plastic limit in most soils was noted close to optimum moisture content. (Smith et al., 1997) studied the influences of compaction on the mechanical behavior of different soils, collected from the forest of South Africa. The %ages of clay and carbon contents in the soil varied from 8 to 66% and 0.26 to 5.77%, respectively. The study concluded that the penetrometer strength increased with an increase in the bulk density, and as well as with decrease in the moisture content. However, the soil strength decreased with an increase in the %ages of clay contents. (Al-Obaidi et al., 2020) correlated relative compaction with bearing capacity and unconfined compressive strength of different soils and reported that as expected, the bearing capacity and cohesion increased with an increase in relative compaction of the soil. (Mitchell and Soga, 1993) showed that the mechanical behavior of soil changed with change in the compaction effort, for both dry of optimum and wet of optimum sides. (Nabil and Mariam, 2014) studied the influences of relative compaction on shear strength of (SW-SC) soil, and stated that an increase in relative compaction relatively increased the angle of internal friction. In addition to this, the stiffness also increased with an increase in the degree of relative compaction. (Reza and Mehrab, 2014) worked on two poorly graded soils, and correlated the effects of normal stress and relative compaction on secant frictional angle of sand. The study concluded that secant friction angle decreased with an increase in normal stress, and furthermore, the secant friction angles for different soils were also increased with an increase in relative compaction from 93% to 100%. (Yusoff et al., 2017) worked on kaolin and laterite soil to correlate the compaction energy with geotechnical properties and concluded that maximum dry density increased with an increase in the compaction energy

2.10.5. Consolidation

Consolidation is a process in which dissipation of pore water in soil occurs under the application of axial effective stress which alternately enhances its density with some particles rearrangement. The plasticity index and percentage of clay contents generally define the consolidation potential of a soil. It is essentially important to define the consolidation potential of a soil to properly utilize the design considerations to keep it within the tolerable limits during the project life. (Robinson and Allam, 1998) correlated consolidation parameters with clay mineralogy while working on different clayey minerals and concluded that for montmorillonite, the coefficient of consolidation decreased and for illite and kaolinite, it increased for an increase in effective stress. (Vikas et al., 2015) correlated compression index with plasticity index of soil samples collected from different zones of River Valley India and concluded that compression index increased for plasticity index between 5 and 30 and for compression index between 0 and 0.35. (Priyadarshini et al., 2015) examined the consolidation behavior of CH, MH, and MH-CH soils and showed that CH showed more compressibility than MH-CH. The test data showed that the coefficient of consolidation provided good relationships with shrinkage index than plasticity index. (Rakesh and Jain, 2016) performed experimental work on different soils collected from different zones of Bhopal city India to correlate plasticity index with compression index and concluded that an increase in plasticity from 18.38 to 98.27resulted in an increase in compression index from 0.17 to 0.893. (Ijimdiya and Igboro, 2012) examined compressibility behavior of oily contaminated reddish-brown laterite CL soil of Shika Nigeria. The study showed that the coefficient of volume compressibility decreased with an increase in the normal stress and with %age of oil as well. The coefficient of consolidation increased with an increase in oil contents up to 6 %, and then decreased beyond this concentration. (Nguyen et al., 2020) examined the coefficient of horizontal consolidation of undisturbed soft soil samples collected from different provinces of Vietnam. A 0.75 m long thin walled piston was used to extract the specimens, and three different types of tests were conducted to examine the coefficient of horizontal consolidation. The study concluded that the ratio of coefficient of horizontal consolidation to vertical consolidation varied from 1.35 to 10.59. The change in the ratio was due to the factors, such as consolidation pressure, organic matter and sediment deposition. (Takaharu and Misao, 1994) examined the effects of sample disturbance on shear strength and consolidation behavior of alluvial marine soft of Kuwana city, Japan. The test data showed that normally consolidated soils showed large settlement for higher degree of sample disturbance. (Vinod and Bindu, 2010) derived an empirical correlation on the basis of consolidation tests for highly plastic clayey samples, collected from Kerala India, and showed that these soils showed compression index higher than from derived empirical correlations.

2.10.6. Shear strength

Shear strength of the soil is an important property that can cause resistance against sliding of the structure due to the internal surfaces of a soil mass. The geotechnical stability of foundation soil mainly depends upon the angle of shearing resistance and cohesion intercept. Direct shear and triaxial methods are generally employed to determine these parameters. Direct shear test is an easy and economically effective method, for the measurement of shear strength parameter but triaxial test is more precise, time consuming and costly method for the measurement of shear strength. Triaxial test is more representative of field conditions than direct shear. Research showed that there are several parameters, which affect the shear strength behavior of a soil. (Das, 1983) stated that shear strength parameters (cohesion and angle of internal friction) are highly important during design consideration against the bearing capacity and settlement of foundation soils. (Nagendra et al., 2013) reported that structural failure occurred due to the improper selection of strength parameters irrespective of the soil conditions, i.e., soft, stiff or firm. According to (Saleh, 2020), the plasticity and grain size distribution of soil were directly proportional to cohesion and inversely proportional to angle of shearing resistance. (Mousavi et al., 2011) stated that shear strength of soil varied with changes in soil type, plasticity index and density of the soil. (Ain et al., 2010) stated that cohesion was dependent upon clay content and type of clay minerals, and shape, texture, grain size distribution, water content and dry density influenced the angle of internal friction. (Castellanos and Brandon, 2013) did a comparative evaluation of triaxial and direct shear tests, while working on different soils, and showed that the direct shear tests provided angle of internal friction 2 to 5 degree less than triaxial tests. (Majid and Azam, 2013) conducted unconsolidated undrained triaxial tests to examine the shear strength parameters of the undisturbed soil samples, collected from University of Technology Malaysia. The samples were extracted from the ground using standard metal tube. The test data showed that a nonlinear relationship existed between apparent shear strength and suction. (Mun et al., 2016) studied effects of strain rate, hydraulic condition and compaction on undrained shear strength of low plastic compacted clay, collected from University of the Colorado, USA. The unconsolidated undrained triaxial tests were performed in this study, which showed that a direct relation exists between undrained shear strength and stain rate at lower excess pore water pressure, however, at higher excess pore pressure, the relationship was nonlinear. Furthermore, the test data showed that the soils showed lower undrained shear strength for wet of optimum side than dry of optimum side at a particular strain.

2.10.7. California bearing ratio (CBR)

California bearing ratio (CBR) is an important parameter, used to examine the thickness of pavement structure, stiffness modulus and subgrade shear strength of soil. CBR value is generally an indicator of subgrade strength. The higher subgrade CBR means that it has more strength, and therefore less thickness of the pavement is needed. The value of CBR is further used to measure resilient modulus that is directly used in designing of flexible pavement. Equation 2.3 is generally used to determine the resilient modulus.

$$\mathrm{M_R} = 1500\ \mathrm{CBR}\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{Eq}(2.3)$$

One-point CBR is generally used for heavy traffic for the economical design consideration of the project. Research shows that different factors such as index properties, soil texture, soil types, dry density, and water content, etc. affect the CBR value. (Alayaki and Bajomo, 2011) examined the effects of moisture content on CBR of soil and concluded that CBR reduced with an increase in soaking period, which was due to the breakage of bonds between water molecules and soil particles. The study also reported that CBR is inversely proportional to the soaking period of soil. (Vishal and Yadav, 2016) developed correlation between CBR values with index properties of the soil, and concluded that CBR value increased for an increase in maximum dry density, grain sizes, and decreased with an increase in moisture content. (Jaleel, 2011) studied the soaked CBR tests at 95% relative modified compaction of fourteen different soils, and as expected the bearing capacity decreased with an increase in the soaking period. (Talukdar, 2014) correlated soaked CBR tests with other properties developing empirical relationships of 16 different soils collected from Assam, India and concluded that CBR increased with a decrease in plasticity index, water content, and increased with an increase in maximum dry density. (Roksana et al., 2018) worked on unsoaked CBR in relation to compaction energy for five different soils in Bangladesh, and concluded that the unsoaked CBR changed with changing compaction effort. (Yashas et al., 2016) developed empirical relationship for CBR in relation to various engineering properties such as specific gravity, field density, dry density, cohesion. properties of soil in India, and concluded that soaked CBR changed with changes in index properties, and CBR showed inverse relationship with angle of internal friction, liquid limit and optimum moisture content. (Khan et al., 2016) carried out soaked CBR of fiber reinforced soils, selecting from various zones of India and reported that CBR increased with an increase in the concentration of fiber contents from 0.1 to 0.3 %, but after this treatment, the CBR showed a decreasing trend. The study also concluded that CBR was noted maximum for fiber length up to 18 mm. (Abdullah et al., 2018)correlated compaction energy with dry density and CBR value of silty clay samples, collected from various zones district Thiruporur, India. The study reported that the maximum dry density increased and moisture content decreased for an increase in compaction energy, which relatively increased both soaked and unsoaked CBR. The study also suggested that the 97% relative provided the most economical thickness of pavement.

2.10.8. Electrical conductivity of soil

Electrical conductivity is an important parameter to define the soil salinity of the soil, which is greatly associated with mineralogy, soil structure, temperature and water content. (Wei Bai et al., 2013) examined the EC of five different laterite soils in China at different densities of compaction curve, and concluded that the relationship between temperature and electrical conductivity is nonlinear. The EC increased with an increase in water content, however showed less changes close to the optimum water content. (Hemu et al., 2018) correlated the index properties with chemicals properties while working on twenty soil samples, collected from different zones of Hyderabad, India and concluded that the EC of soil specimens varied from 1.01 to 3.02 ms/cm with changes in index properties.

2.10.9. pH

pH plays an important role in defining the mechanical behavior of a soil. (Najme et al., 2019) reported that an increase in %age of sodium hydroxide in clay minerals increased the compression index, and addition of acidic solution in clay minerals resulted in reduction of the compression index. Furthermore, the consistency limits increased and decreased with alkaline and acidic natures of the solution. It was noted that pH solution in the presence of iron oxide influenced the strength too. (Momeni et al., 2020) studied geotechnical behavior of different soils changing the pH value. In this study, a certain amount of sulphuricacid and nitric acid was used in water to change the PH of water before mixing with the soil. The study showed that the liquid limit, plasticity index and coefficient of permeability increased with an increase in pH value. The undrained shear strength also significantly changed with changes in pH value. (Sunil et al., 2006) studied chemical and geotechnical properties of the laterite soil for different pH solutions. The test data showed that the maximum dry density and specific gravity of soil decreased with an increase in the soaking period with same pH value. The alkalinity, hardness and sulphate contents of the soil also decreased with a gradual increase in the soaking period.

CHAPTER 3

3. MATERIALS AND METHOD OF THE TESTING

3.1. Introduction

The purpose of this chapter is the detailed procedure of the testing to achieve the objective of the whole research work and also gives an overall idea about the completion of the whole research study starting from sample collection to laboratory testing. These tests can be performed in the laboratory by following proper standards procedures.

3.2. Materials

3.2.1. Soil sampling

Figure 3.1: Map of soils samples collected area

For the research work, 13 different sites were selected from different zones of Pakistan to examine their engineering behavior, which are necessary to consider in geotechnical design. All disturbed samples were collected at least 1 m depth, which was due to keep them free from organic matter and other impurities. Figure 3.1 shows the location map of different zones, selected for the study. The soils were classified for various engineering properties including basic soil information, grain size distribution, soil plasticity, moisture density relationships, compressibility, shear strength parameters and resilience modulus used for infrastructure project designing.

3.3. Methods used

3.3.1. Moisture content determination

This test was performed in the laboratory by using (ASTM D 2216-05) procedure for finding water content in the soil. Natural moisture content of soil tells us about the state of soil also it is used for the calculation of bearing capacity and settlement. It was observed that moisture content in clayey soil was larger than sandy soil. It value depend on type of clay minerals and quantity of clay. Figure 3.2 show soil sample for moisture content determination in laboratory.

Figure 3.2: Soil sample for natural moisture content determination

3.3.2. Grain size distribution

Grain size distribution is useful for the classification of relative proportioning of different sizes in the soils. Sieve analysis and hydrometer tests are generally employed for the mechanical analysis of soil. Sieve analysis test was performed in the laboratory by following the procedure as discussed in (ASTM D 422-63) standard, and the test was performed on soil particles with size greater than 75 µm. These tests were conducted thrice to ensure the repeatability of the test data. Hydrometer test was performed according to specifications as in (ASTM D 422-63) standard, and this test was generally performed on soil particles with sizes smaller than 75 µm. This test is generally not suitable for the soils, if the %age of passing through sieve # 200 is less than 10%, as discussed in the standard. Sodium metaphosphate was used as a dispersing agent using concentration of

40g/litre, and the solution was placed about 16 hours for soaking, before starting the test. Hydrometer readings were noted at a time interval of 1, 2, 4, 8, 15, 30, 60, 240, 480, and 1440 min as mentioned in the standard. The temperature was noted in parallel to the hydrometer reading in order to calibrate the test data. It was observed that hydrometer dropped quickly in soils with higher %age of silt as compared to soils with higher clay contents. It was due to the fact the silt settled fast than the clayey soil due to larger particle sizes. Furthermore, the turbidity was quite visible in clayey soils as compared to silty soils. For Rajanpur soil, the %age passing from sieve # 200 was 3.23%, less than 10%, so hydrometer test was not performed. Figure 3.3 show pictorial view for sieve analysis and hydrometer analysis performed in laboratory.

Figure 3.3: Scheme of sieve analysis and hydrometer in the laboratory

3.3.3. Atterberg limit test

The liquid limit and plastic limit were determined following the procedures as discussed in (ASTM D 4318-05) standard. Both fall cone and Atterberg limits methods were employed to determine the liquid limit of soils. Fall cone method was followed to determine the liquid limit of sensitive soils. In this study, the fall cone method was used to determine the liquid limit of 03 samples as it was unable to determine their liquid limit with Casagrande devices as these were sensitive soils.

The plasticity index was then determined from these limits to classify the soil. Figure 3.4 shows liquid limit and plastic limit setup in laboratory.These value are used in the calculation of soil activity, and consolidation. Triplicate samples were prepared for each moisture content to ensure the repeatability of the test results. Soil having high plasticity index indicates more clay content that showed high swelling properties. Low plasticity index values indicate low clay contents or soil having more silt content while plasticity index zero indicates little or no quantity of silt and clay were present.

$$\mathrm{PI} = \mathrm{LL} - \mathrm{PL} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. \mathrm{Eq}(3.1)$$

Figure 3.4: Liquid limit and Plastic limit test arrangement in laboratory

Shrinkage limit test was carried out on cohesive soil by following the procedure as discussed in ASTM D 427-04 Standard. Soil having more swelling potential required more water to form homogenous mixture to remove air voids during tapping action. Figure 3.5 shows a scheme for shrinkage limit tests in laboratory.

Figure 3.5: Shrinkage limit was in progress

3.3.4. Specific gravity test

Specific gravity tests were performed, following the guidelines given as discussed in (ASTM D 854-06)standard. It was observed during experimentation that the coarse particles took less time for the removal of air voids than clayey soils during heating on hot plate. Figure 3.6 shows the specific gravity tests in progress.

Figure 3.6: Specific gravity test was in progress

3.3.5 Moisture density relationship

The standard compaction test was carried out following the guidelines as set out in the (ASTM D 698-07) standard to develop compaction curves for various soils. In these tests, the water was increased with an increment of 2% for sandy soil and 3% for clayey soils, and it was done to develop the compaction curve in proper shape. The clayey soil needs more water due to the larger surface area for its soil water homogeneity. At higher water contents, the clayey soils were adhered with the mold and hammer, which made the compaction of these specimens difficult. Therefore, a knife was used to remove the adhered materials from hammer and mold. Each test was performed two times to ensure the repeatability of the test data. Figure 3.7 shows the compaction test in progress.

Figure 3.7: Compaction test pictorial views

3.3.6. Consolidation test

Consolidation tests were performed in the laboratory according to the guidelines as discussed in (ASTM D 2435-03)standard. This test is important as it defines the settlement behavior of clayey soils along with other parameters such as, compression index, coefficient of consolidation, swelling index and hydraulic conductivity. All samples were prepared at 95% of the maximum dry density of the compaction curve. After preparation, the samples were transferred to a consolidometer ring with size 6 cm dia. and 2 cm height. The load was applied in a load incremental ratio (LIR) of 12.5, 25, 50, 100, 200 kPa, and the consolidation for respective loading was recorded at a time interval of 1, 2, 4, 8, 15, 30, 60, 120, 1440 min. However, it was assured that the samples were reached their maximum consolidation capacity. The samples were unloaded in the same sequential order as in the loading stage. The reloading was again applied in a sequential order of 100, 200, 400, 800, 1600 kPa. After this, the sample was taken out of the consolidometer ring and the weight of the specimen was measured along with the change in volume. The specimen was placed in the oven for a period of 16 hours and the moisture content was determined. Finally, the load vs deformation plots were plotted to determine the coefficient of consolidation, compression index and swelling index for all soils. It was noted that the soil having more clay contents showed more swelling behavior due to higher water absorption than the soils with less clay contents. Figure 3.8 shows the consolidation test in progress in the laboratory.

Figure 3.8: Consolidation tests were in progress

3.3.7. Triaxial test

The triaxial compression tests were conducted following the guidelines as set out in ASTM standards. Regarding this, consolidated undrained tests were performed in the laboratory to examine the shear strength parameters of different soils.

3.3.7.1. Consolidated undrained triaxial test

Consolidated undrained tests were performed, following the guidelines as discussed in (ASTM D 4767-04)standard. A split mold with 70 cm dia., and 140 cm height showing height to diameter ratio of 2 as per specification in the standard was used to do these tests. Similar to that of consolidation tests, the specimens were prepared at 95% of the maximum dry density of the compaction curve in the split mold, and then the sample was extracted from split mold with extreme care. After then, the sample was wrapped with a rubber membrane, before placing in the triaxial chamber, and then transferred to the triaxial chamber with extreme care to eliminate any chances of sample disturbances. The sample was sandwiched in the chamber between two porous stones at the top and the bottom. The samples were saturated less than 90% so the dry mounting method was used to complete the full saturation as suggested by standard. The triaxial chamber was filled with water. The entrapped air within the specimen was minimized, releasing the cell air pressure of the chamber, initially, and the specimen was left in the cell in this condition for 24 hours. After then, a 20 kPa cell pressure was applied to initiate the saturation, which alternately changed the pore water pressure in the specimen, and as the pore pressure reached 15 ~ 20 kPa, then the back pressure was applied, almost equal to that of cell pressure, until the pore pressure reached 20 kPa. Similarly, the cell pressure was applied with an increment of 20 kPa such as 40, 60, 80, 100 kPa, and correspondingly, the back pressure was also changed in the same increment. During saturation, it was assured that the difference between chamber pressure and backpressure should not exceed 35 kPa to avoid the swelling / collapse of the specimens as per specification in standard. Now, the Skempton's pore pressure parameter B is the differences between pore pressure and chamber pressure. The pore pressure parameter B equal to or greater than 0.95 means that the sample is fully saturated. As the saturation was completed then the sample was consolidated for the cell pressure opening the drainage valves of chamber, until the pore water pressure was entirely squeezed out from the specimen. After consolidating the sample, the deviator stress was increased gradually to fail the specimen, and during this stage, the drainage valves were remained closed to shear the sample under undrained condition. A shearing rate of 0.5 mm/min was used in these tests. The samples were sheared until the specimens either failed or attained the 20% axial stain under the deviator stress. After the specimen failure, both chamber and backpressure were released to zero, after relieving the deviator stress.

Figure 3.9: Triaxial compression test is in progress in the laboratory -Sample after sharing

The specimen was removed from the chamber quickly to minimize its chances of water absorption from the porous disk. The rubber membrane was removed from the sample and it was placed in an oven to determine the water content. As general, it was noted that the specimens with more clay content needed more time for full saturation than specimens with less clay contents. The load, deformation, and pore pressure parameters were recorded continually during the continuation of the tests. As expected, the specimens with higher %ages of clay size particles showed more consolidation than with lower %age of clay size particles. Figure 3.9 shows the progression of consolidated undrained triaxial test in the laboratory.

3.3.8. California bearing ratio (CBR)

The CBR tests were conducted following the guidelines as discussed in (AASHTO T 193-99) to examine the potential strength of different soils. Similar to that of triaxial and consolidation tests, CBR tests were carried out at 95 % maximum dry density of the compaction curve. The mold with 6×7 in was used in these studies. The standard suggests filling the mold in 5 equal layers with 65 blows for each layer. A straight edge remover was used to trim and level the excessive soil from the top of the mold. The mold was inverted to add the filter paper and also to replace the solid base plate with perforated one on the bottom. The bulk density of each specimen was calculated, measuring its weight and volume, and the bulk density was transformed to dry density, determining the moisture content. For moisture content, the specimens were placed in an oven for 16 hours. A 5 lbs surcharge weight was placed on the top of the mold, which was then attached with a swelling plate recorder to record the swelling potential of the specimens. The mold was then immersed in water tank, and after 96 hours, the swell plate recorder coupled with tripod was used to record the %age swell. After then the mold was taken out from the water, and placed horizontally to allow the drainage for 15 minutes. The mold was then positioned back and then again placed the surcharge weight on the mold and placed the penetration piston of CBR machine on it, touching the top layer of specimen. During penetration, a uniform load of 0.05 in/min was applied on the penetration piston. The load vs penetrations were recorded at an interval of 0.025 to estimate the CBR at 0.1 in and 0.2 in. Figure 3.10 shows the progression of California Bearing Ratio (CBR) test in the laboratory.

Figure 3.10: Pictorial views of the California Bearing Ratio test

3.3.9. Electrical conductivity and pH tests

pH tests were performed in the laboratory, following the guideline as discussed in (ASTM D 4972-01) standard to assess the acidity and alkalinity of the soil specimens. Generally, the pH of the soil varies from 0-14 (S.P. L Sorenson). pH meter is used to determine the pH of a soil. For both pH and EC, 10gms of oven dry soil mass passing sieve # 4 were mixed with 10 ml of distilled water to prepare soil-water slurry. The pH meter directly measures the pH value of the suspension, and EC meter was used to determine the EC of these specimens.

Figure 3.11: Pictorial view of EC and pH tests in laboratory

CHAPTER4

4. RESULT AND DISCUSSION

4.1. General

The results of all the soil tests performed during this research are discussed and shown in this chapter.

4.2. Grain size distribution

Figure 4.1 shows the grain size distribution profiles for different soils, and the sand, silt and clay %ages are presented in Table 4.1.

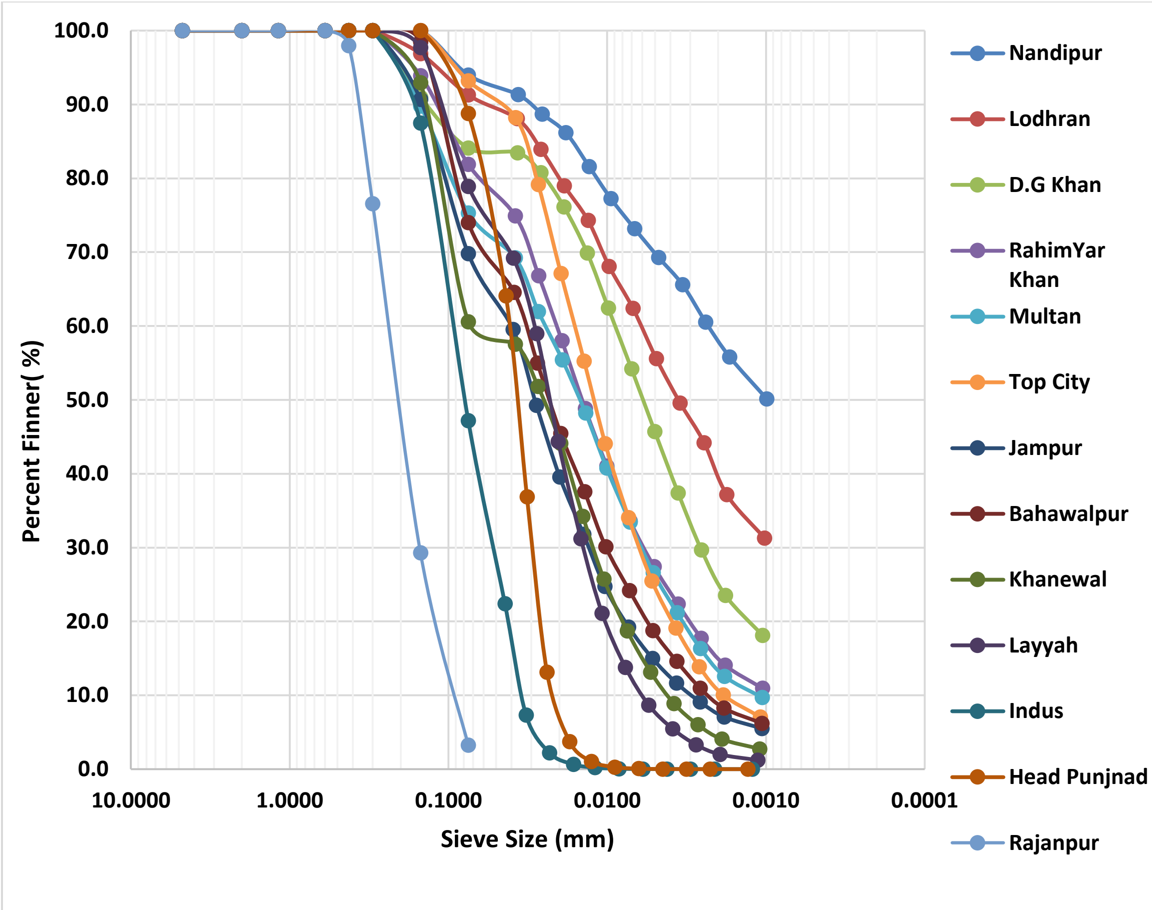

Figure 4.1: Grain size distribution of soil

Table 4.1: Ranges of various fractions present in test samples

Soil Name	Soil Type		Silt	Clay
	Medium Sand	**Fine Sand**		
	0.42-2.0 (mm)	**0.075-0.42 (mm)**	**0.002-0.075 (mm)**	**<0.002 (mm)**
Nandipur	0	6	36.33	57.67
Lodhran	0	8.71	52.11	39.18
D.G Khan	0	15.9	58.82	25.28
Rahimyar Khan	0	18.1	66.93	14.97
Multan	0	24.7	61.79	13.51
Top City	0	6.8	83.14	10.05
Jampur	0	30.02	62.23	7.57
Bahawalpur	0	26	65.07	8.93
Khanewal	0	39.42	56.27	4.3
Layyah	0	21.08	76.91	2
Indus	0	52.8	47.56	0
Head Punjnad	0	11.2	88.79	0

Rajanpur	2	94.77	3.23	0

It can be seen from Table 4.1 that Nandipur soil provides 57.67 % clay contents, comparatively higher than all other soils. Layyah, Indus, Head Punjnad and Rajanpur soils are classified as sandy soils as they show almost zero clay contents, and the grain sizes distribution is also not too diversified.

4.3. Atterberg limits

The Atterberg limit test results are presented in Table 4.2 which shows that Nandipur and Layyah soils show maximum and minimum liquid limits of 60 and 19.5 respectively. The shrinkage limit of Nandipur soil is also higher than other soils. The higher liquid and shrinkage limits of Nandipur soil are due to the higher %age of clay contents. The liquid limit is greatly associated with the %age of clay contents. It can be seen from Figure 4.2 that a linear relationship exists between clay contents and liquid limit. The coarse particles need less water content to reach at liquid state due to less specific area than the fine particles. The test data shows that the Nandipursoil appears to be more problematic than others due to its higher PI and shrinkage limit. As in Table 4.2, the degree of selected soils varies from highly plastic to non-plastic soils.

Table 4.2: Summary of Atterberg limit values

Soil Sample	**Liquid limit**	**Plastic Limit**	**Plasticity Index**	**Shrinkage Limit**	**Degree of Plasticity**	**Shrinkage index**
Nandipur	60	21.95	38.05	20.24	Highly Plastic	39.76
Lodhran	42.5	19.65	22.85	18.13	Highly Plastic	24.37
D.G Khan	37	17.98	19.02	16.37	Highly Plastic	20.63
Rahim Yar Khan	32.5	17.14	15.36	15.52	Highly Plastic	16.98
Multan	30	16.84	13.16	14.78	Medium Plastic	15.22

Top City	28	16.64	11.36	13.42	Medium Plastic	14.58
Jampur	25	15.12	9.88	12.65	Medium Plastic	12.35
Bahawalpur	24	14.19	9.81	12.18	Medium Plastic	11.82
Khanewal	23	13.76	9.24	11.56	Medium Plastic	11.44
Layyah	19.5	11.2	8.3	10.39	Low Plastic	9.11
Indus River	16	N/A	N/A	N/A	Non Plastic	N/A
Headpunjnad	10.3	N/A	N/A	N/A	Non Plastic	N/A
Rajanpur	5.54	N/A	N/A	N/A	Non Plastic	N/A

4.3.1. Relationship between Atterberg limit and other properties of soil

Figure 4.2 shows that the liquid limit increases with an increase in the percentage of clay contents and provides a linear relationship, i.e., y = 0.6701x+19.857 with coefficient of determination R^2=0.9754. This is similar to the findings of (B Widjaja and K Kurniawan, 2020), in which a strong relationship was reported between %age clay content and liquid limits.

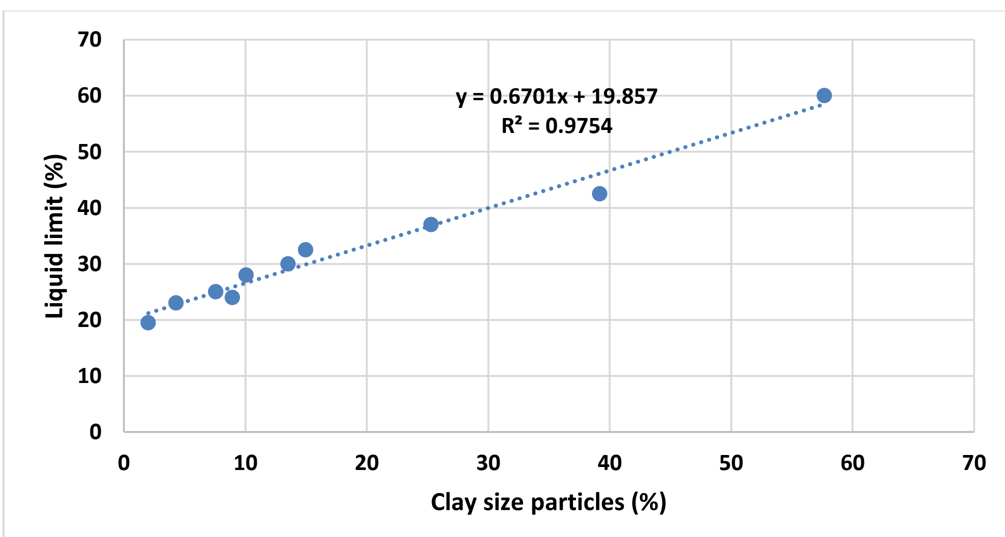

Figure 4.2: Relationship between liquid limit and %age clay size of particles

Figure 4.3 shows that plastic limit increases with an increase in %age of clay size particles, similar to that of liquid limit following a strong linear relationship, i.e., y = 0.1595x+13.522 with coefficient of determination R^2= 0.837, similar to that of (B Widjaja and K Kurniawan, 2020).

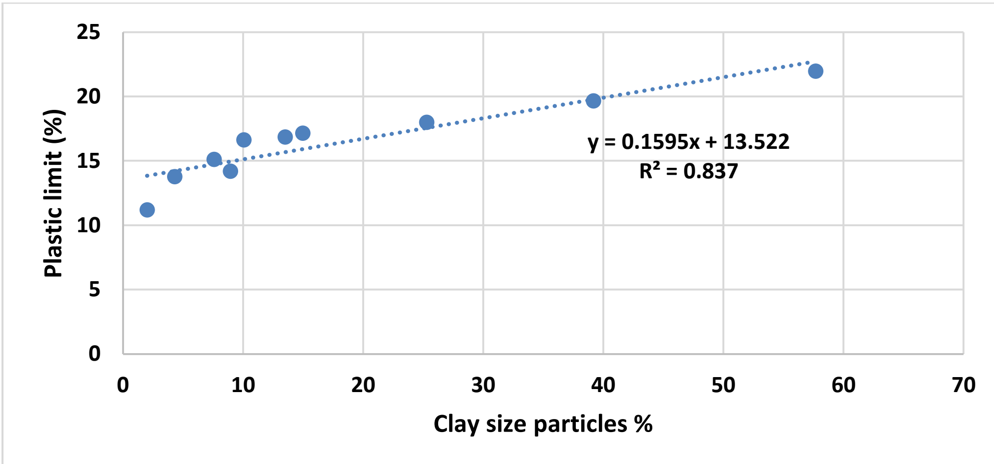

Figure 4.3: Relationship between plastic limit and %age of clay size particles

Figure 4.4 shows that the plasticity index increases with an increase in %age of clay size particles following an excellent linear relationship, i.e., y = 0.5105x+6.3369 with coefficient of determination R^2 = 0.9708, similar to (Akayuli et al., 2013) in which a strong relationship reported between %age clay contents and plasticity index.

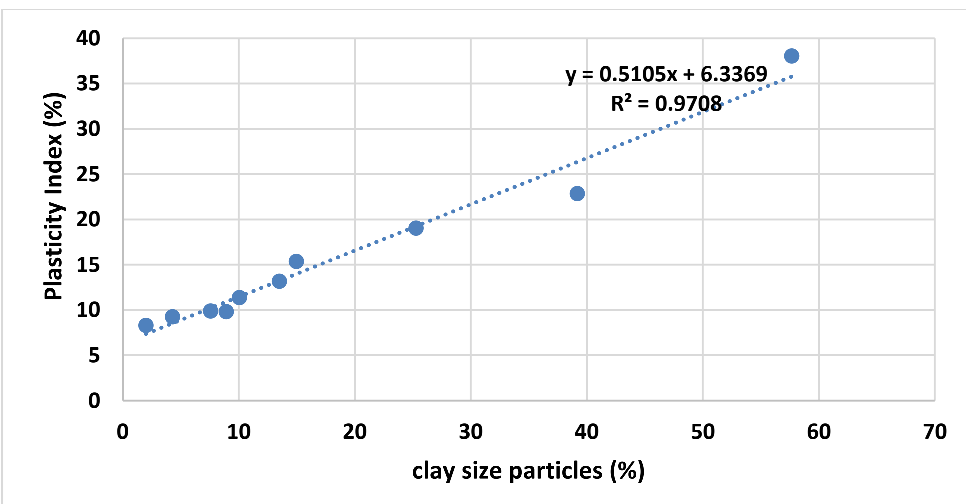

Figure 4.4: Relationship between plasticity index and %age of clay size particles

Similarly, as in Figure 4.5, plasticity index and liquid limit also shows a strong relationship similar to the findings of (Sen and Pal, 2014).

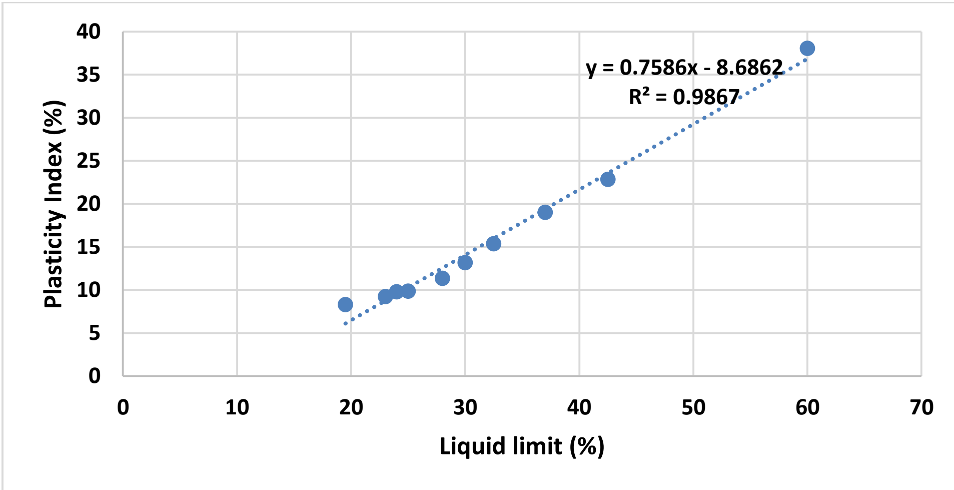

Figure 4.5: Relationship between plasticity index and liquid limit

Figure 4.6 shows that the plasticity index decreases with an increase in %age of sand particles as expected, following a relationship, i.e., $y = 63.289x^{-0.537}$ with coefficient of determination R^2=0.5227. It means that there is a fair relationship between these parameters, which is similar to that of (Islam et al., 2015).

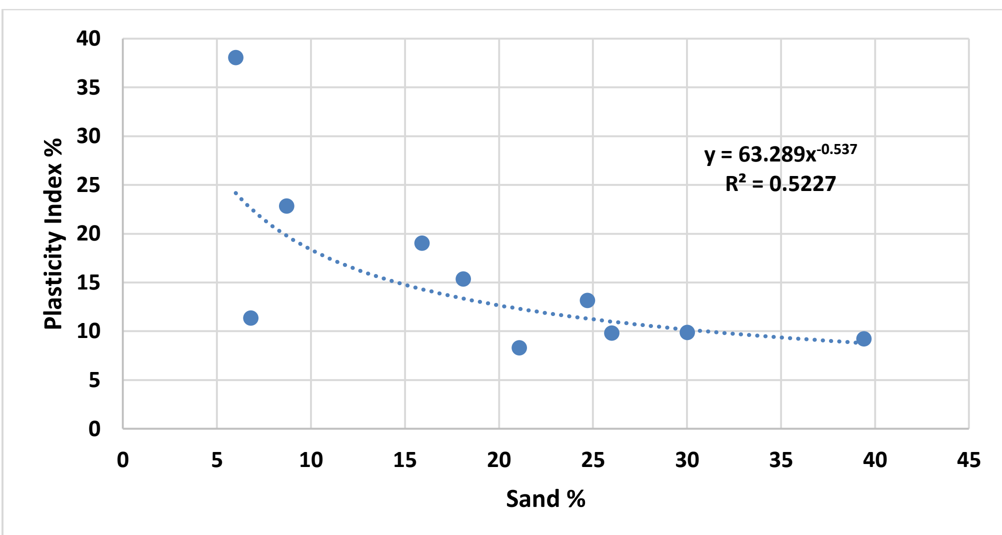

Figure 4.6: Relationship between plasticity index and %age sand

4.4. Soil classifications:

Figure 4.7 shows USCS classification A-line chart for different soils and Table 4.3 shows the soil classifications according to AASHTO and USCS systems. It is clear from the test data that the specimens of most regions fall in CL class, except Nandipur soil which is in CH class as per USCS system. However, according to AASHTO soil classification system, the soils relatively show diversified groups ranging from A-3 to A-7-6.

Indus soil (A-4) falls in SC, Head Punjnad soil in ML (A-4) and Rajanpur soil (A-3) in SP. AASHTO standard states that the soils with %age passing less than 35 % fall in A-1, A-2, and A-3 groups, and are classified as granular materials, and if %age passing is greater than 35% then falls in A-4, A- 5, A-6 and A-7 groups with silty and clayey in nature. According to USCS, a soil is classified as coarse grained if percentage of passing # 200 is less than 50%, otherwise it is a fine-grained soil.

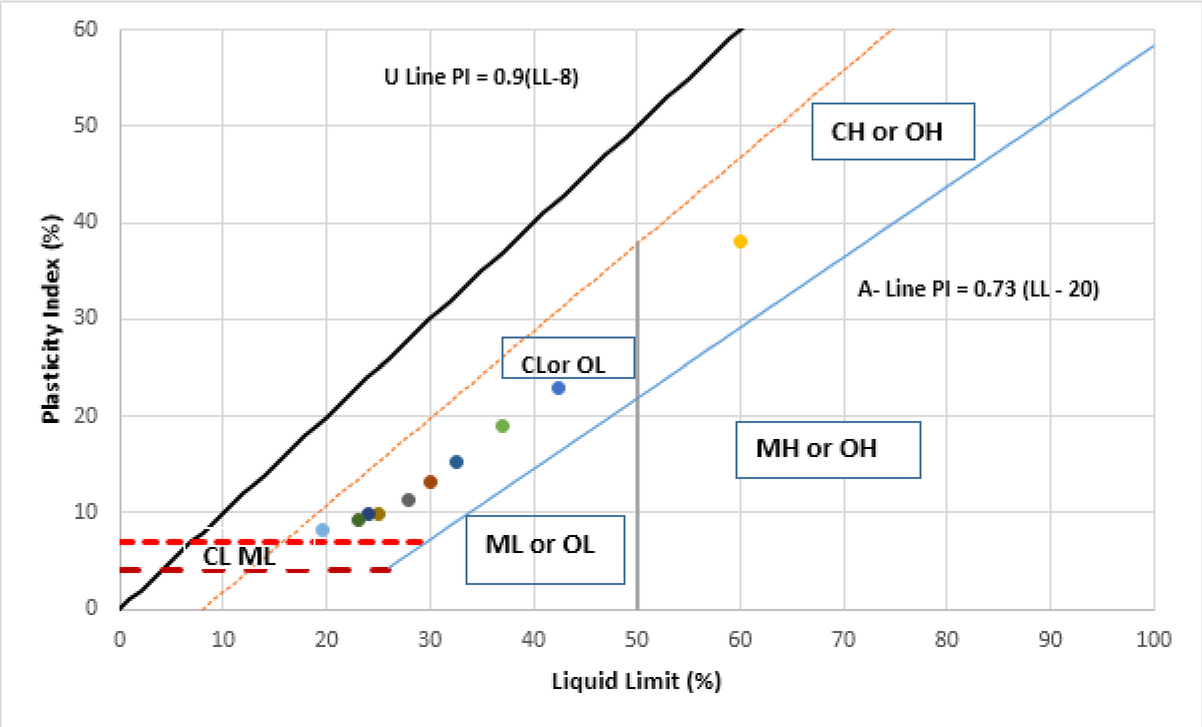

Figure 4.7: USCS classification - A line Chart

Table 4.3: Soil classification summary by using AASHTO and USCS

Soil Sample	**AASHTO**	**USCS**	**Soil Sample**	**AASHTO**	**USCS**
Nandipur	A-7-6	CH	Bahawalpur	A-4	CL
Lodhran	A-7-6	CL	Khanewal	A-4	CL
D.G Khan	A-6	CL	Layyah	A-4	CL
Rahim Yar Khan	A-6	CL	Indus River	A-4	SC
Multan	A-6	CL	Headpunjnad	A-4	ML
Top City	A-6	CL	Rajanpur	A-3	SP
Jampur	A-4	CL			

4.5. Specific gravity

Table 4.4 shows specific gravity of different soils used in the study. It is clear that the specific gravity varies from 2.65 to 2.76 for different soils. The clayey soil shows higher specific gravity than sandy soils, similar to that of Bowles, 2012, in which the inorganic clay (2.70 – 2.80) showed higher specific gravity than sand and silty sand (2.65 – 2.70). Furthermore, CH soil shows higher specific gravity than CL soils (Table 4.4)

Table 4.4: Specific gravity of different soils

Soil Sample	**Specific Gravity**	**Soil Sample**	**Specific Gravity**
Nandipur	2.76	Bahawalpur	2.71
Lodhran	2.74	Khanewal	2.7
D.G Khan	2.73	Layyah	2.7
Rahimyar Khan	2.72	Indus	2.69
Multan	2.71	Head Punjnad	2.67
Top City	2.7	Rajanpur	2.65
Jampur soil	2.7		

4.6. Moisture-density relationship

Figure 4.8 shows moisture density relationships for different soils. It is clear that highly plastic soil (Nandipur soil) provides maximum dry densities at higher water content as compared to other soils, and furthermore, these soils relatively show lower density. As expected the sandy soils, such as ML, SC and SP require less water to reach at maximum dry densities. It means that grain size distribution plays an important role defining the moisture density relationships, and as in Figure 4.8 the maximum dry density and optimum water contents are the function of clay contents. It is also clear from the grain size distribution profile (Figure 4.1) that the compaction curve shifts towards the higher moisture content and lower density as the grain size curve shifts towards the smaller grain sizes.

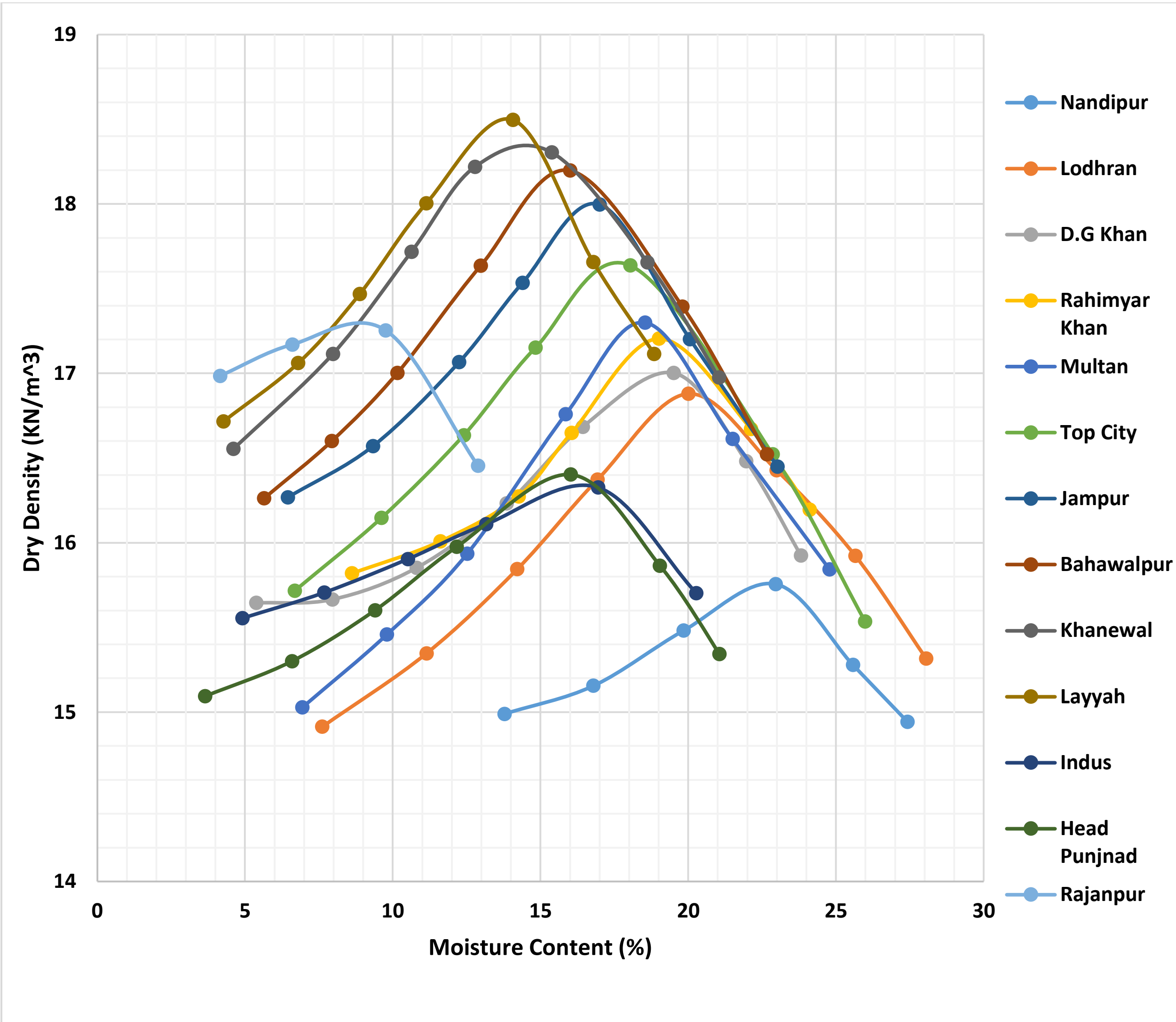

Figure 4.8: Moisture density relationships for various soils

4.6.1. Moisture density relationships with other properties

Figure 4.9 shows that MDD decreases with an increase in the %age of clay size particles following a relationship, i.e., $y = 0.0004x^2 - 0.0698x + 18.485$ with $R^2 = 0.9017$. It can clearly be seen that when the clay contents are less, the rate of increase in MDD is more than higher %ages of clay contents similar to the findings of (Ali et al., 2019).

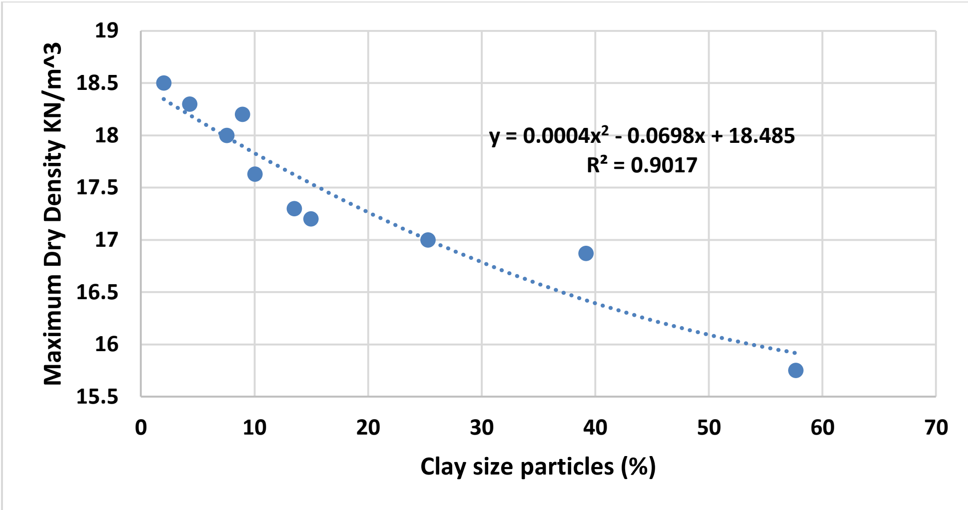

Figure 4.9: Relationship between maximum dry density and %age of clay size particles

Figure 4.10 shows the relationship between MDD and percentage silt contents. The MDD increases with an increase in the %age of silt particles, following a relationship y = -0.0014x^2+0.2176x+9.7237 with coefficient of determination R^2 = 0.6239. It means that a fair relation exists between MDD and %age of silt size particles, similar to that of (Ali et al., 2019).

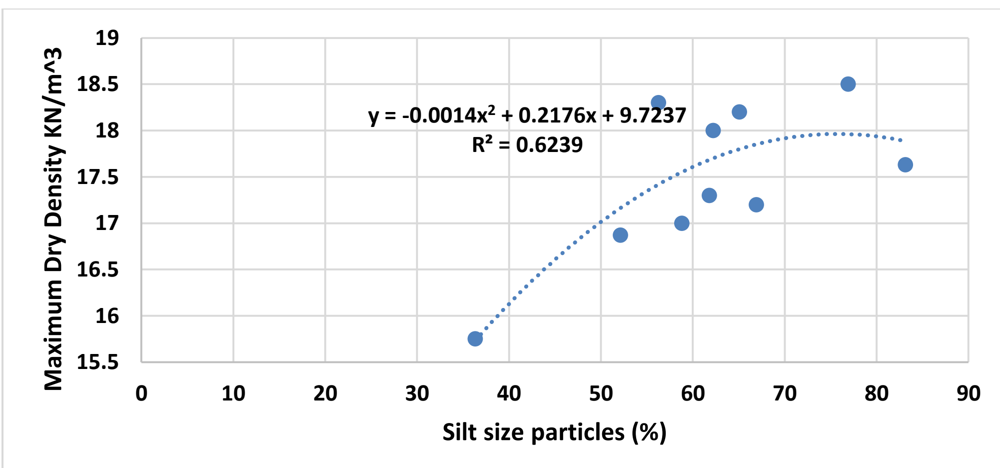

Figure 4.10: Relationship between maximum dry densities and %age silt content

Figure 4.11 shows that a strong relationship y = -2.473ln(x) +25.926 having R^2 = 0.9753 exists between maximum dry density and liquid limit of soils, similar to the finding of (Hassan et al., 2017). The rate of increase in MDD is relatively more towards lower liquid limit than higher liquid limits.

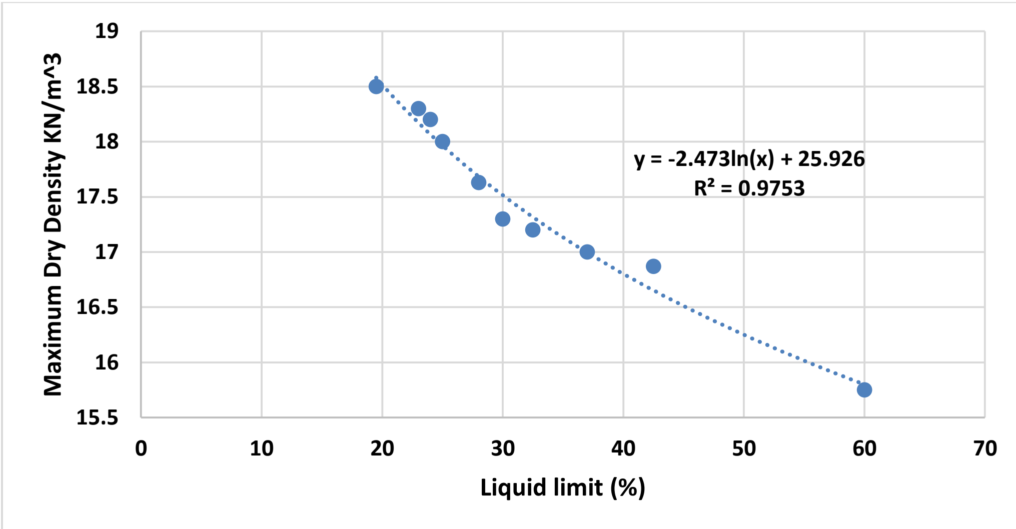

Figure 4.11: Relationship between maximum dry density and liquid limit

Similar to that of liquid limit, a strong relationships exit between MDD and PL and PI as in Figures 4.12 and 4.13, similar to the findings of (Ali et al., 2019) and (Khalid and Rehman, 2018) respectively.

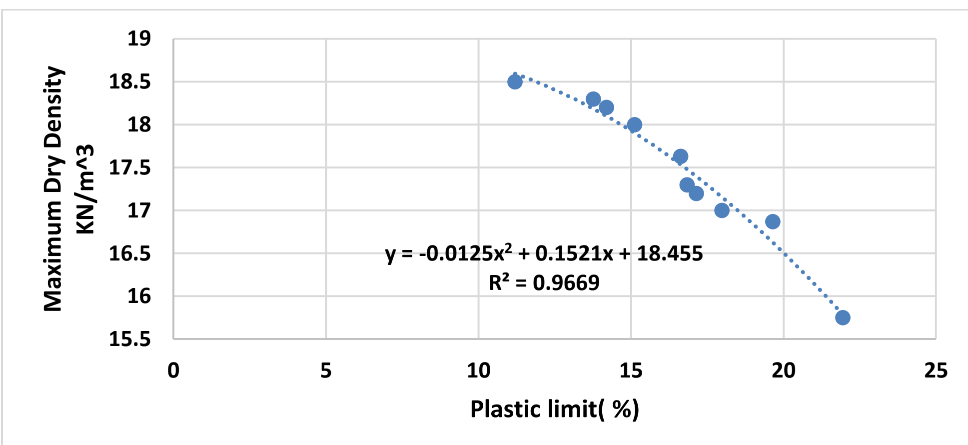

Figure 4.12: Relationship between maximum dry density and plastic limit

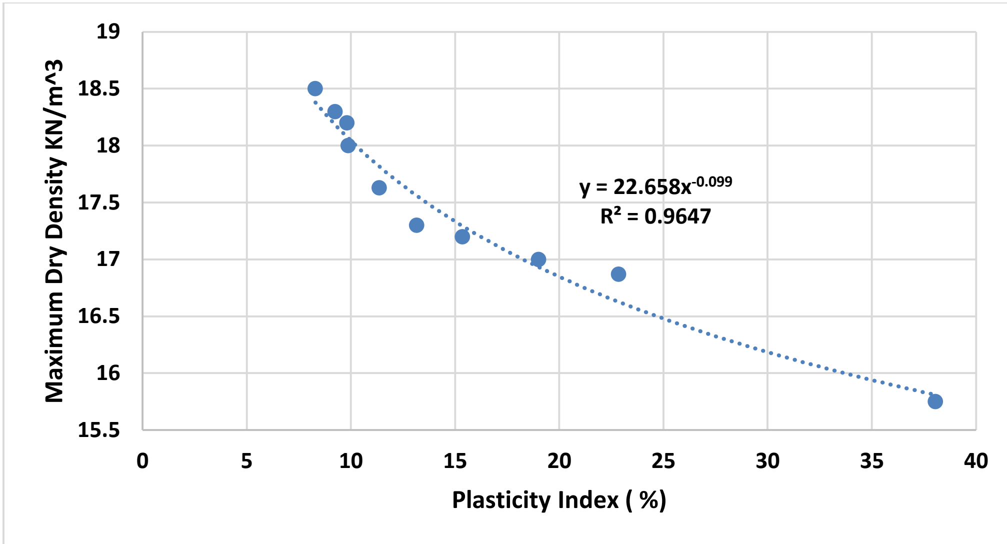

Figure 4.13: Relationship between maximum dry density and plasticity index

Figure 4.14 shows a relationship between optimum moisture content and liquid limit providing an excellent relationship similar to (Ali et al., 2019). This relationship is only for cohesive soil.

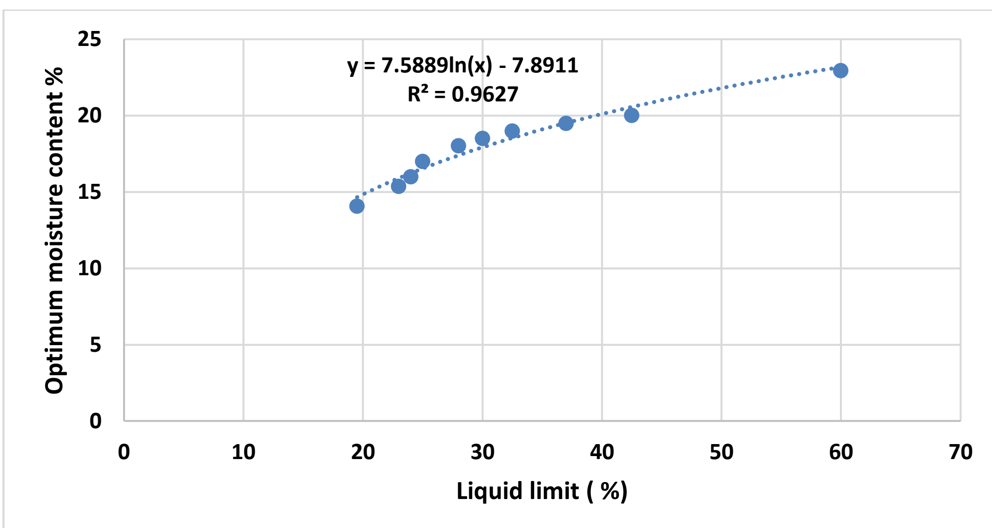

Figure 4.14: Relationship between optimum moisture content and liquid limit

Similar to that of liquid limit, a strong relationships exit between OMC and PL and PI as in Figures 4.15 and 4.16, similar to the findings of (Ali et al., 2019) and (Khalid and Rehman, 2018), respectively.

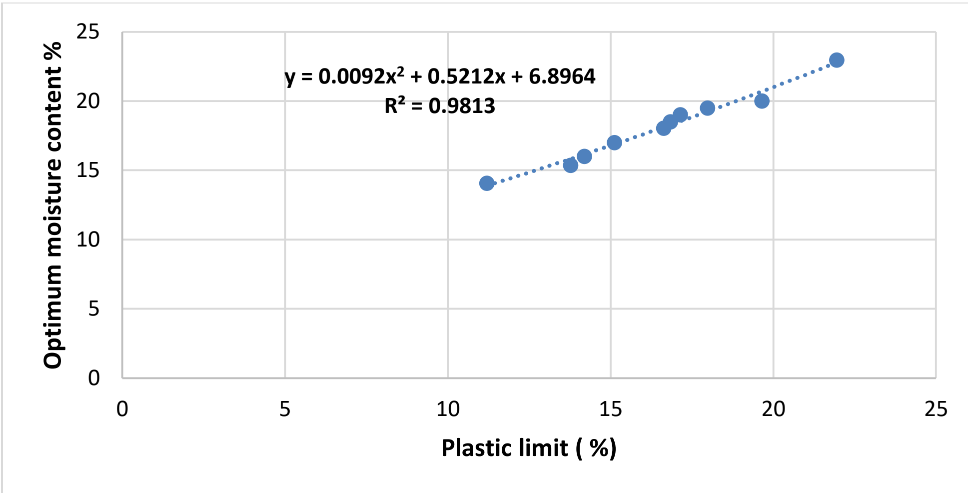

Figure 4.15: Relationship between optimum moisture content and plastic limit

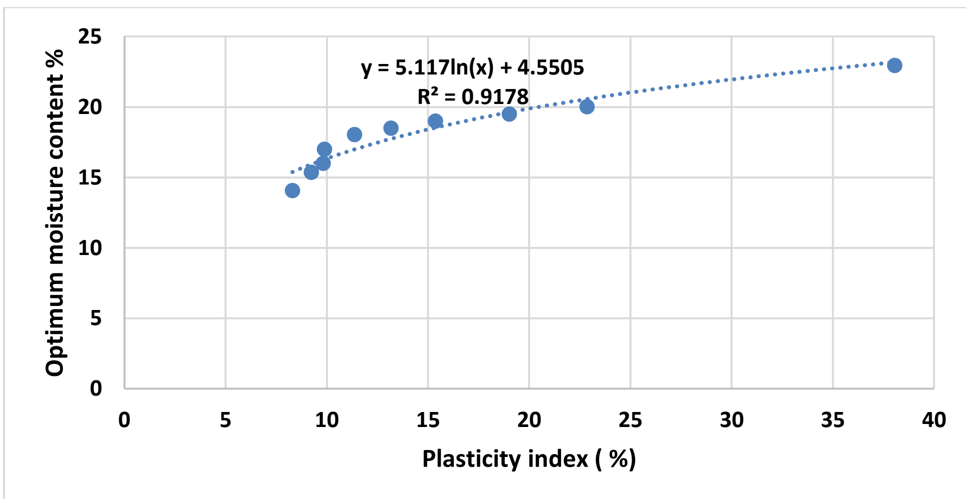

Figure 4.16: Relationship between optimum moisture content and plasticity Index

4.7. Consolidation test result

Figure 4.17 shows the graph between void ratio and preconsolidation pressure. This graph is to be used for the calculation of the compression index and recompression index. Soil having more clay content have more compression index and soil that have less amount of clayey content have less compression index. Compression index is calculated under loading curve while recompression index is to be calculated under unloading curve. It was observed that when pre consolidation pressure increases the void ratio can be decreases.

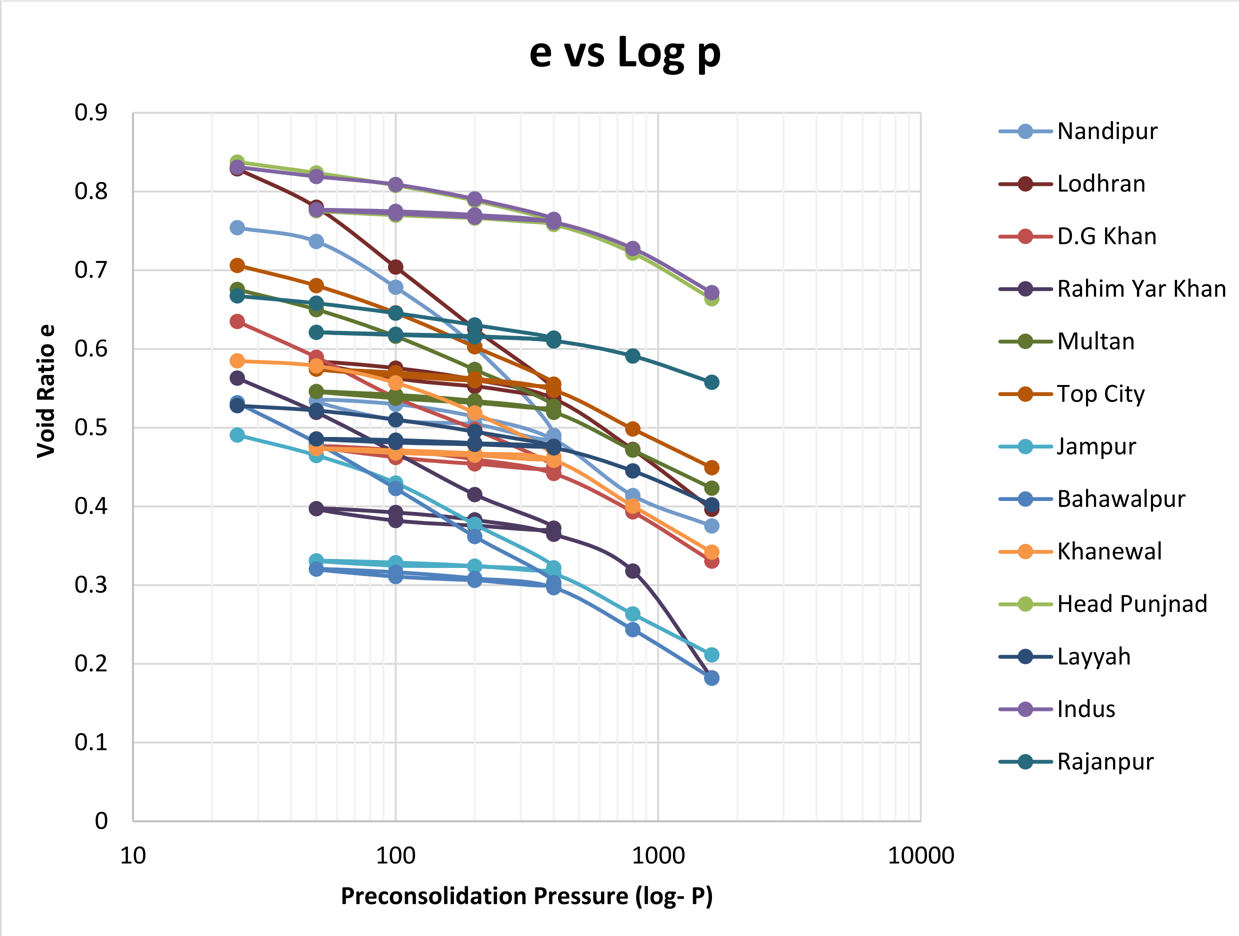

Figure 4.17: Relationship between pre consolidation pressure and void ratio

4.7.1. Relationship between compression index (C_c) and other properties of soil

Figure 4.18 shows the relationship between compression index and liquid limit following a polynomial relationship, such as y = $-0.0002x^2+0.0175x-0.1962$ with R^2 is 0.9192 which means that a good relationship exists between these parameters an in (Sabrin et al., 2021) in which the study reported an excellent relationship between compression index and liquid limit for cohesive soils

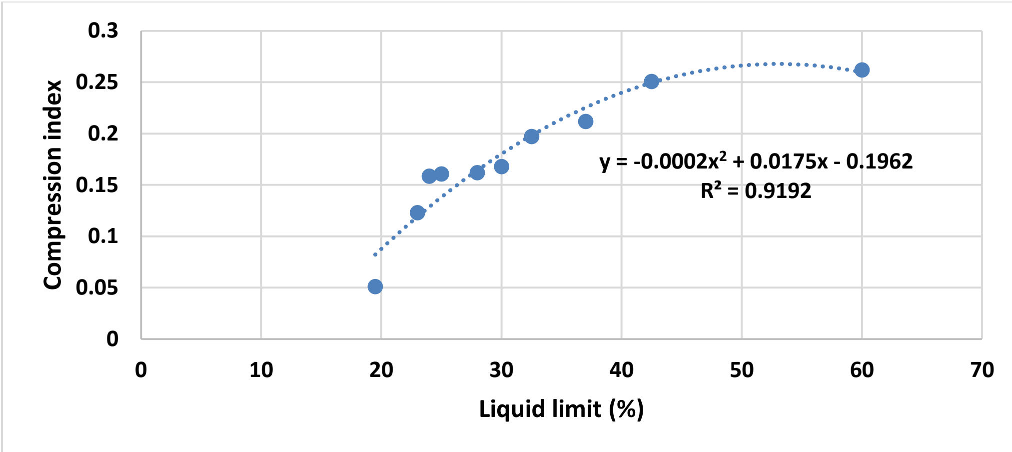

Figure 4.18: Relationship between compression index and liquid limit

Similarly, as in Figures 4.19, 4.20 compression index provides good relationships with plastic limit and plasticity index, similar to that of (Shien et al, 2018) and (Rashed et al., 2017), respectively. Furthermore, as in Figure 4.21, a strong relationship exists between compression index and shrinkage limit too, indicating that compression index increases with an increases in the shrinkage limit, similar to the finding of (Rashed et al., 2017).This relationship is only for cohesive soil.

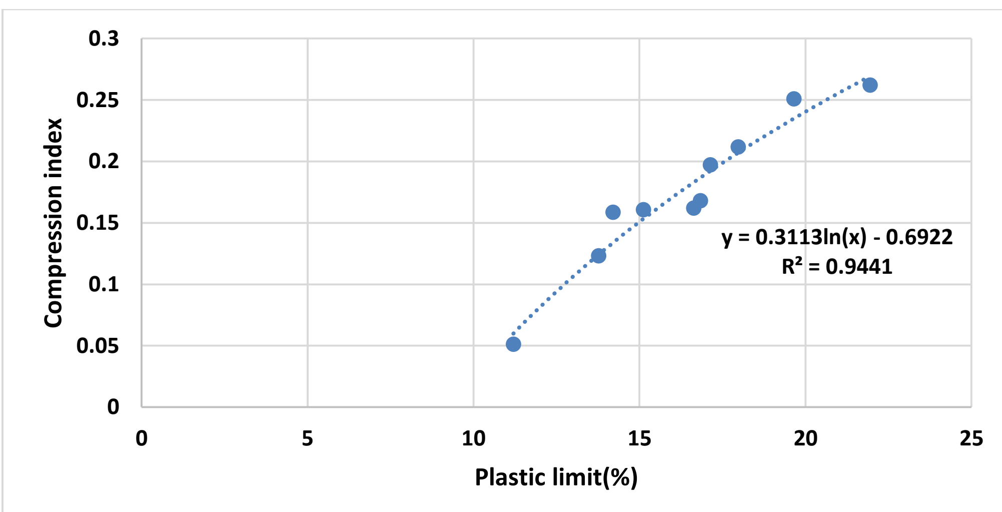

Figure 4.19: Relationship between compression index and plastic limit

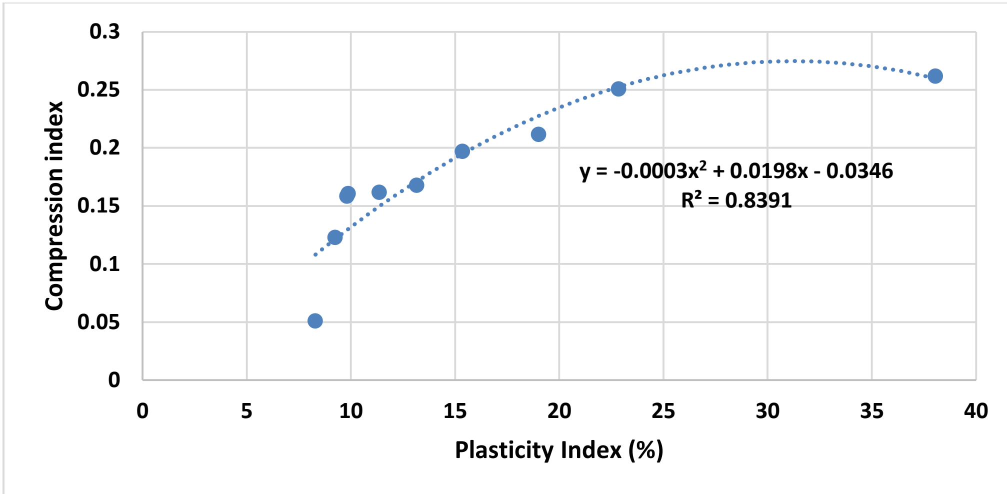

Figure 4.20: Relationship between compression index and plasticity index

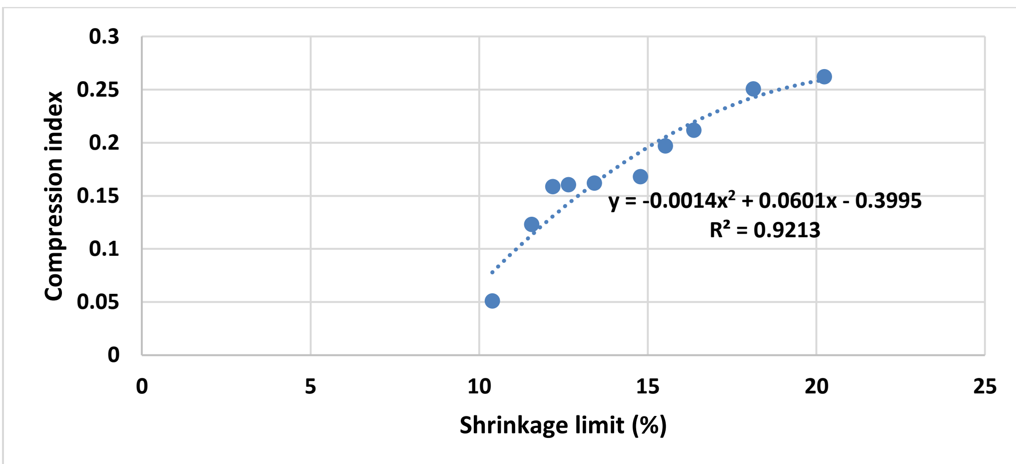

Figure 4.21: Relationship between compression index and shrinkage limit

Figure 4.22 shows relationship between compression index and initial void ratio which indicates that a good relationship, such as $y = 0.3515x^2+0.1579x-0.0804$ exists between compression index and initial void ratio with $R^2 = 0.7856$. The compression index increases with an increase in initial void ratio, similar to the finding of (Majdi et al., 2019). This relationship is only for cohesive soil.

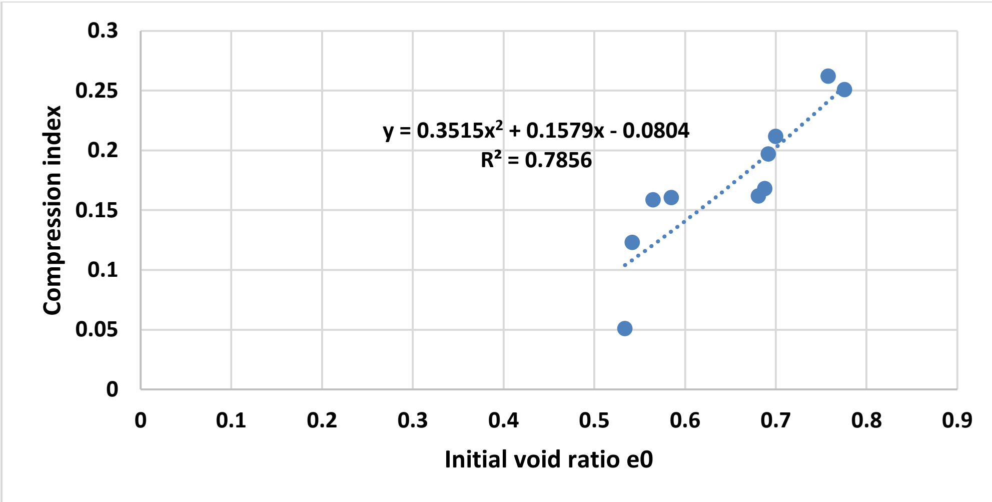

Figure 4.22: Relationship between compression index and initial void ratio

Figure 4.23 shows that compression index increases with an increase in %age of clay size particles, following a relationship, such as y = 0.0603ln(x) +0.0242 with R^2 = 0.9721 which means that an excellent relationship exists between these parameters, similar to the finding of (Amit and Dedalal, 2004). This relationship is only for cohesive soil.

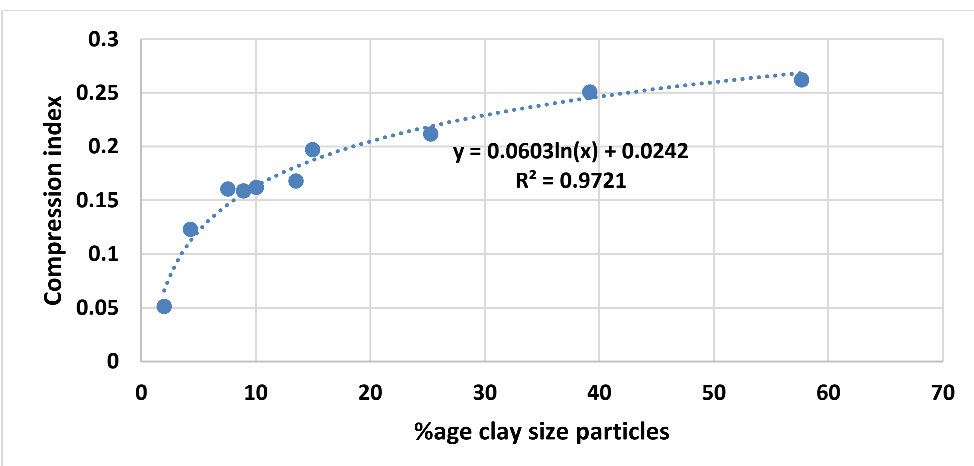

Figure 4.23: Relationship between compression index and %age clay size particles

Figure 4.24 shows that compression index increases with an increase in the optimum moisture content, following a polynomial relationship, such as y = $-0.0014x^2+0.0734x-0.689$ R^2 = 0.9058 which means that an excellent relationship exists between these parameters, similar to the finding of (Arpan and Sujit, 2012). This relationship is only for cohesive soil.

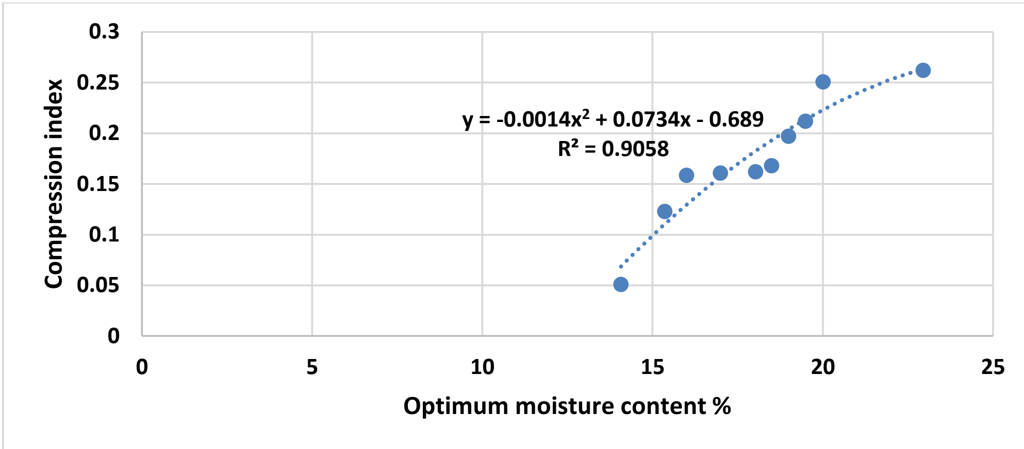

Figure 4.24: Relationship between compression index and optimum moisture content

4.7.2. Relationship between Swelling index (C_s) and other properties of soil

Figure 4.25 shows relationship between swelling index and liquid limit, indicating that swelling index increases with an increase in the liquid limit and the rate of increase in Cs is quite higher for liquid limit up to 35, but after this point, the rate of increase in Cs is minimal towards higher liquid limits. The test data is similar to the findings of (Shien et al, 2018) showing good relationship between Cs and LL.

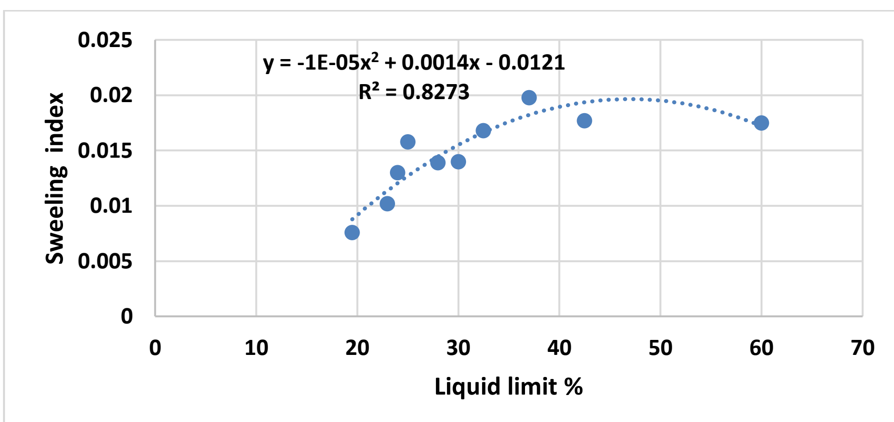

Figure 4.25: Relationship between swelling index and liquid limit

Similar to that of liquid limit, Cs provides good relationships with PL, PI and SL as shown in Figures 4.26, 4.27 and 4.28, similar to the findings of (Shien et al, 2018) and (Rashed et al., 2017).

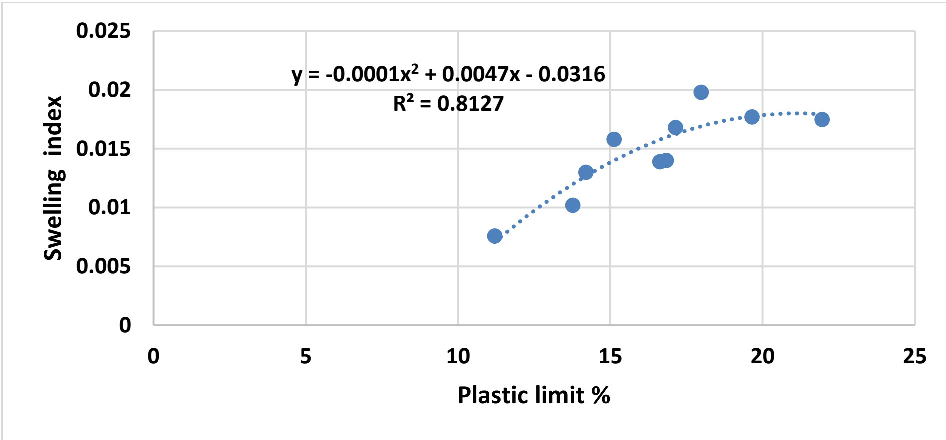

Figure 4.26: Relationship between swelling index and plastic limit

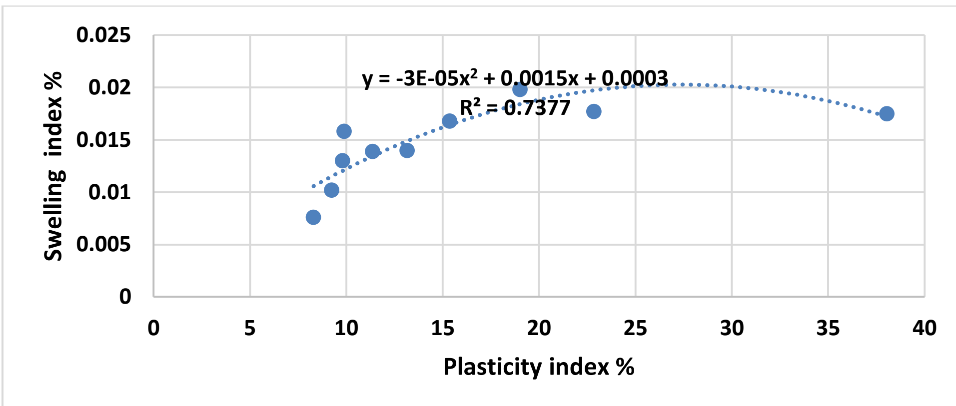

Figure 4.27: Relationship between swelling index and plasticity index

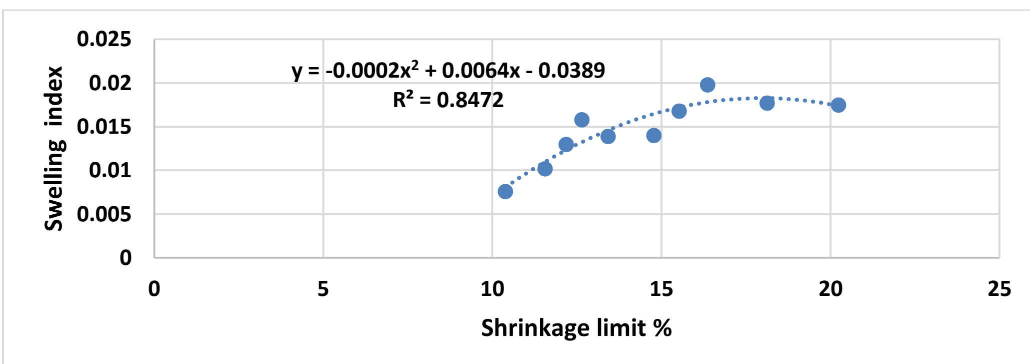

Figure 4.28: Relationship between swelling index and shrinkage limit

Figure 4.29 shows relationship between Cs and specific gravity indicating that similar to liquid limit, Cs increases with an increase in specific gravity following a relationship, such as y = -2.8687x^2+15.72x-21.517 and R^2 = 0.8772, similar to the findings of (Kordnaeij et al., 2015).

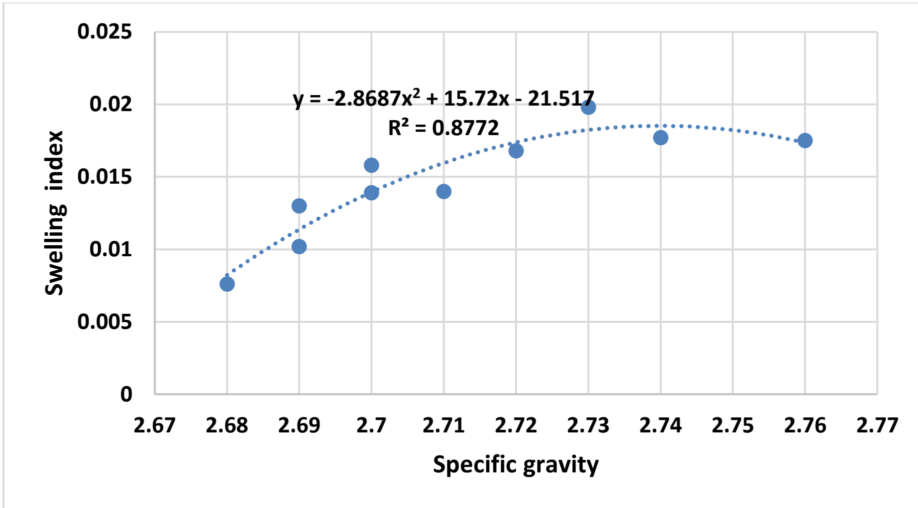

Figure 4.29: Relationship between swelling index and specific gravity

Figure 4.30 shows that an inverse relationship exists between C_s of the soil and maximum dry density, following a polynomial relationship, such as $y = -0.0021x^2+0.0699x-0.555$ and $R^2 = 0.7871$ which means that maximum dry density of various soils provides a good relationship with C_s, similar to the finding of (Kordnaeij et al., 2015). Similarly, as in Figure 4.31, a direct relationship exists between Cs and optimum moisture content following a relationship, such as $y =0.0002x^2+0.008x-0.0684$ with $R^2 = 0.8475$, similar to the work of (Kordnaeij et al., 2015) in which the optimum moisture content provided good relationship with Cs for various clayey soils.

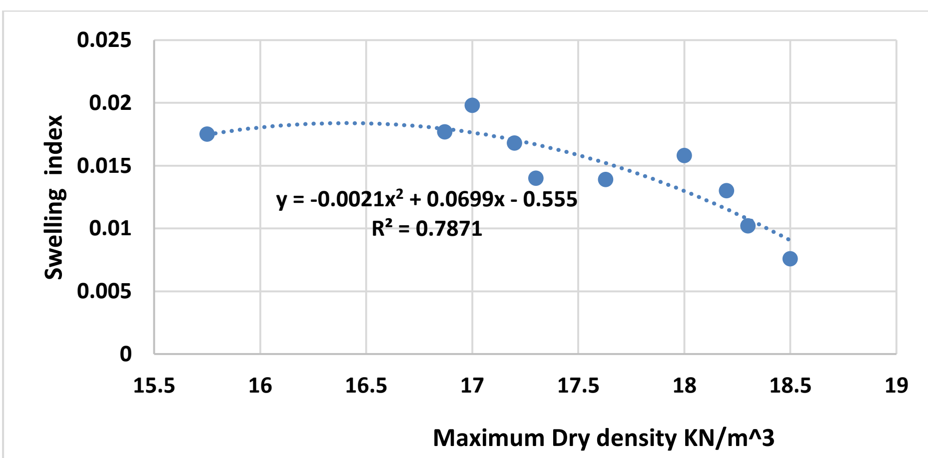

Figure 4.30: Relationship between swelling index and maximum dry density

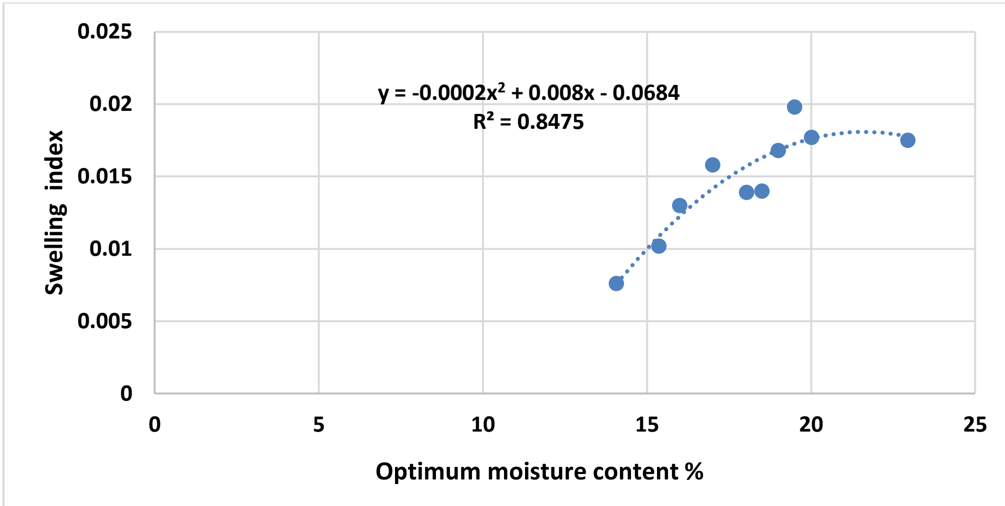

Figure 4.31: Relationship between swelling index and optimum moisture content

Figure 4.32 shows that a direct relationship exists between Cs and initial void ratio, providing a relationship, such as $y = -0.1588x^2+0.239x-0.0726$ with $R^2 = 0.6965$ means that there is a fair relationship between these parameters, similar to the finding of (Kurnaz et al., 2016). It can be seen from Figure 4.32 that the rate of increase in C_s is more for initial void ratio up to 0.7, but after this point though C_s provides and increasing trend but the rate of increase is relatively low.

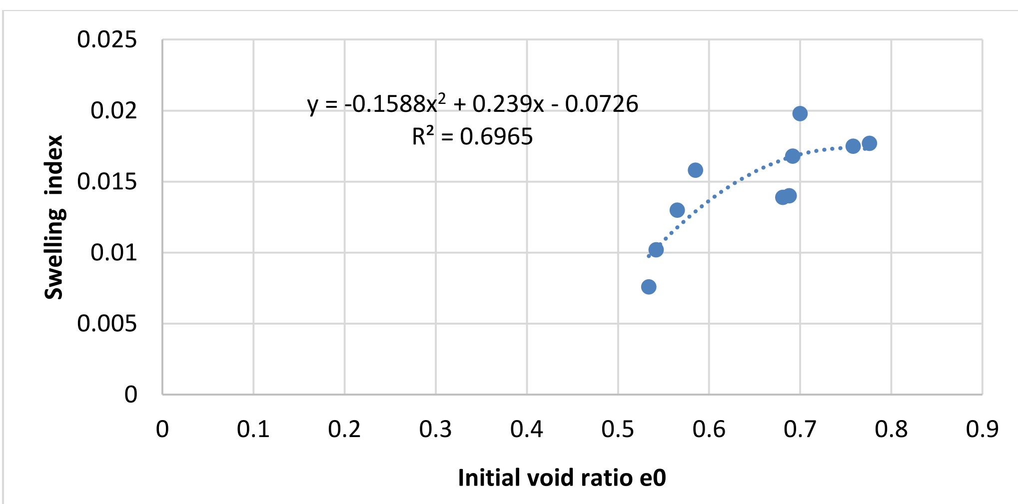

Figure 4.32: Relationship between swelling index and initial void ratio

4.7.3. Relationship between coefficient of consolidation (C_v) and other properties of the soil

Figure 4.33 shows that an inverse relationship exists between coefficient of consolidation and liquid limit content, following a relationship, such as $y = 4E^{-06}x^2-0.0004x+0.0317$ with $R^2 = 0.9652$ which means that there is an excellent relationship between these parameters, similar to the finding of (Sridharan and Nagaraj, 2012). Similarly, as in Figures 4.34, 4.35 and 4.36, the coefficient of consolidation provides good relationships with PL, PI and SL, similar to the finding of (Shien et al, 2018)and (Sridharan and Nagaraj, 2012). It is clear from Figures 4.33 to 4.36 that coefficient of consolidation provides inverse relationships with Atterberg limits.

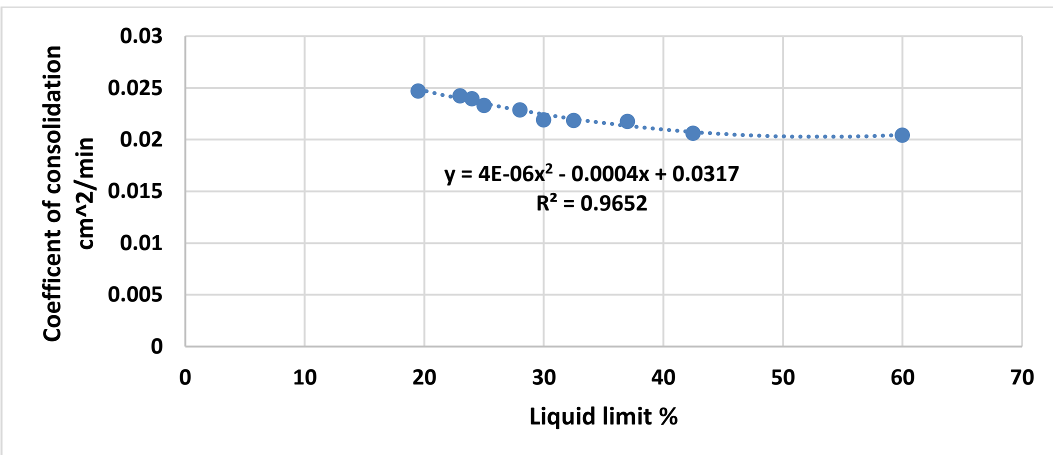

Figure 4.33: Relationship between coefficient of consolidation and liquid limit

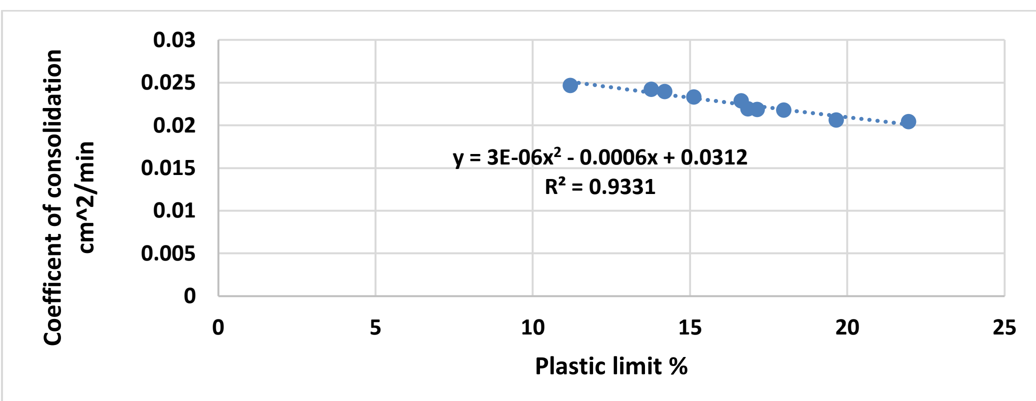

Figure 4.34: Relationship between coefficient of consolidation and plastic limit

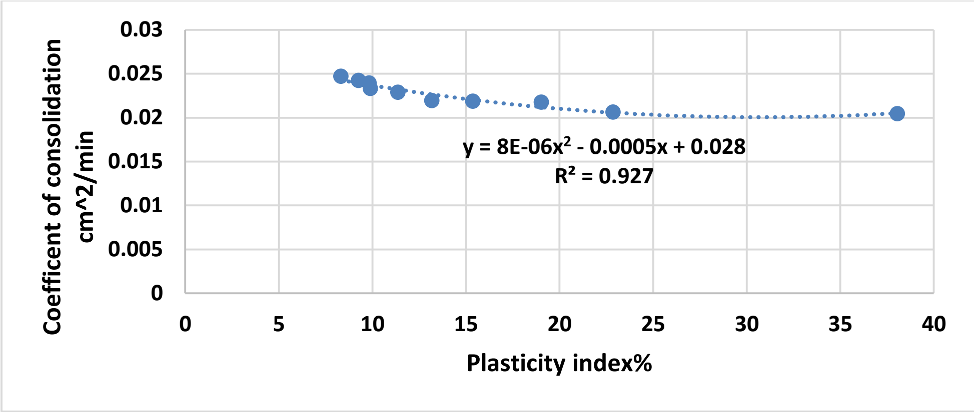

Figure 4.35: Relationship between coefficient of consolidation and plasticity index

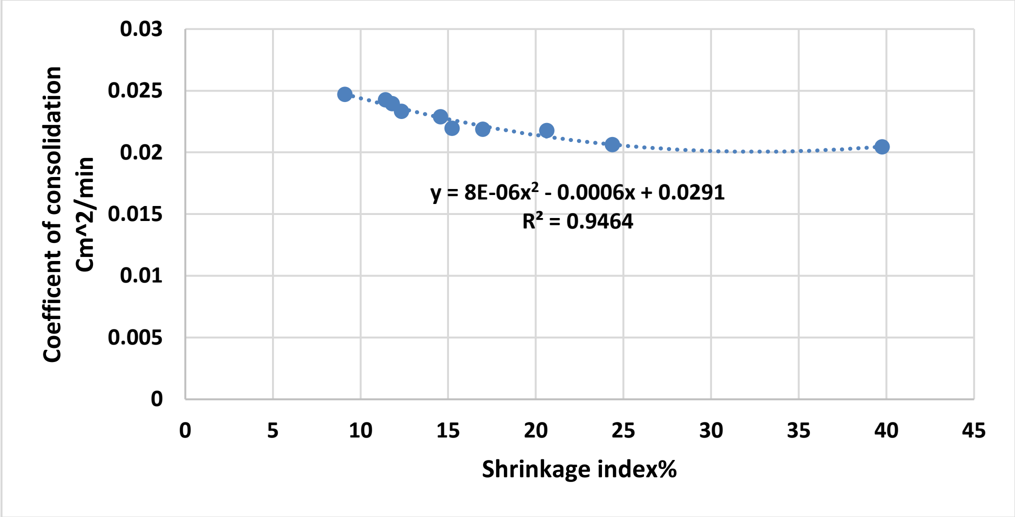

Figure 4.36: Relationship between coefficient of consolidation and shrinkage index

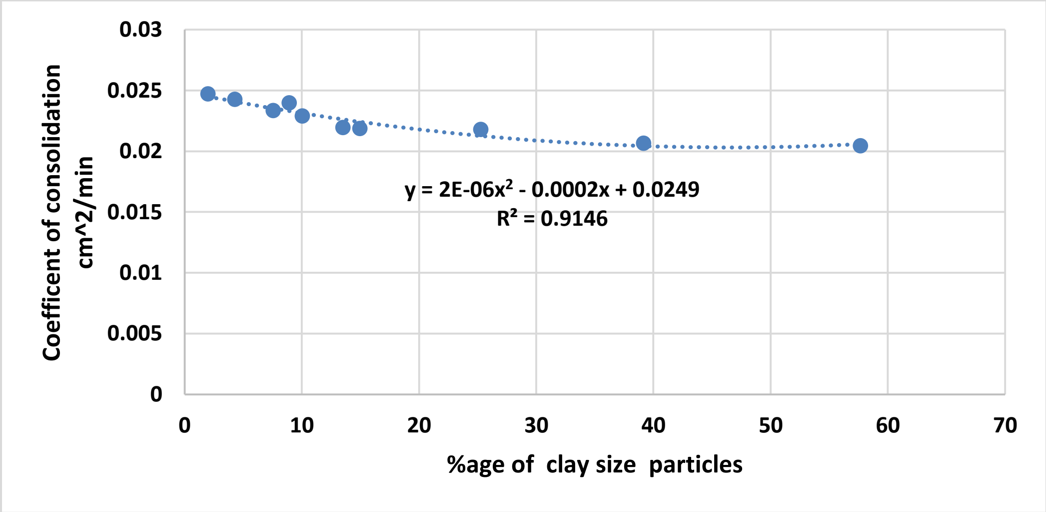

Figure 4.37: Relationship between coefficient of consolidation and %age of clay size particles

Similar to that of Atterberg limits, the coefficient of consolidation provides an inverse relationship percent clay contents, following a relationship, such as $y=2E^{-06}x^2-0.0002x+0.0249$ with coefficient of determination R^2 = 0.9146. The test data is similar to the finding of (Shien et al, 2018)for cohesive soils.

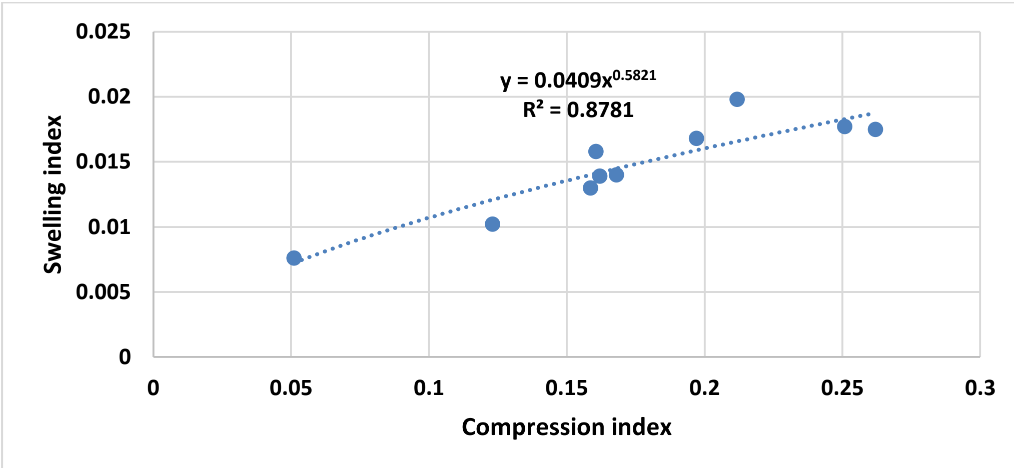

Figure 4.38: Relationship between compression index and swelling index

Figure 4.38 shows the relationship between Cs and Cc indicating that Cs increases with an increase in Cc of the soil. A strong relationship, such as $y = 0.0409x^{0.5821}$ with R^2 = 0.8781 exits between these parameters, similar to the test data of *(Gunduz and Arman, 2007)*for cohesive soils.

4.8. Consolidated undrained triaxial test result

4.8.1. Relationship between shear strength parameters with other properties

Figure 4.39 shows the relationship between cohesion and angle of internal friction which shows that a strong correlation, such as $y= 0.0189x^2-1.5436x+31.969$ with R^2=0.9819 exist between cohesion and angle of internal friction. Nandipur soil shows maximum cohesion and minimum angle of internal friction value whereas Head Punjnad soil shows minimum cohesion and maximum angle of internal friction. The difference in shear strength parameters is due to different in %ages of clay contents and Atterberg limits.

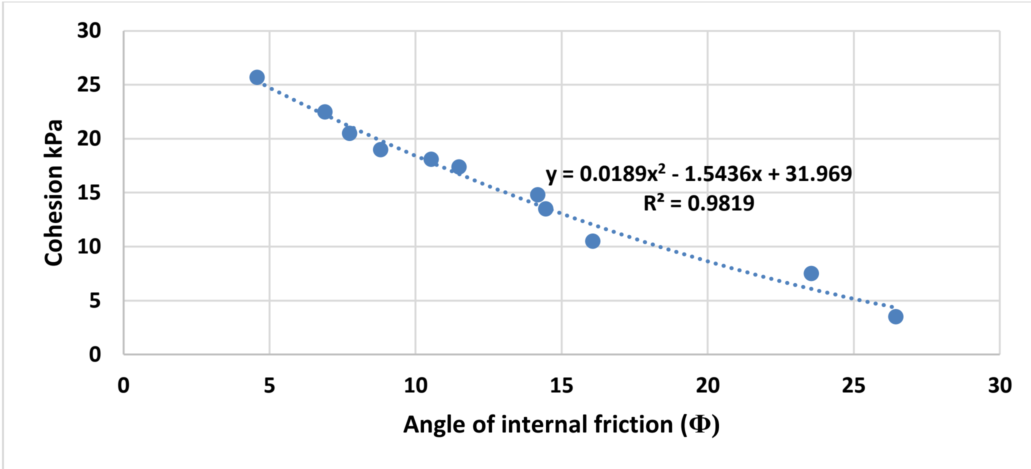

Figure 4.39: Relationship between cohesion and angle of internal friction

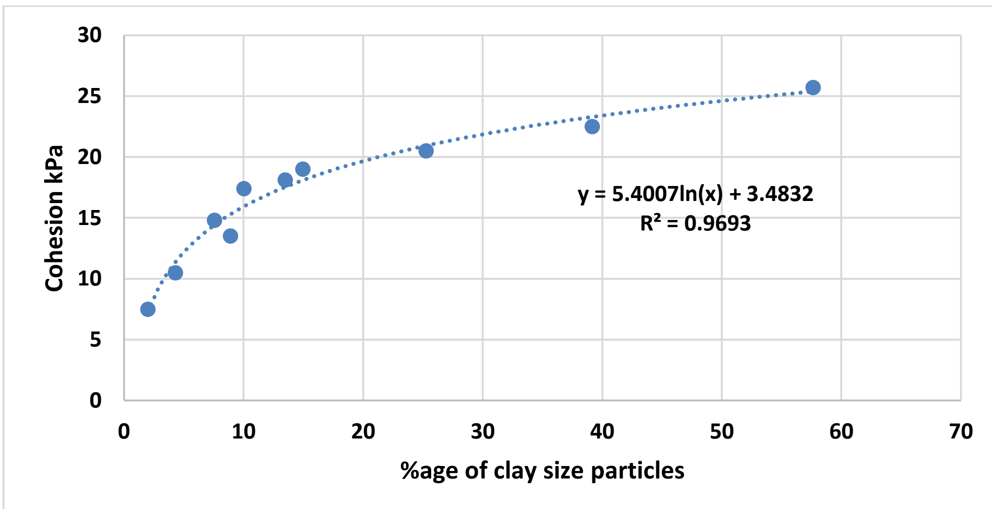

Figure 4.40: Relationship between cohesion and %age clay size particles

Similarly, as in Figure 4.40, a direct relationship exists between cohesion and clay size particles too, following a relationship, such as y = 5.4007ln(x) +3.4832 with R^2=0.9693. The test data is in good agreement with the findings of (Akayuli et al., 2013).

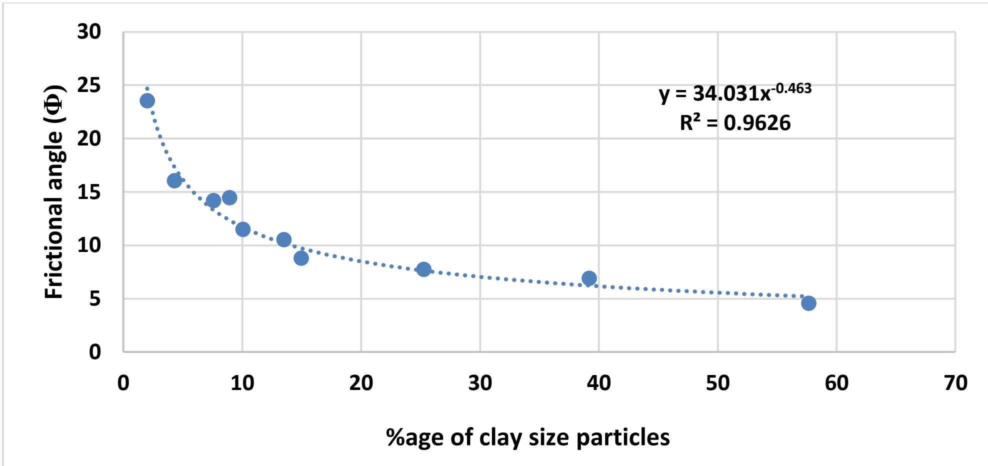

Figure 4.41: Relationship between frictional angle and %age of clay size particles

As in Figure 4.41, an inverse relationship exists between angle of internal friction and %age of clay size particles, following a relationship, such as $y = 34.031x^{-0.463}$ with R^2=0.9626. The test data is in a good agreement with the findings of (Akayuli et al., 2013).

4.10. CBR test result

Figure 4.42represents the relationship between penetration and applied stress for different soil, which shows that Rajanpur soil provides maximum stress than other soils, which is due to its coarser particles. The Nandipur soil shows lower stress, which is due to higher %age of clay contents and finer grain size. Furthermore, as in Table 4.7, the CBR varies from 6.124 to 19.1 depending upon the soil type.

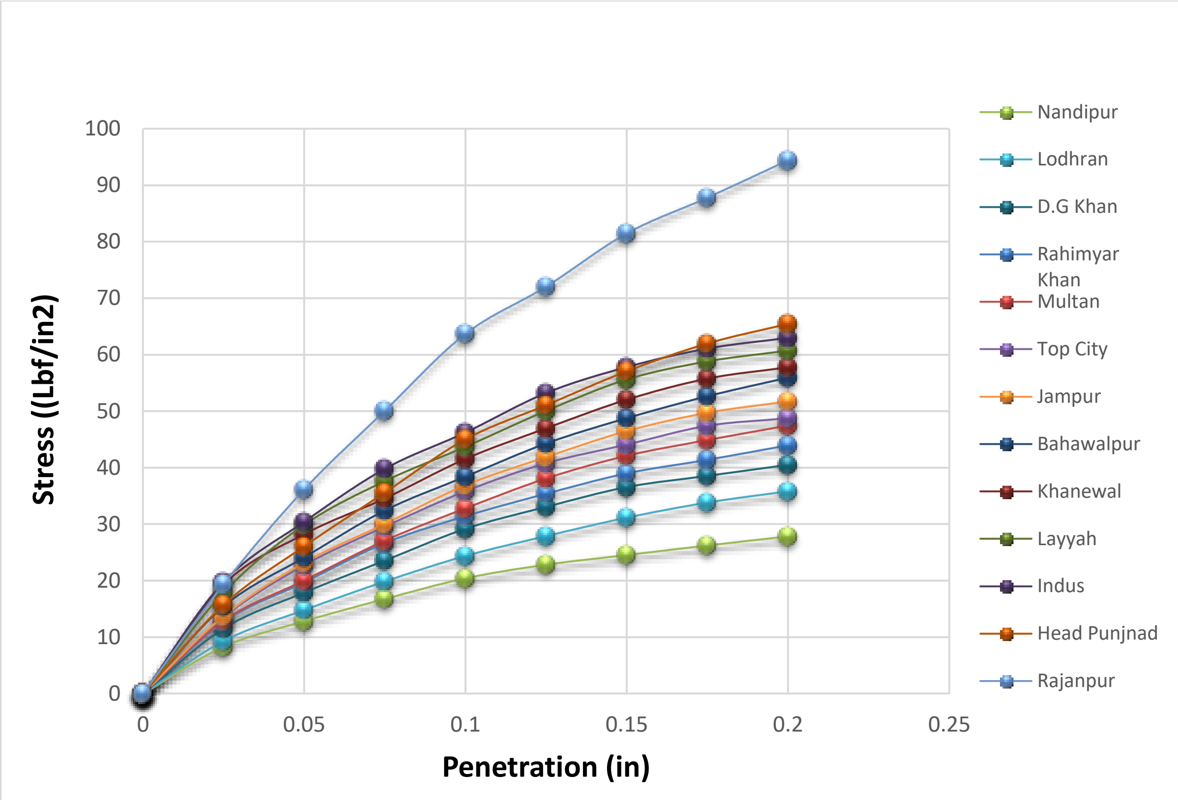

Figure 4.42: Relationship between stress and penetration

Table 4.5: Soil CBR rating according to E.Mina et.al (2019)

Sample Name	**CBR**	**General rating**
Nandipur	6.124	Poor to fair
Lodhran	7.3	Fair
D.G Khan	8.75	Fair
Rahimyar Khan	9.42	Fair
Multan	9.84	Fair
Top City	10.75	Fair
Jampur	11.06	Fair
Bahawalpur	11.52	Fair
Khanewal	12.46	Fair
Layyah	13.11	Fair
Head Punjnad	13.5	Fair
Indus	13.86	Fair
Rajanpur	19.1	Fair

4.10.1. Relationship between CBR and other properties of soil

Figure 4.43 shows that an excellent relationship, i.e., $y=2E+12x^{-26.2}$ with $R^2= 0.9827$ exists between specific gravity and soaked CBR which reports that CBR decreases with an increase in specific gravity of soil. This is due to the fact that inorganic clayey soils provide more specific gravity and lower maximum dry density than sandy soils. Similar findings were reported by (Yashas et al., 2016) stating that a strong relationship occurred between specific gravity and CBR.

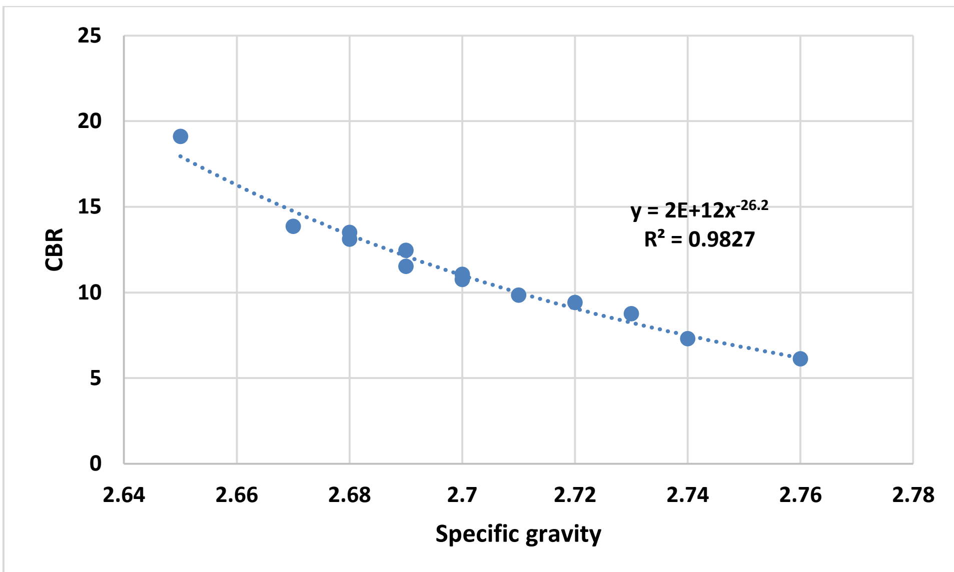

Figure 4.43: Relationship between CBR and specific gravity

Similarly, as shown in Figures 4.44, 4.45 and 4.46, an excellent relationship exists between CBR and Atterberg limits, similar to the findings of (Sharma et al., 2020) and (Roksana et al., 2018). The CBR decreases with an increase in the liquid limit having a relationship, i.e., $y= -5.242\ln(x) +27.82$ with $R^2=0.9532$. Similarly, CBR decreases with an increase in PL and PI following relationships, i.e., $y = 0.0054x^2-0.9605x+24.458$ with $R^2 = 0.9691$ and $y = 35.781x^{-0.491}$ with $R^2 = 0.9792$, respectively.

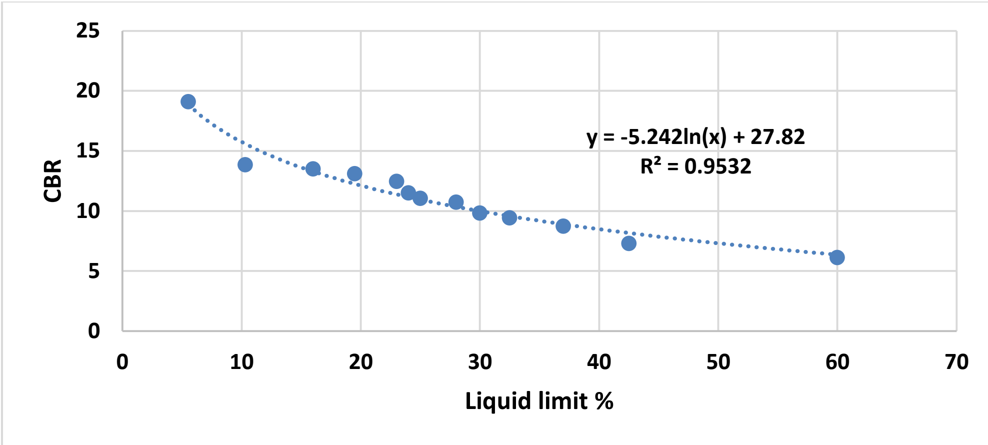

Figure 4.44: Relationship between CBR and liquid limit

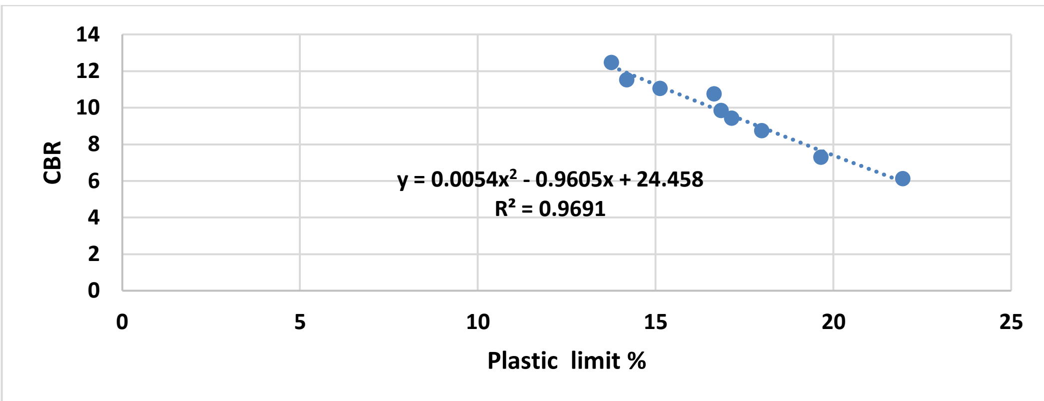

Figure 4.45: Relationship between CBR and plastic limit

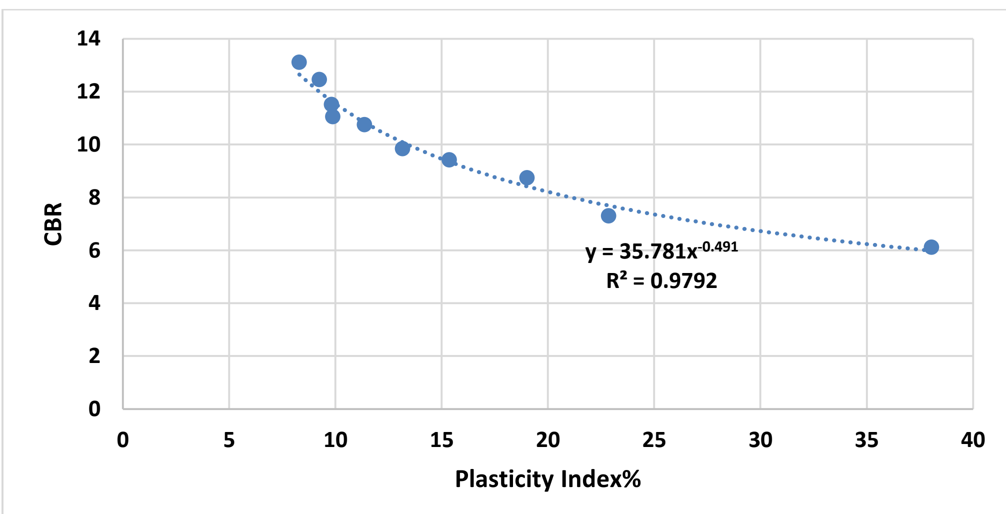

Figure 4.46: Relationship between CBR and plasticity index

Figure 4.47 showed an inverse relationship exists between soaked CBR and %age of silt- clay particle sizes, following a relationship $y = 8E^{-05}x^2-0.118x+19.352$ with $R^2=0.6651$. The test data is in good agreement with the findings of (Reddy et al., 2019).

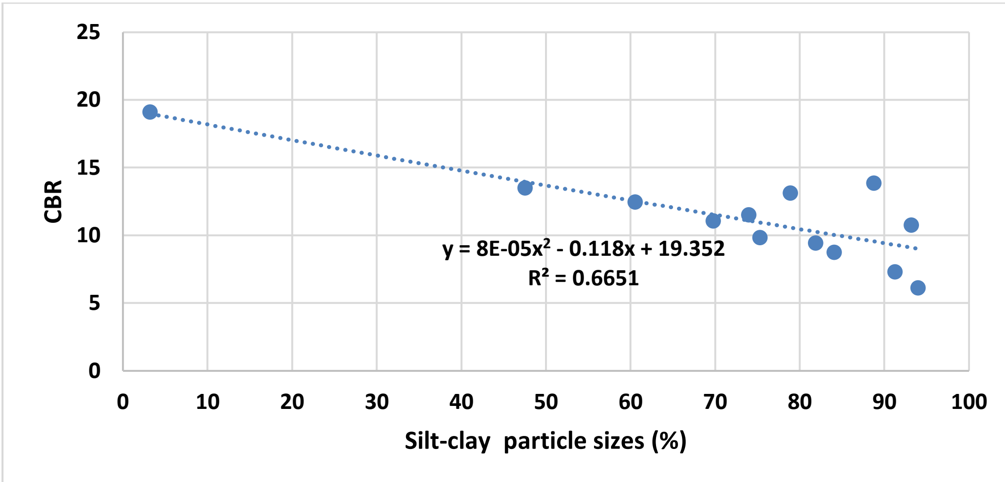

Figure 4.47: Relationship between CBR and %age of silt-clay size particles

Figure 4.48 shows that CBR increases with an increase in maximum dry density as expected, following a relationship, i.e., $y = 0.2946x^2-7.5546x+51.917$ with $R^2=0.0813$ at 65. It is clear that an excellent relationship exists between CBR and MDD, same as in (Korde and Yadav, 2015)

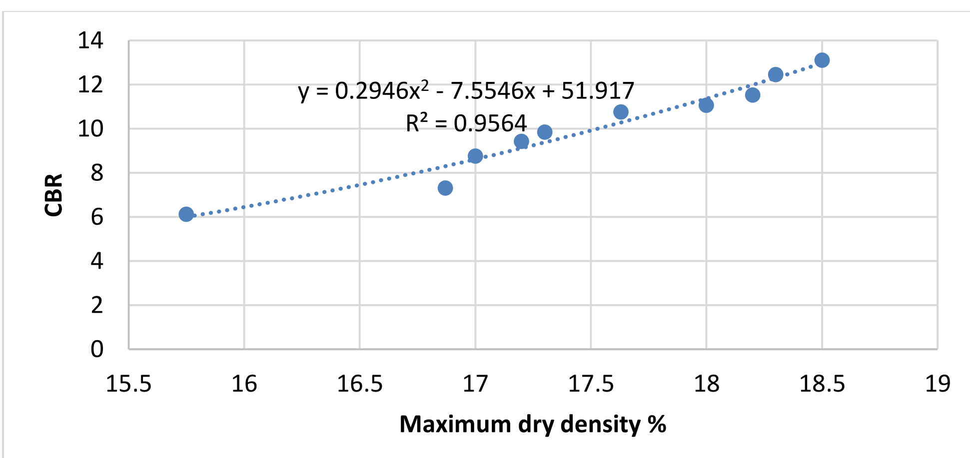

Figure 4.48: Relationship between CBR and maximum dry density

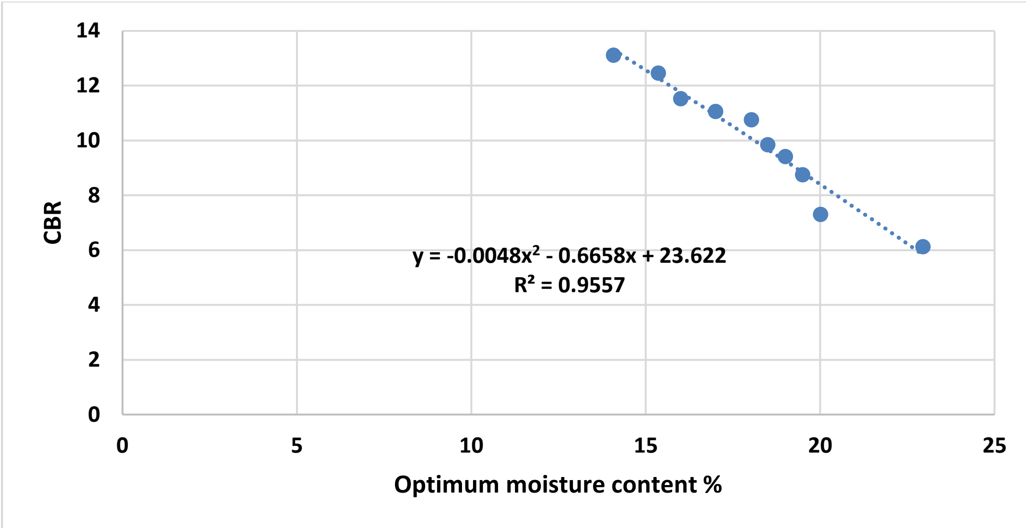

Figure 4.49: Relationship between CBR and optimum moisture content

Figure 4.49 shows a relationship between CBR and optimum moisture content, indicating that an inverse relationship exists between these parameters following relationship y = $-0.0048x^2-0.6658x+23.622$ with $R^2=0.9557$. The test data behaves in a similar fashion to that of (Sharma et al., 2020) showing that an excellent correlation exist between these parameters.

Furthermore, as in Figure 4.50 and 4.51, CBR increases with an increase in angle of internal friction and decreases with an increase in cohesion, similar to that of (Yashas et al., 2016). The relation $y= -0.0125x^2+0.0197x+13.617$ with $R^2=0.9889$ shows an excellent relationship between cohesion and CBR. Furthermore, the linear relationship such as $y= -0.0189x^2+0.9078x+2.4689$ with $R^2=0.9781$ also indicates an excellent relationship between angle of internal friction and CBR.

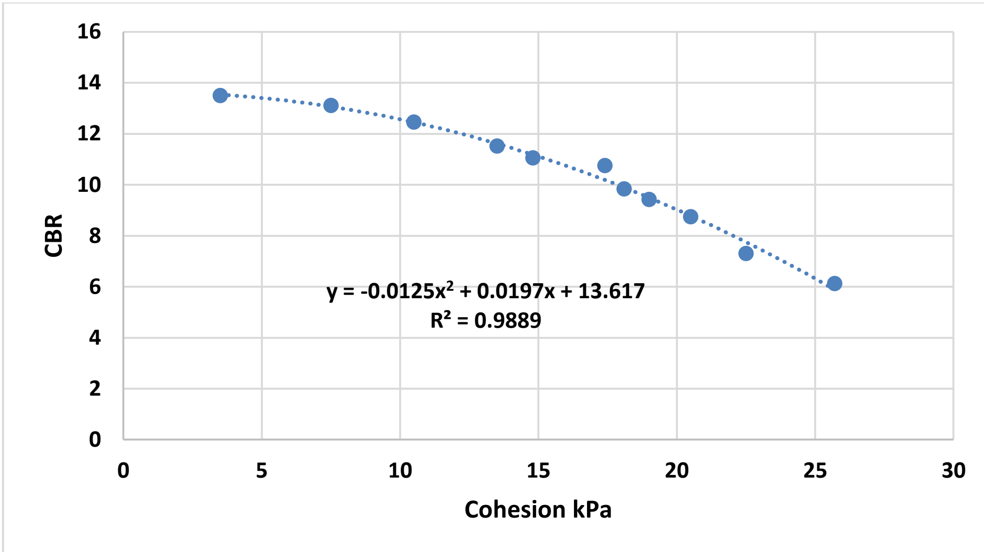

Figure 4.50: Relationship between CBR and cohesion

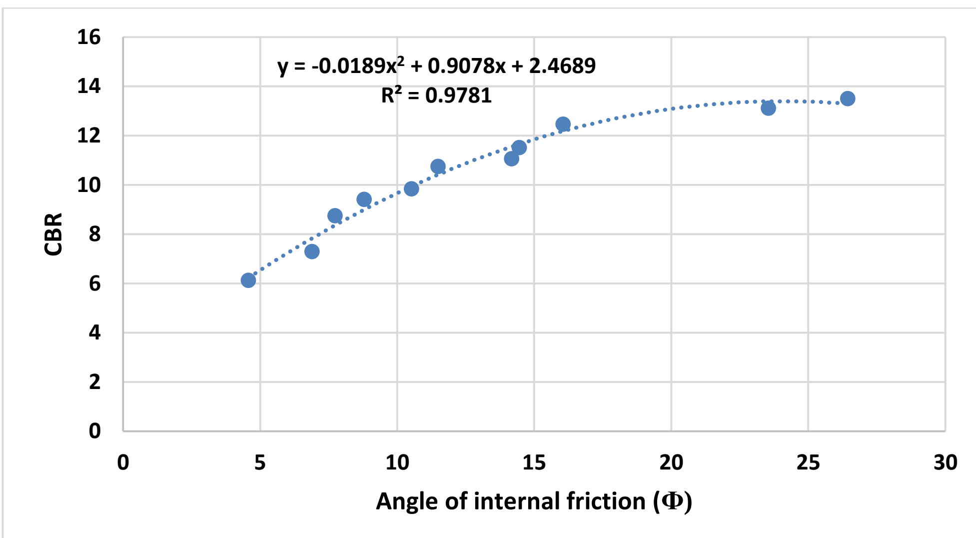

Figure 4.51: Relationship between CBR and angle of internal friction

4.11. Electrical conductivity and pH test results

Table 4.6 represents the electrical conductivity and pH values for different soils. The test data shows that electrical conductivity varies from 0.21 to 0.34 µS/cm which is minimum for Nandipur soil, and maximum for Layyah soil. Similarly, pH of various soils in this study varies from 6.84 to 11.1 Rajanpur soil is acidic in nature as its pH is < 7, and other soils are basic in nature due to their pH>7. It is clear from Figure 4.52 that the relationship between pH and electrical conductivity is very poor which is due to the fact that other parameters, such as soil composition, mineralogy, soil structure, temperature and water content in addition to pH also influence the EC. Similarly, EC and pH also unable to provide relationships with PI, shear strength parameters and CBR as shown in Figures 4.53, 4.54, 4.55, 4.56, 4.57, 4.58, 4.59, 4.60 and 4.61 respectively

Table 4.6: Summary of electrical conductivity and pH

Soil Sample	**Electrical Conductivity**		**PH**	
	Value	**Temp**	**Value**	**Temp**
Indus River Soil	0.22	16.8	7.74	18.2
Lodhran Soil	0.32	18.5	7.75	18.8
Multan Soil	0.22	16.8	8.09	18.1
Rajanpur Soil	0.24	17.3	6.84	18.3
Bahawalpur Soil	0.25	16.8	8.22	17.9
Headpunjnad Soil	0.22	16.8	8.15	16.9
Jampur Soil	0.24	16.7	9.38	18.7
Khanewal Soil	0.22	17.8	8.83	16.3
Layyah Soil	0.34	17.7	7.63	18.7
Rahim Yar Khan Soil	0.27	17	11.1	18.8
D.G KHAN Soil	0.18	17.2	10.93	18.3
Nandipur Soil	0.21	17.3	8.22	18.1
Top City	0.22	17.8	8.21	18.4

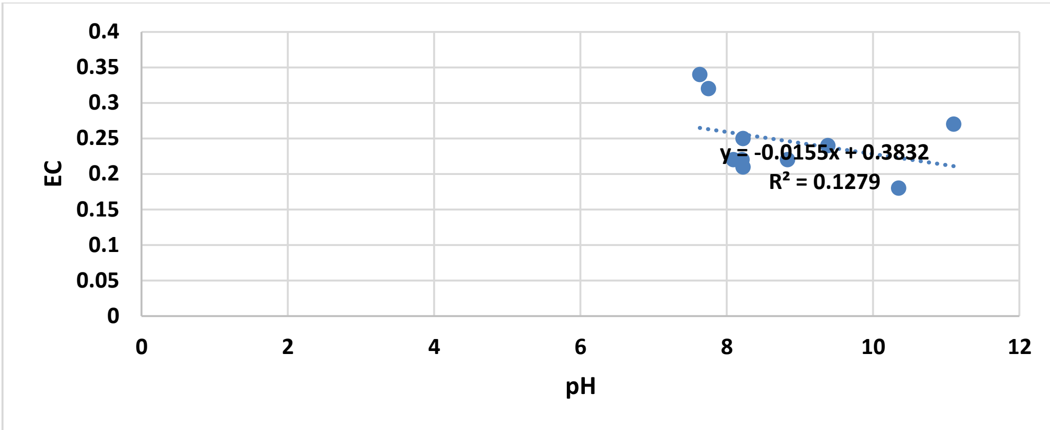

Figure 4.52: Relationship between EC and pH

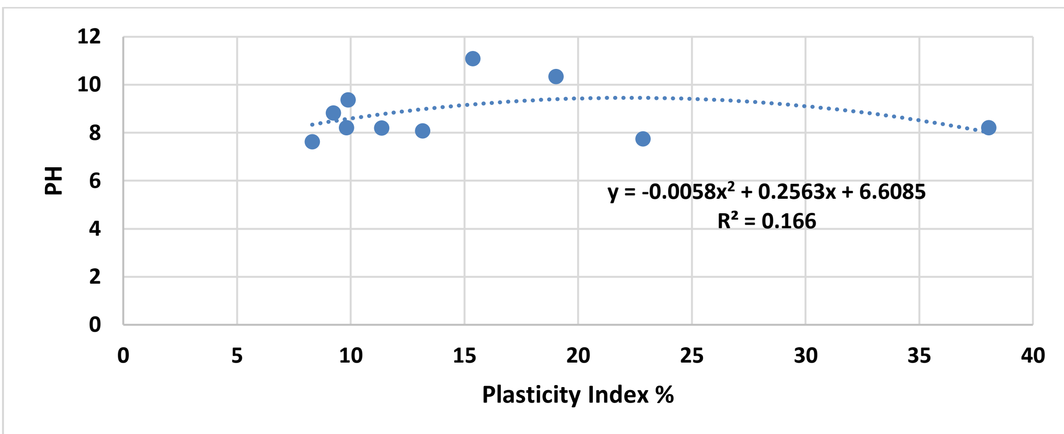

Figure 4.53: Relationship between pH and PI

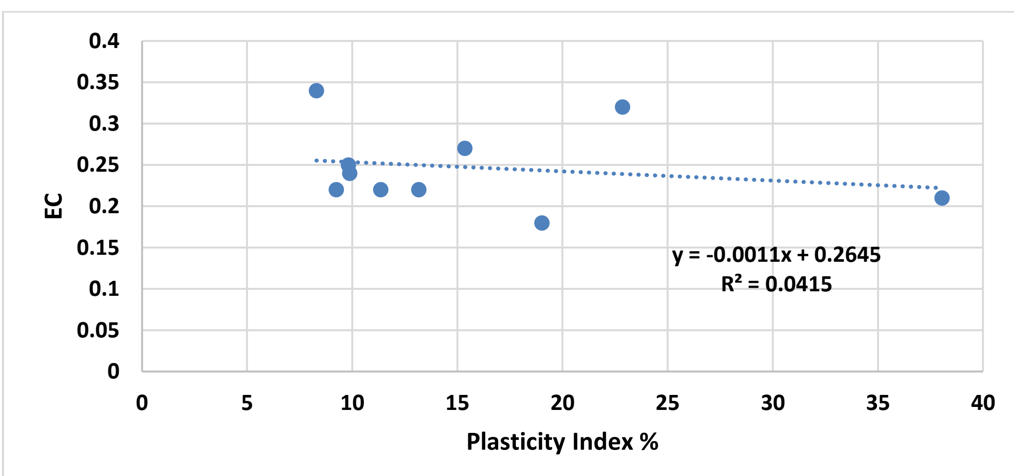

Figure 4.54: Relationship between EC and PI

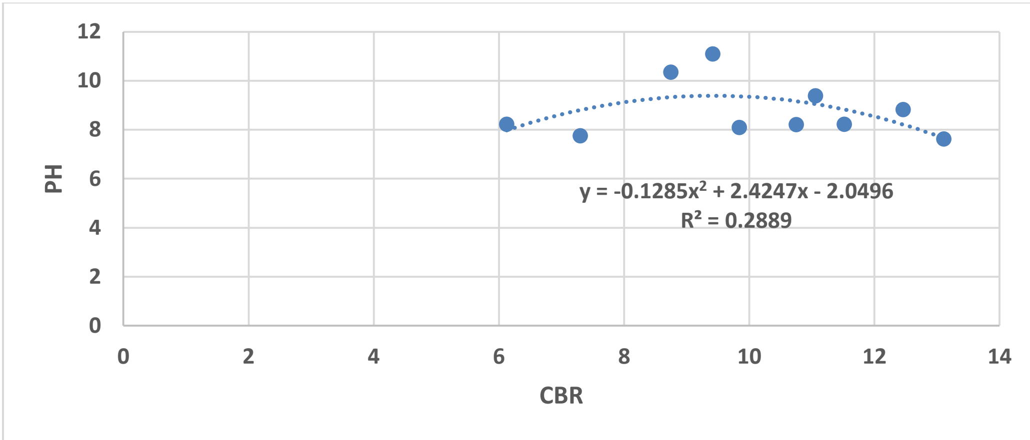

Figure 4.55: Relationship between pH and CBR

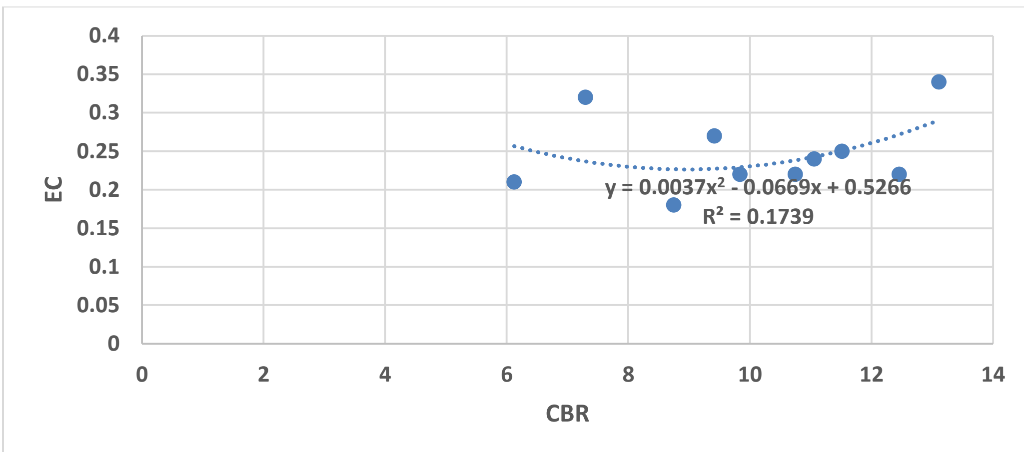

Figure 4.56: Relationship between EC and CBR

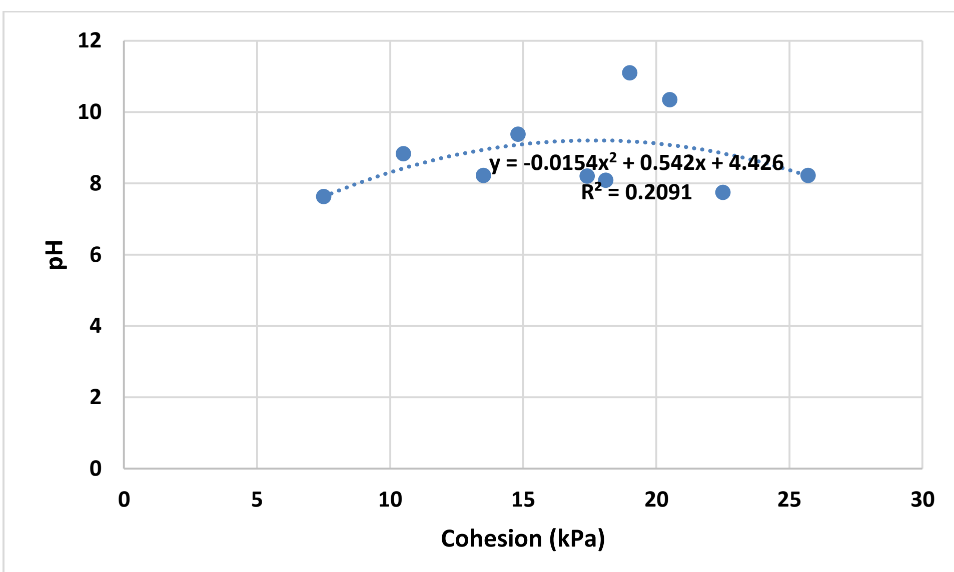

Figure 4.57: Relationship between pH and cohesion

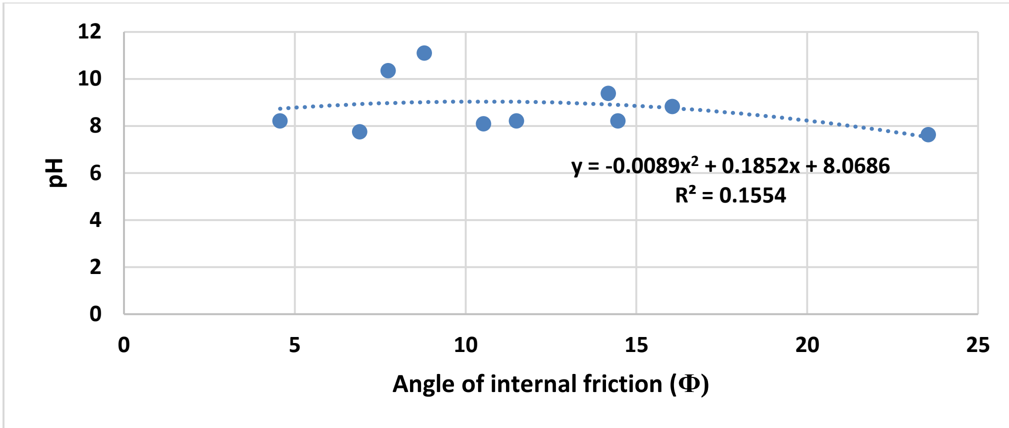

Figure 4.58: Relationship between pH and angle of internal friction

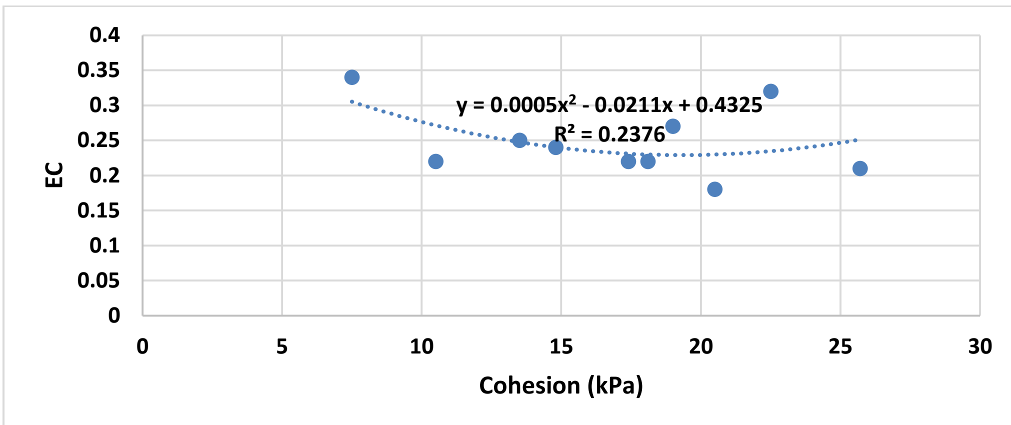

Figure 4.59: Relationship between EC and cohesion

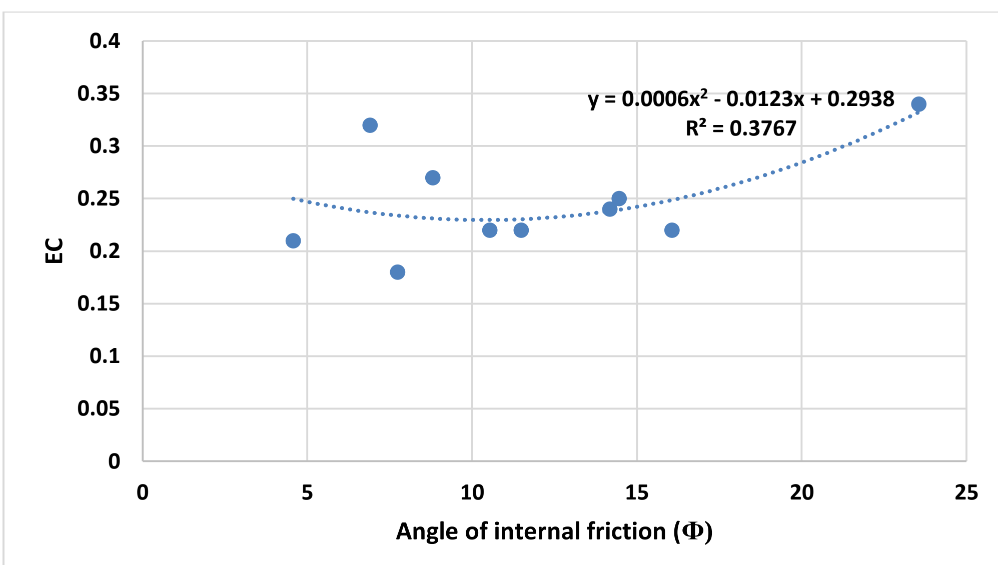

Figure 4.60: Relationship between EC and angle of internal friction

CHAPTER 5

5. CONCLUSION AND RECOMMENDATIONS

5.1. Conclusion

Following conclusion are drawn from this study.

- Based on Atterberg limits, Nandipur and Layyah soils show maximum and minimum liquid limits of 60 and 19.5, respectively. The shrinkage limit of Nandipur soil is also higher than other soils. The higher liquid and shrinkage limits of Nandipur soil are due to the higher %age of clay size contents while Layyah soil have minimum amount of clay contents. Furthermore, the liquid limit and plasticity index are a function of clay contents
- Highly plastic soil (Nandipur soil) provides maximum dry density with lower peak at higher water content than other soils. The sandy soils, such as ML, SC and SP require less water to reach at maximum dry densities. The maximum dry density decreased with an increase in %age clay size particles providing a strong correlation.
- The maximum dry density decreased and optimum moisture content increased with an increase in plasticity index, providing strong relationships.
- The consolidation tests showed that compression and swelling indices showed strong relationship with Atterberg limits, %age clay size contents, and optimum moisture contents. There is strong direct relationship exist between compression index and recompression index ($C_r = 0.0409(C_c)^{0.5821}$ with R^2= 0.8781). Furthermore, the coefficient of consolidation provided a strong inverse relationship with Atterberg limits as well.

- The triaxial test showed that high plastic soil (CH) provided more cohesion and less angle of internal friction while low plastic soil (CL) relatively provided less cohesion and higher angle of internal friction. The %age clay size particles provided strong direct and inverse relationships with cohesion and angle of internal friction, respectively ($C = 5.40007 \ln(\text{clay size particle}) + 3.4832$ with $R^2 = 0.9626$ and $\theta = 34.031(\text{clay size particle})^{-0.463}$ with $R^2 = 0.9693$).
- CBR test results showed that CBR showed strong inverse relationship with specific gravity, Atterberg limit, optimum moisture content, cohesion and direct relationship with maximum dry density and angle of internal friction.

5.2. Recommendations:

Following recommendations are suggested from the study on the basis of experimental test data.

1. There is a need to perform the direct shear and consolidated drained triaxial tests for the comparative evaluation of the test data, extensively.
2. Some admixture can be used in weaker soils to improve their geotechnical properties for use in infrastructure projects.

References

AASHTO T 193-99. Standard method of test for the California Bearing Ratio .

Abdullah et al. (2018). Role of compaction energy on dry density and CBR. International Research Journal of Engineering and Technology , 1848-1852.

Akayuli et al. (2013). The influence of observed clay content on shear strength and compressibility of residual sandy soils. International Journal of Engineering Research and Applications , 2538-2542.

Alayaki and Bajomo. (2011). Effect of moisture variation on the strength characteristics of laterite: A case proceedings of the environmental management conference. Abeokuta, Nigeria.

Ali et al. (2019). A correlation between compaction characteristics and soil index properties for fine-grained soils. Polytechnic Journal. , 93-99.

Alias et al. (2014). Particle size effect on shear strength of granular materials in direct shear test. International Journal of Civil, Architectural, Structural and Construction Engineering .

Al-Obaidi et al. (2020). Effect of relative compaction on the bearing capacity of cohesive soils. Materials Science and Engineering. IOP Publishing.

Amit and Dedalal. (2004). The role of plasticity index in predicting compression behaviour of clays. The Electronic Journal of Geotechnical Engineering .

Arpan and Sujit. (2012). Geotechnical characteristics of two different soils and their mixture and relationships between parameters. Retrieved from Http://www.ejge.com/index.htm.

ASTM D 2216-05. Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass .

ASTM D 2435-03. Standard test methods for one-dimensional consolidation properties of soils using incremental loading .

ASTM D 422-63. Standard test method for particle-size analysis of soils.

ASTM D 4318-05. Standard test methods for liquid limit, plastic limit, and plasticity index of soils.

ASTM D 4767-04. Standard test method for consolidated undrained triaxial compression test for cohesive soils.

ASTM D 4972-01. Standard test method for pH of soils .

ASTM D 698-07. Standard test methods for laboratory compaction characteristics of soil using standard effort.

ASTM D 854-06. Standard test methods for specific gravity of soil solids by water pycnometer.

Atkinson (1993). The mechanics of soils and foundations. Taylor Francis.

B Widjaja and K Kurniawan. (2020). Development of relationship between liquid limit and clay. Materials Science and Engineering. IOP Publishing.

Bowles. (2012). Engineering properties of soils and their measurements 4th edition. Mc Graw-Hill.

Carter and Bentley. (1991). Correlation of soil properties. London: Pentech Press Limited.

Castellanos and Brandon. (2013). A comparison between the shear strength measured with direct shear and triaxial. Paris: Proceedings of the 18th International conference on Soil Mechanics and Geotechnical Engineering.

Dafalla. (2013). Effects of clay and moisture content on direct shear tests for clay-sand mixtures. Advances in Materials Science and Engineering .

Dolinar, B. (2010). Predicting the normalized, undrained shear strength of saturated fine-grained soils. Applied Clay Science , 428-432.

Grim. (1953). Clay Mineralogy. New York: Mc Graw-Hill.

Gunduz and Arman. (2007). Possible relationships between compression and recompression indices of a low–plasticity clayey soil. Arabian Journal For Science and Engineering .

Hassan et al. (2017). Correlation of maximum laboratory dry density and optimum moisture content of soil with soil parameters. NFC-IEFR Joural of Engineering & Scientific Research .

Hemu et al. (2018). Correlation between chemical and index properties of soils. Civil Engineering Journal .

Holtz and Kovacs. (1981). Introduction to geotechnical engineering. New Jersey: Englewood Cliffs.

Ijimdiya and Igboro. (2012). The Compressibility Behavior of Oil Contaminated Soils . The Electronic Journal of Geotechnical Engineering .

Ikeagwuani et al. (2018). Correlation between maximum dry density and cohesion of remoulded nsukka clays. Nigerian Journal of Technology , 13-18.

Jaleel. (2011). Effect of soaking on the CBR-value of subbase soil. ENG. & TECH. Journal .

Janbu. (1970). Grunnlag I Geoteknikk. Tapir.

Khalid and Rehman. (2018). Evaluation of compaction parameters of fine-grained soils using standard and modified efforts . International Jounal of Geo- Engineering .

Khalkhali and Mirghasemi. (2009). Numerical and experimental direct shear tests for coarse-grained soils. Particuology , 83-91.

Khan et al. (2016). Analysis of california bearing ratio under soaked condition for RDFS. International Journal of Engineering Research .

Korde and Yadav. (2015). A study of correlation between CBR value and physical properties of some soils. International Journal of Emerging Technology and Advanced Engineering , 237-239.

Kordnaeij et al. (2015). Prediction of recompression index using gmdh-type neural network based on geotechnical soil properties. Soils and Foundations , 1335-1345.

Kurnaz et al. (2016). Prediction of compressibility parameters of the soils using artificial neural network. Springer Plus .

Majdi et al. (2019). Determination of the soil compression index (Cc) in clayey soils using shear wave velocity (Case Study: Tabriz City). Iranian Journal of Science and Technology, Transactions of Civil Engineering , 577-588.

Majid and Azam. (2013). Unsaturated shear strength behavior under unconsolidated undrained tests. The Electronic Journal of Geotechnical Engineering .

Mitchell and Soga. (1993). Fundamentals of Soil Behavior. John Wiley & Sons.

Momeni et al. (2020). Laboratory Investigation On The Effects of pH Induced Changes on Geotechnical Characteristics of Clay Soil. Geomechanics and Geoengineering .

Mun et al. (2016). Rate effects on the undrained shear strength of compacted clay. Soils and Foundations , 719-731.

Najme et al. (2019). Effect of pH changes on the geotechnical properties of clay liners in landfill. Advance Researches in Civil Engineering , 49-57.

Nguyen et al. (2020). Research on horizontal coefficient of consolidation of Vietnam’s. Hindawi Journal of Engineering .

Obasi and Anyaegbunam. (2005). Correlation of the undrained shear strength and plasticity index of tropical clays. Nigerian Journal of Technology .

Oyediran and Durojaiye. (2011). Variability in the geotechnical properties of some residual clay soils from southwestern nigeria. International Journal of Scientific & Engineering Research .

Priyadarshini et al. (2015). Study on Consolidation and Correlation With Index Properties of Different Soils In Manipur Valley. International Journal of Engineering Research and Development , 57-63.

Rakaraddi and Gomarsi. (2015). Establishing Relationship Between CBR With Different Soil Properties. International Journal Of Research In Engineering and Technology .

Rakesh and Jain. (2016). Prediction of compression index (Cc) of fine grained remolded soils from basic soil properties. International Journal of Applied Engineering Research , 592-598.

Rashed et al. (2017). Correlation of consistency and compressibility properties of soils in Sulaimani city. Sulaimani Journal For Engineering Sciences .

Rashmi and Desai. (2010). CBR predicted by index properties for alluvial soils of south gujarat. Indian Geotechnical Conference. Mumbai: GEOTRENDZ.

Reddy et al. (2019). Prediction of california bearing ratio through empirical correlations of index properties for tropical indian soils. International Journal of Innovations In Engineering And Technology .

Rehman et al. (2017). Prediction of CBR value from index properties of different soil. Technical Journal, University of Engineering and Technology (UET) Taxila, Pakistan .

Reza and Mehrab. (2014). Effect of normal stress and relative compaction on secant friction angle of sands. Turkish Journal of Engineering & Environmental Sciences , 382-391.

Robinson and Allam. (1998). Effect of Clay Mineralogy on Coefficient of Consolidation. Clays and Clay Minerals , 596-600.

Roksana et al. (2018). Relationship between CBR and soil index properties of bangladesh soil samples. Journal of Modern Science and Technology , 1-9.

Sabrin et al. (2021). Correlation of compression index with index properties of soil samples from several places in chattogram, bangladesh. International Research Journal of Engineering and Technology , 752-755.

Schmitz et al. (2004). Chemo–Mechanical interactions in clay: A correlation between clay mineralogy and atterberg limits. Applied Clay Science , 351-358.

Scott and Ronald. (1963). Principles of soil mechanics. London: Applied Science Publishers Ltd. London.

Sen and Pal. (2014). Index properties of soils collected from different locations and correlations of parameters. 3443-3452.

Sharma et al. (2020). Prediction of california bearing ratio through empirical correlations of index properties for tropical indian soils. International Journal of Innovations In Engineering and Technology .

Shien et al. (2018). Prediction of consolidation characteristics from index properties. E3S Web of Conferences. EDP Sciences.

Skempton. (1953). The colloidal ‘activity’ of clays. Proceedings of the 3rd international conference on soil mechanics and foundation engineering, (PP. 57-60). Switzerland.

Smith et al. (1985). Shrinkage And Atterberg Limits In Relation To Other Properties of Principal Soil Types in Israel. Geodarma , 47-65.

Smith et al. (1997). The effect of soil compaction and soil physical properties on the mechanical resistance of south african forestry soils. Geoderma , 93-111.

Soga and Mitchell. (1993). Fundamentals of soil behavior. John Wiley & Sons.

Sridharan and Nagaraj. (2012). Coefficient of consolidation and its correlation with index properties of remolded soils. Geotechnical Testing Journal .

Sunil et al. (2006). Effect of pH on the geotechnical properties of laterite. Engineering Geology , 197-203.

Takaharu and Misao. (1994). Effects of sample disturbance on strength and consolidation parameters of soft clay. Soils and Foundations , 1-10.

Talukdar. (2014). A study of correlation between california bearing ratio. International Journal of Emerging Technology and Advanced Engineering .

Thair and Sarapaa . (2008). Clay and clay mineralogy. Geological Survey of Finland.

Tuncer and Lohnes. (1977). An engineering classification for certain basalt-derived lateritic soils. Engineering Geology , 319-339.

Tyler and Wheatcraft. (1992). Fractal scaling of soil-particle size distributions. Soil Science Society of American Journal , 362-369.

Velde. (1992). Introduction to clay minerals. Applied Clay Science.

Vikas et al. (2015). Correlation of plasticity index and compression index of soil. International Journal of Innovations in Engineering and Technology .

Vinod and Bindu. (2010). Compression index of highly plastic clays – an empirical correlation. Indian Geotechnical Journal , 174-180.

Vishal and Yadav. (2016). Study of correlation of CBR value with engineering properties and index. International Research Journal of Engineering and Technology , 772-778.

Wei bai et al. (2013). Effects of physical properties on electrical conductivity of compacted lateritic soil. Journal of Rock Mechanics and Geotechnical Engineering , 406-411.

Yashas et al. (2016). Effect of california bearing ratio on the properties of soil. American Journal of Engineering Research , 28-37.

Yilmaz. (2000). Evaluation of shear strength of clayey soils by using their liquidity index. Bull Eng Geol Env , 227-229.

Zolfaghari et al. (2015). Soil atterberg limits and consistency indices as influenced by land use and slope position in western Iran. Journal of Mountain Science , 1471-1483.

Zumrawi and Mohammed. (2016). Correlation of placement conditions and soil intrinsic properties with shear strength of cohesive soils. University of Khartoum.